2025 Arizona Water Association Student Design Competition

CENE 486C, Spring 2025

Final Design Report:
Expansion of SPA 1 Water Reclamation Facility for the City of Surprise

Walnut Canyon Wastewater

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Date: 5/6/2025

Revision: 6

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List of Abbreviations

AZWA- Arizona Water Association

AZWA SDC- Arizona Water Association Student Design Competition

ADMM- Average Day Maximum Month

DIP- Ductile Iron Pipe

BOD- Biochemical Oxygen Demand

CFS- Cubic Feet Per Second

CHP- Combined Heat and Power Generation Station

CSTR- Completely Stirred Tank Reactor

EOPCC- Engineers' Opinion of Probable Construction Cost

EPA- Environmental Protection Agency

HRT- Hydraulic Residence Time

MGD- Million Gallons per Day

MOPO- Manual of Permitted Operations

RAS- Return Activated Sludge

SPA- Special Planning Area

SRT- Solids Retention Time

SS - Suspended Solids

TN- Total Nitrogen

TSS- Total Suspended Solids

WAS- Waste Activated Sludge

WRF- Water Reclamation Facility

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Abstract

The Special Planning Area (SPA) 1 Water Reclamation Facility (WRF), owned and operated by the City of Surprise, Arizona, needs to increase its total treatment capacity from 12.8 to 16.3 million gallons per day (MGD). The City has requested the evaluation of three alternatives: two that handle the increased capacity by modifying how Plants 4 and 5 operate without changing their footprint, and one that adds a new Plant 6. Alternatives were developed and analyzed, and it is recommended to modify the existing Plant 4 and 5 oxidation ditches to operate with two half anaerobic/aerobic independent tanks to remove nitrogen and biochemical oxygen demand (BOD) and improve facility redundancy. The design team also analyzed each treatment process to look for areas to optimize or improve and ensure the facility operates jointly with proposed design changes to secondary treatment. Further recommendations include changing preliminary treatment to incorporate band screens to mitigate damage to brush aerators and retrofitting existing unused aerobic digestors into anaerobic digestors as an income source and improve sludge quality. These upgrades will prepare the SPA 1 WRF facility to handle the increased flow that is expected, while leaving space for further expansion.

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Acknowledgements

The Walnut Canyon Wastewater team would like to first and foremost thank our advisor Dr. Jeffrey Heiderscheidt for his guidance, advice, feedback, and support through this process. We would also like to thank Dr. Diana Calvo who provided additional guidance on modeling secondary treatment. Finally, we would like to thank Adias Fostino, the competition chair, as well as her representatives, Katie Stowers and Sam Nieslanik, for organizing this competition and for all their work to ensure we had the information we needed to complete this project.

1 Project Introduction

The following subsections explain the design problem, project background, project constraints/limitations, major objectives, and unique deliverables.

1.1 Design Problem

The City of Surprise's (the City) Special Planning Area 1 (SPA 1) Water Reclamation Facility (WRF) is the largest wastewater treatment facility in the City of Surprise, Arizona and currently processes 8.87 million gallons per day (MGD). The facility currently has five plants each consisting of an oxidation ditch and secondary clarifier with shared headworks, advanced treatment, disinfection, and sludge processing facilities. SPA 1 WRF has a functional capacity of 12.8 MGD with its outdated Plants 1 and 2 offline. It must increase its capacity to 16.3 MGD per state permitting as the city is nearing 80% of its permitted capacity, requiring additional equipment or reconditioning of existing equipment. The City wishes to accomplish increasing its capacity through reconditioning Plants 4 and 5 [1].

For secondary treatment, the City requested two alternatives be evaluated to modify the existing Plants 4 and 5 oxidation ditches to increase their treatment capacity without changing their footprint. The City also requested an alternative to assess the addition of a sixth new plant. Additional alternatives were generated for all treatment processes to ensure that the whole plant is able to function under the increased flow capacity and with any design changes made to secondary treatment.

With increased loading, the sizing and hydraulics of all existing equipment were assessed including: piping, pumps, headworks, oxidation ditches, clarifiers, disk filters, disinfection contact basins, and solids processing facilities. Systems that will be able to handle the increased loading and remain in permitted effluent levels were left unchanged, but systems that are flawed or unable to handle the increased loading were redesigned. These designs were ideally designed to fit within the facility's existing infrastructural footprint, per the City's request, to allow for further expansion of the facility as the City grows. The City additionally requested an Engineers' Opinion of Probable Construction Cost (EOPCC), estimated Operation and Maintenance (O&M) cost, a construction sequencing plan that would allow for continuous operation at the facility, a new hydraulic profile, a new site layout, and a new process flow diagram.

1.2 Project Background

The project site is located in the City of Surprise, Arizona which is northwest of Phoenix as seen in *Figure 1-1*, below.

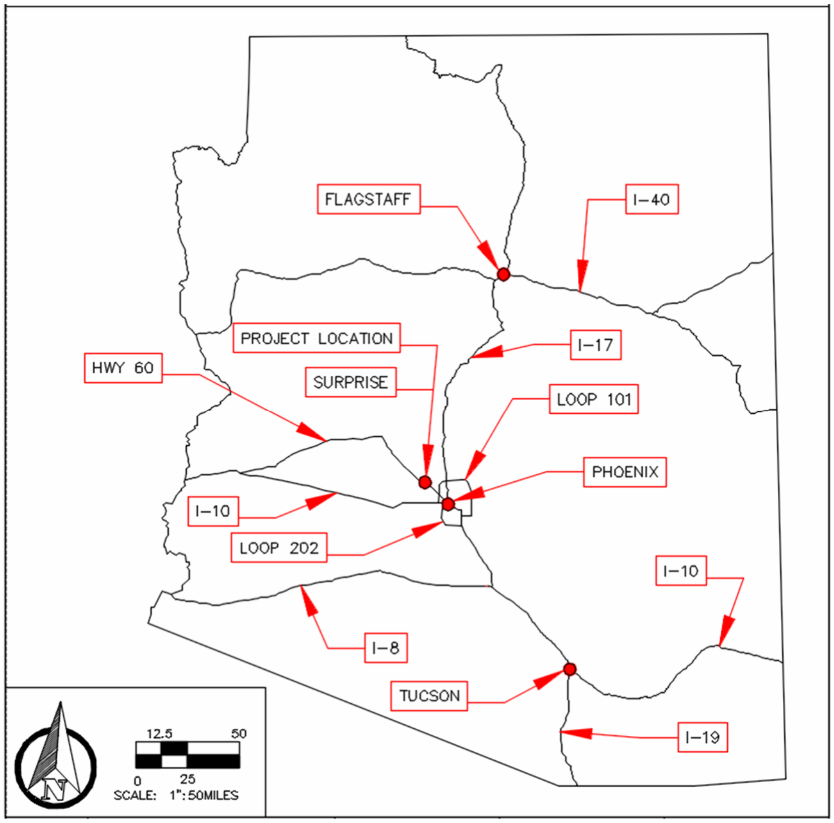

Figure 1-1: Location Map

More specifically, the SPA 1 WRF's address is 13663 Cactus Rd, Surprise, AZ. *Figure 1-2*, below, shows a vicinity map of the project site.

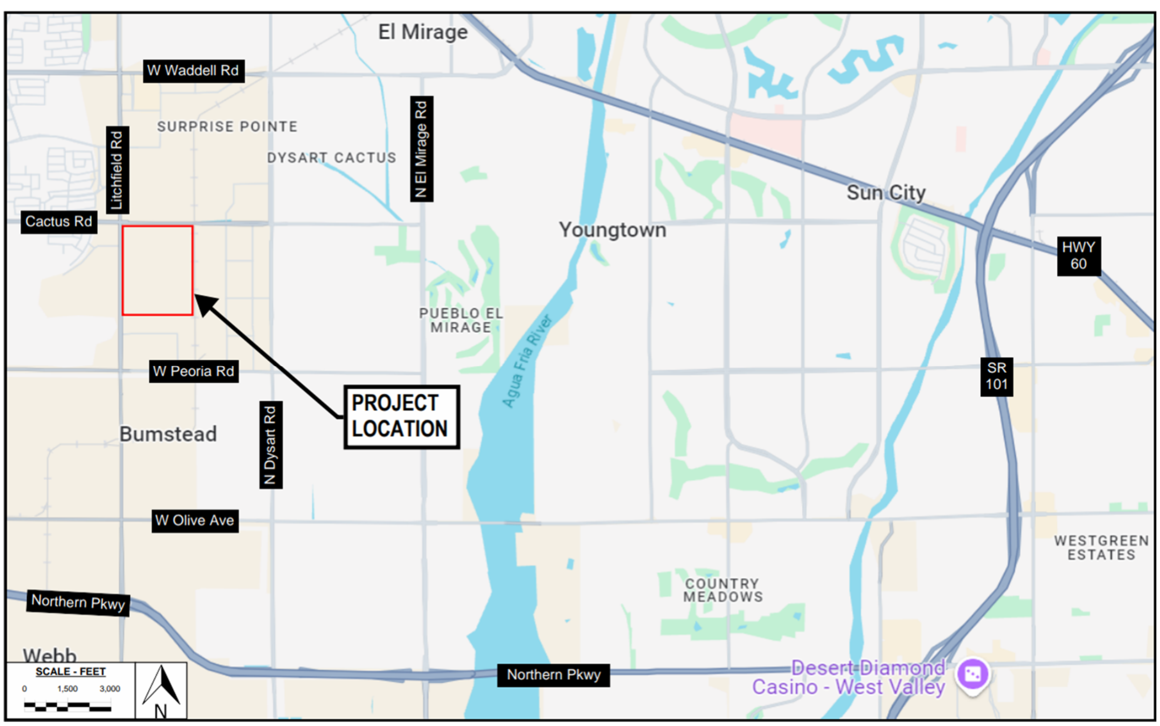

Figure 1-2: Vicinity Map [1]

The project site is surrounded by residential and commercial development. The City of Surprise is divided into six special planning areas (SPA 1 through SPA 6); this project focuses specifically on expansion and improvements for SPA 1 WRF.

Originally constructed in 1995, the City of Surprise SPA 1 WRF has undergone many expansion projects to get to the present-day condition. The current functional capacity of SPA 1 is 12.8 MGD with Plants 1 and 2 offline, refer to *Figure 1-3*. SPA 1 produces Class A+ reclaimed water which is reused either directly for landscaping and irrigation or through recharge via spreading basins for later recovery. Additionally, SPA 1 WRF produces Class B biosolids.

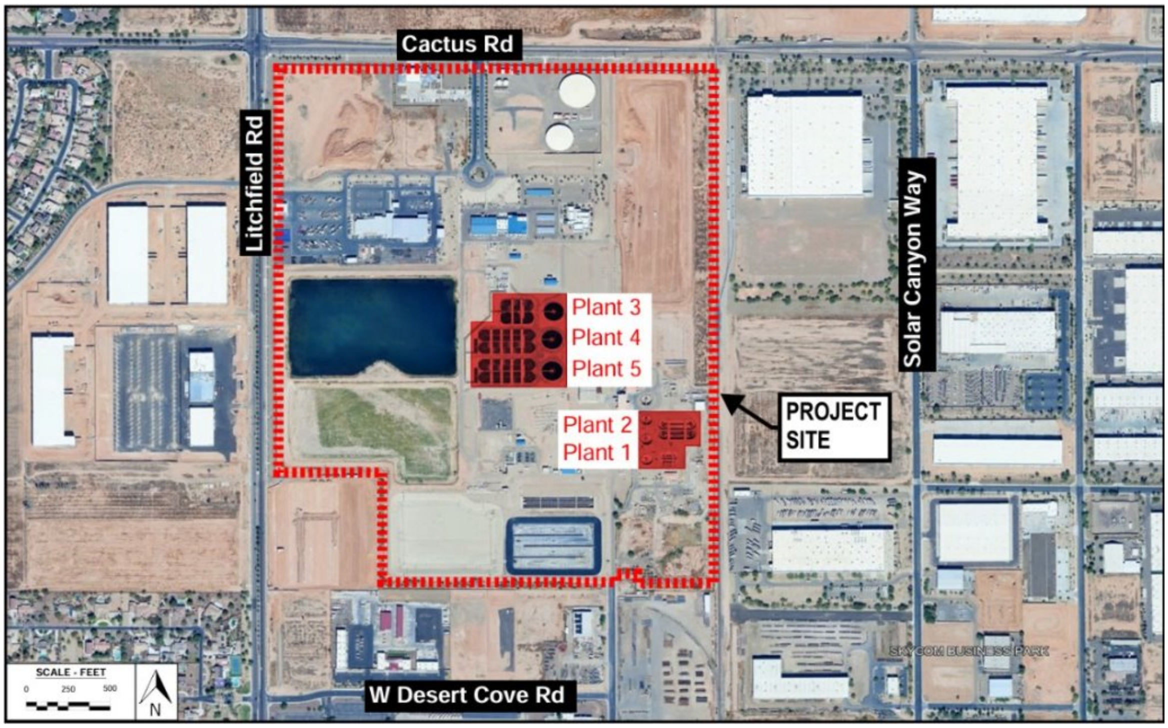

Figure 1-3: Project Site Map [1]

1.3 Constraints/Limitations

Modifications to Plants 4 and 5 cannot change the footprint of the plants. SPA 1 is surrounded by residential and commercial properties; this requires strict odor and noise control to keep the surrounding community content. New construction and operation of infrastructure are not permitted to exceed any existing noise or odor pollution to surrounding environments. Lastly, SPA 1 WRF contains several non-operational facilities that limit the available space and may require additional costs to demolish.

1.4 Major Objectives of Project and Unique Deliverables

Major objectives of the project include selecting the best alternatives using decision matrices for preliminary treatment, primary treatment, secondary treatment, advanced treatment, disinfection, and solids management. Final designs for the selected alternatives for each treatment step were created.

Unique deliverables include the AZWA SDC final report and presentation. The report is a 20-page in-depth discussion of the design problem, alternatives evaluated, and recommended design solution. The 20-minute presentation was given at the Arizona Water Conference detailing all engineering work done, supporting reasoning behind decisions, and giving a full description of the final design solution.

2 Analysis Performed of Existing Plant

A site visit was conducted on January 29th, 2025 at the SPA 1 WRF. The objective of the visit was to evaluate the infrastructure at the site, identify the model and manufacturer of key equipment, capture reference photographs for design purposes, obtain missing design information, and observe the wastewater flow throughout the facility. Photos taken during the site visit are found in *Appendix A: Site Visit Photo Log.*

Area takeoffs were performed using Bluebeam [2] on the scaled drawings provided by the Arizona Water Association Student Design Competition (AZWA SDC). The surface areas of the secondary clarifiers and oxidation ditches of Plant 4 and 5 were obtained in *Table 2-1*. These area measurements were utilized in the sizing of the oxidation ditches to ensure adequate nitrogen and BOD removal.

Table 2-1: Area Takeoffs of Plants IV and V

Area Takeoffs	Plant IV and V
Oxidation Ditches (ft^2)	21,764
Secondary Clarifier (ft^2)	12,727

The existing hydraulic profile was analyzed to assess the current water surface elevations and identify critical hydraulic areas within the treatment process. Additionally, the existing process flow diagram was analyzed to understand the flow type, size, and direction from each process of the treatment train. The existing hydraulic profile and process flow diagram, both provided by the AZWA SDC [3], are included in *Appendix B: Existing Hydraulic Profile* and *Appendix C: Existing Process Flow* Diagram. The existing pipes and open channels were analyzed to confirm they can handle the increased flows; this analysis is included in *Appendix P: Hydraulics Calculations*.

The existing facility configuration includes three operating wastewater treatment plants: Plants 3, 4, and 5. The following design criteria are shown in *Table 2-2* [3].

Table 2-2: Existing Design Parameters

Facility		
	Plant III	**Plant IV and V**
Peak Flow Rate, mgd	12	10
ADMM, mgd	4.8	4
Peak Hour Factor	2.08	2.5
ADMM BOD_5, mg/L	264	300
ADMM TSS, mg/L	270	300
ADMM TKS, mg/L	54	72

The existing oxidation ditch operates through Krueger's proprietary BioDenitro Process across four phases [3]. These four phases rely on influent BOD as a carbon source to facilitate both nitrification and denitrification. The plants utilize jet aerators or brush aerators to ensure that dissolved oxygen levels meet the requirements of the microorganisms, continuously adjusting to maintain optimal conditions.

The existing preliminary treatment includes fine screens followed by a grit removal process. During operation, rags and flushable wipes have been observed to pass through and cause damage to the brush aerators in Plants 4 and 5. This treatment system is rated for a 16.3 MGD, accommodating the combined flow of all plants (1-5).

The preliminary, advanced, disinfection, and sludge treatment processes are currently rated to accommodate the combined flow of 16.3 MGD, with all plants in operation. These processes do not require any increases in sizing at this time, but alternatives were still evaluated to ensure that they would work in tandem with any changes made to secondary treatment.

3 Evaluation of Alternatives

The following sections cover each alternative analyzed for each treatment process at the SPA 1 WRF with the goal of optimizing treatment or increasing capacity. Each alternative evaluated in a decision matrix was given a weighted score 1-5 with 1 being the worst and 5 being the best. The alternative with the highest score in each decision matrix was selected for final design.

3.1 Preliminary Treatment Evaluation

Preliminary treatment is intended to remove initial large solids and debris from the raw wastewater coming into the facility. The following sections detail preliminary alternatives for the SPA 1 WRF. The alternatives considered include: performing no technology changes, switching to a band screen (In-to-Out) screening system, and adding a grinder in addition to the existing fine screen treatment. Vendor information for each piece of equipment analyzed is included in *Appendix D: Vendor Information*.

3.1.1 No Change to Treatment Technology

The current equipment being used for screening is the JWC Environmental Finescreen Monster with 3mm perforations. Fine screens work by having continuous perforated panels to capture and lift solids and debris out of the raw wastewater; refer to *Appendix A: Site Visit Photo Log* for pictures of existing fine screens.

Not changing the equipment used in preliminary treatment will save the plant on initial capital costs, but the existing equipment will end up costing more in operations and maintenance in the long run. The existing fine screens are inefficient in removing rags and unwanted debris from entering downstream processes and damaging downstream equipment. Brush aerators used in the oxidation ditches are the most common piece of infrastructure damaged because of the inefficient removal of the existing fine screens.

3.1.2 Band Screen

A band screen is a preliminary treatment system that works in a similar way as the existing fine screens used on site. However, band screens possess a different screening and cleaning system which offer a high removal efficiency of particles and solids and prevent their reintroduction to the waste stream. Refer to *Figure 3-1* for an example of how a band screen operates. The make and model intended for use is the JWC Environmental Bandscreen Monster with 3-millimeter perforations and a rated capacity of 7 MGD [4].

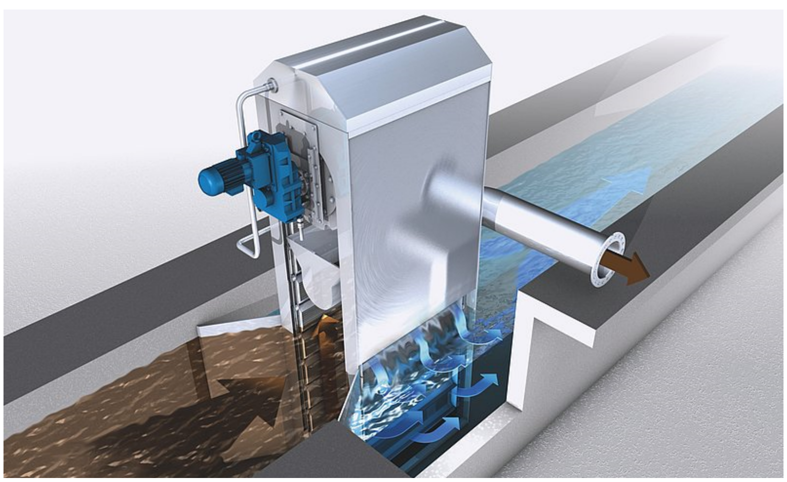

Figure 3-1: Band Screen Operation Visual Aid [5]

3.1.3 Addition of Grinder to Preliminary Treatment

A grinder is a rotating slotted cylinder that focuses on reducing the size of solids and debris in raw wastewater. Reducing the size of these large particles allows for the particles to settle out in processes further down the line;

this process works especially well when a primary clarifier is in the treatment train. SPA 1 WRF does not have a primary clarifier which means implementing a grinder will not yield optimal usefulness.

3.1.4 Preliminary Treatment Selection

The criteria used in the preliminary treatment decision matrix included lifecycle costs, removal efficiency, minimizing construction time, and adaptable capacity. Refer to *Appendix E: Criteria for Scoring Decision Matrices* for how each criterion is scored. Life cycle costs and removal efficiency were weighted the highest to ensure each alternative made sense from an economic perspective and proper consideration was given to prevent damage in downstream systems. Minimizing construction and adaptable capacity were weighted the same since construction is important for a project of this scale, yet designing the system to be adaptable is needed to ensure the facility can maintain operation in unforeseen circumstances. In the end, implementing band screens was chosen as the best option for the final design. A simplified decision matrix is found in *Table 3-1*, while a detailed decision matrix can be found in *Appendix F: Detailed Decision Matrices.*

Table 3-1: Preliminary Treatment Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Band screen from JWCE	Alternative 3: Add grinder to the preliminary treatment process
Life Cycle Costs (Capital Cost and O&M)	30%	3	2	1
Removal Efficiency	30%	1	3	4
Minimizing Construction Time	20%	5	4	3
Adaptable Capacity	20%	1	3	2
Total	100%	2.4	2.9	2.5

3.2 Primary Treatment Evaluation

The following sections consider primary treatment options at the SPA 1 WRF. This facility currently does not have a primary treatment process.

3.2.1 No Change to Current Treatment Train

No analysis was performed for this alternative.

3.2.2 Addition of Primary Clarifier to Treatment Train

Primary treatment technologies remove TSS and biochemical oxygen demand (BOD) within the wastewater. BOD is a required substrate for denitrifying organisms but in excess a primary settler can help remove excess BOD. For the oxidation ditches in Plants 4 and 5, BOD was determined to be a limiting substrate for denitrification. It is therefore important for denitrification to allow as much BOD through to secondary treatment. A secondary clarifier will decrease the sludge content by 5% and remove BOD by up to 20%, but this in turn will make the process of nitrification and denitrification more difficult for the microorganisms. The primary clarifier will cost approximately $1.5 million in capital cost (adjusted for inflation) and will take several months to implement for construction [6].

3.2.3 Primary Treatment Selection

The three criteria used to judge the primary treatment alternatives were life cycle costs, downstream effects, and minimizing construction time. Refer to *Appendix E: Criteria for Scoring Decision Matrices* for how each criterion was scored. Downstream effects was weighted the highest in this decision since a primary clarifier that has few or no improvements on downstream systems is not worth the large investment.

Based on analysis of the oxidation ditches it was determined that a primary clarifier is detrimental to secondary treatment due to the decrease in BOD. This option was ultimately deemed unfeasible. *Table 3-2*, below, shows the scored decision matrix consisting of whether or not to add a primary clarifier. The detailed decision matrix can be found in *Appendix F: Detailed Decision Matrices.*

Table 3-2: Primary Treatment Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Primary clarifier
Life Cycle Costs (Capital Cost and O&M)	30%	3	1
Downstream Effects	40%	1	2
Minimizing Construction Time	30%	5	2
Total	100%	2.8	1.7

3.3 Secondary Treatment Evaluation

The purpose of secondary treatment is to remove biodegradable pollutants, namely BOD and nitrogen. BOD is removed through the growth of microorganisms in the mixed liquor who use the compounds that comprise BOD as substrate. Nitrogen is removed in a two-step process where firstly aerobic organisms convert ammonia (NH_4) to nitrite (NO_2) and ultimately to nitrate (NO_3) (Nitrification). Finally, anaerobic organisms convert the nitrate to harmless nitrogen gas (N_2) (Denitrification). Initial assessments of secondary treatment found that the rate limiting process was nitrification and not BOD removal. It was also found that that the influent soluble BOD did not provide enough substrate to completely denitrify all influent nitrogen. This meant that an ideal design would utilize as much influent BOD for denitrification as possible to limit the effluent total nitrogen (TN) concentrations. Current conditions are effective at supplying BOD to the denitrification process. However, the key weakness identified in the efficiency of current operation under Krueger's proprietary BioDenitro mode is the time it takes to cycle through its various phases. Each tank under this mode has a phase where it is simply batching for a prolonged period where it is neither producing effluent nor taking on new influent. There is additionally a loss in efficiency each time a tank must change between aerobic and anaerobic conditions. It was determined that a more efficient and higher capacity operation method should not include a phasing schedule.

The following sections will evaluate three alternatives for increasing the design capacity of secondary treatment for SPA 1 WRF. The three alternatives include converting Plants 4 and 5 oxidation ditches to conventional oxidation with denitrification, converting Plants 4 and 5 to sequential aerobic and anaerobic tanks, and the addition of a sixth plant.

3.3.1 Conventional Oxidation with Denitrification

This alternative, as seen in *Figure 3-2*, separates the tanks of each plant. Each tank then operates with one anaerobic pass, which is fed by the influent, and one aerobic pass. This alternative directs influent BOD to the anaerobic pass, providing the necessary substrate for denitrification. This process also removes the bulk of soluble BOD. The aerobic pass is largely focused on nitrification due to the low amount of BOD entering the pass, and the nitrate produced is reintroduced to the anaerobic pass via internal recycle within each tank. This alternative would have limited construction time and costs, limited increases to O&M, and improve facility redundancy. However, because there is not enough substrate to completely denitrify, the effluent TN concentrations will be higher than other alternatives, albeit still within permit levels.

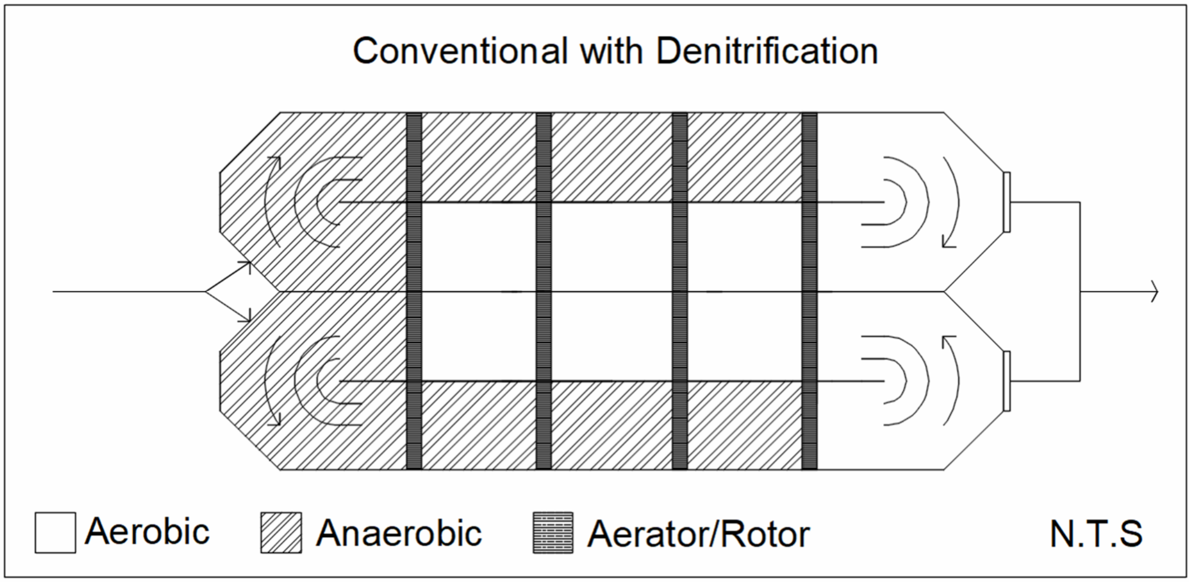

Figure 3-2: Conventional Oxidation with Denitrification Sketch

3.3.2 Sequential Aerobic and Anaerobic Tanks

This alternative, as seen in *Figure 3-3*, is similar to the conventional alternative but has an aerobic tank which is fed with influent and eventually drains into a second anaerobic tank. The anaerobic tank could not be placed first in this alternative since there is no internal recycle and nitrification must be accomplished before denitrification. This means that much of the BOD would be consumed in the first tank, and that a BOD feedstock, likely methanol, would be required to supply enough substrate for denitrification. The downside of this would be an increased cost to install a methanol feeding system and the ongoing cost of purchasing methanol, but it would have the upside that more denitrification could be accomplished, decreasing the effluent TN.

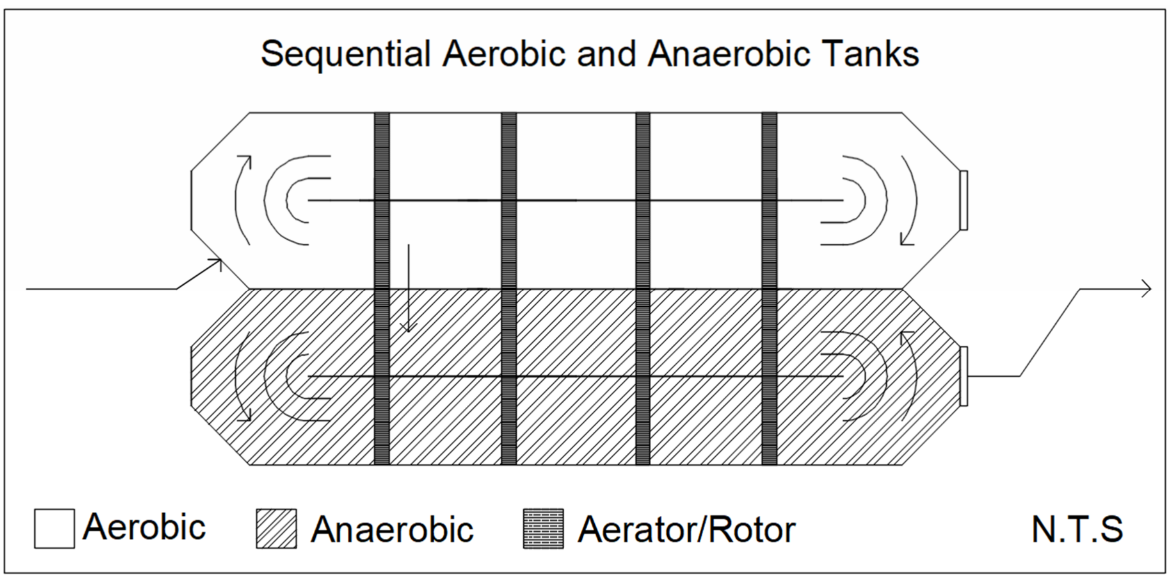

Figure 3-3: Sequential Tank Sketch

3.3.3 Addition of a Sixth Plant

The final alternative would involve adding a sixth plant that would be comprised of two parallel oxidation ditches and a secondary clarifier identical to Plants 4 and 5. The new plant would be operated like Plants 4 and 5 using Krueger's proprietary BioDenitro mode [3]. This would be able to add a permitted capacity of 4 MGD, exceeding the required increase of 3.5 MGD. This alternative would require a high capital cost to install the new systems and would require a much longer construction time. Additionally, a sixth plant would require a higher O&M cost to operate and maintain the new required aerators and pumps. The cost of a new sixth plant was estimated using RSMeans [7] and similar engineering projects. Its effluent quality would be identical to current conditions, which is within permit levels. The addition of a new clarifier would help reduce solids loading across all clarifiers compared to other alternatives. This would also likely reduce total suspended solids (TSS) loading on the downstream disk filters compared to other alternatives. The head loss that is created because of new pipes for a new sixth plant were modeled to be identical through pipe size iteration to maintain the current hydraulic profile at the facility. *Appendix G: Plant 6 Analysis* contains the cost and hydraulic analysis performed for this alternative.

3.3.4 Secondary Treatment Selection

Each secondary treatment alternative was graded based on five criteria; the criteria were: capital cost, O&M and lifecycle costs, ability to meet permit limits, minimizing construction time, and adaptable capacity. The first four criteria were required, and adaptable capacity was selected by the design team to give credit to an alternative that improved redundancy, helped mitigate peak flow, or set up for further expansion. Each criterion was scored 1-5 with 1 being the worst and 5 being the best. A summary of how each criterion was scored can be found in *Appendix E: Criteria for Scoring Decision Matrices.* Each criterion was also given a weight based on the relative importance of that criteria. The highest weight of 25% was given to O&M and lifecycle as well as capital costs since cost was determined to be of critical importance. It is also important to balance the immediate and long-term costs. Minimizing construction time was also given the highest weight of 25% since it was deemed important to finish construction quickly to limit disruption to the facility's ongoing operations. Ability to meet permit limits was given a 15% because while it is nice if effluent quality is better than permit requirements, all alternatives would at minimum meet permit levels. Finally, the lowest weight of 10% was given to adaptable capacity because while it was deemed to be a benefit of a design, it was not something that was essential to the City. The full decision matrix can be found in *Appendix F: Detailed Decision Matrices*. A summary decision matrix can be found in *Table 3-3*.

Table 3-3:Secondary Treatment Selection Summary Decision Matrix

Criterion	Weight	Alternative 1: Conventional oxidation with denitrification	Alternative 2: Sequential aerobic/anaerobic tanks	Alternative 3: Addition of a sixth plant
Capital Cost	25%	4	4	1
O&M and Lifecycle Costs	25%	3	2	1
Ability to Meet Permit Levels	15%	1	2	1
Minimizing Construction Time	25%	5	5	2
Adaptable Capacity	10%	3	1	4
Total	100%	3.45	3.15	1.55

Using this decision matrix, it was decided to move forward with conventional oxidation with denitrification. This alternative was selected largely due to its low capital costs, and limited changes to O&M costs, along with its improvement of facility redundancy.

3.4 Advanced Treatment Evaluation

For advanced treatment three alternatives were evaluated. Alternative 1 was continued use of disk filters with no change. Alternative 2 involved continued use of disk filters and reincorporating existing but unused sand filters for additional treatment. Alternative 3 was the installation of membrane filters to treat the water to drinking water quality. These alternatives were scored on a decision matrix based on four weighted criteria: lifecycle costs (capital and O&M), water quality, minimizing construction time, and downstream effects. Lifecycle costs was given the highest weight of 40% since the disk filters are adequate and an alternative would need to be cost effective to justify replacing them. Minimizing construction time was given the next highest weight since minimizing construction time is essential for limiting disruptions to ongoing operations. Water quality and downstream effects were given the lowest weights of 20% and 15% respectively because while they were considered important, the water quality coming from the disk filters is already adequate and there are currently no negative effects on disinfection or distribution, therefore a viable alternative would need to make significant improvements.

Analysis revealed that Alternative 2 would have limited effects on effluent quality as disk filters are typically more efficient than sand filters. Alternative 3, on the other hand, would significantly improve effluent quality, but would have significant capital costs and would increase O&M. A summary decision matrix can be seen in the following table, while the full decision matrix can be found in *Appendix F: Detailed Decision Matrices*.

Table 3-4: Advanced Treatment Summary Decision Matrix

Criterion	Weight	Alternative 1: No change	Alternative 2: Reincorporate existing sand filter	Alternative 3: Upgrade effluent to drinking water quality
Lifecycle Costs (Capital and O&M)	40%	3	2	1
Water Quality	20%	1	1	5
Minimizing Construction Time	25%	5	3	2
Downstream Effects	15%	1	3	4
Total	100%	2.8	2.2	2.5

Based on the results of this decision matrix it was determined that the benefits of Alternatives 2 and 3 do not outweigh their costs, and that no changes should be made to advanced treatment at this time.

3.5 Disinfection Evaluation

The purpose of disinfection is to eliminate pathogenic microorganisms that are present in wastewater. Disinfection can be accomplished using several different strategies. Currently, SPA 1 WRF utilizes chlorine injection in combination with a chlorine contact basin. The following sections will evaluate three different alternatives for disinfection. The alternatives include no change to the disinfection technology, ultraviolet (UV) disinfection, and ozone disinfection.

3.5.1 No Change to Treatment Technology

SPA 1 WRF generates their own supply of chlorine using an on-site sodium hypochlorite generation system. Specifically, the Microclor OSHG from CleanWater1. A photo of the site's current Microchlor system can be found in *Appendix A: Site Visit Photo Log*. The system takes in brine, which is water with a high salt (NaCl) content, the brine is subjected to an electrical current and electrolysis takes place. The salt is ionized producing sodium hypochlorite. The process produces two off gases, hydrogen and oxygen, which are released into the atmosphere.

On-site sodium hypochlorite generation is advantageous compared to the traditional buying of chlorine. The sodium hypochlorite concentration is below hazardous threshold limits making it safe to store and handle. In addition, expenses are reduced since no more deliveries are needed. Additionally, SPA 1 WRF is not required by their permit to dechlorinate since they do not discharge to any waterways, further reducing the cost of chlorination [3].

Two contact basins are located on site. Basin #1 has an estimated volume of 7.27 million gallons (MG), and basin #2 has a known volume of 12.8 MG. The basins have enough volume to treat to the increased design capacity of

16.3 MGD. The required chlorine contact time differs depending on injection concentrations, but the volumes of both contact basins are sufficient to handle contact times of several hours, even with the other offline.

3.5.2 UV Disinfection

UV disinfection utilizes high powered beams of UV light to eliminate pathogens in the water. Elimination of the pathogens is a fast-paced process with a contact time ranging from 20-60 seconds. The capital cost of a UV disinfection system with a design capacity of 16.3 MGD is around $1.1 million. Yearly operation and maintenance cost are around $50,000. [8]

3.5.3 Ozone Disinfection

Ozone disinfection works by injecting ozone (O_3) into the water stream. The dissolved ozone will then oxidize pathogens thus eliminating them. Ozone is generated by passing oxygen molecules (O_2) through an electrical current which splits the oxygen molecules into atomic oxygen, from there the oxygen atoms will bind together to form ozone.

For ozone disinfection the existing contact basins would be used. However, ozone works faster than chlorine with a contact time of about 10-30 minutes, increasing the potential capacity of the system [9]. Capital cost of an ozone generator and injection system for a design capacity of 16.3 MGD is estimated to be $250,000. Yearly operation and maintain cost are approximately $138,500 [9].

3.5.4 Disinfection Selection

The criteria used to evaluate disinfection alternatives were life cycle costs (capital cost and O&M), ability to meet permit limits, minimizing construction time, and contact time. Refer to *Appendix E: Criteria for Scoring Decision Matrices* for how the criteria were scored. Life cycle costs was weighted the highest since existing equipment is highly efficient and cost-effective meaning, for a new alternative to be chosen it would have to save large amounts of money compared to existing systems.

In the end, Alternative 1 was chosen due to the existing sophisticated chlorine production system and the added benefit of not requiring construction. *Table 3-5* shows a simplified decision matrix with the scores of each alternative, while *Appendix F: Detailed Decision Matrices* shows a decision matrix with supporting details as to why each score was assigned.

Table 3-5: Disinfection Simplified Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Ultraviolet disinfection	Alternative 3: Ozone disinfection
Life Cycle Costs (Capital Cost and O&M)	40%	3	1	2
Ability to Meet Permit Limits	25%	1	2	2
Minimizing Construction Time	25%	5	2	4
Contact Time	10%	2	5	4
Total	100%	2.9	1.9	2.7

3.6 Solids Handling Evaluation

Solids handling refers to how biosolid material from waste activated sludge flow is stabilized, reduced in volume and water content, and ultimately disposed of. The following sections will evaluate two different alternatives: no change to current solid handling treatment technologies and retrofitting antiquated aerobic digesters into anerobic digesters.

3.6.1 No Change to Treatment Technology

Existing on-site solids handling equipment consist of dewatering centrifuges that utilize a chemical polymer to aid solids in coagulating and clumping together. The centrifuges generate a 5% dry solids product [10]. After the solids have been centrifuged, they are transported to solar drying beds which dry out the solids to a concentration of 80% dry solids [11]. From there the solids are landfilled.

3.6.2 Retrofit Aerobic Digesters into Anerobic Digesters

Anerobic digestion utilizes the microorganisms already in the sludge to consume themselves and produce biogases, composed largely of biomethane (CH_4), that can be sold or burned on-site to generate energy and heat. An in-depth analysis was undertaken to determine if this alternative is financially feasible.

First, the flow rate of all sludge being wasted per day (Q_w) was calculated using the new oxidation ditch aeration style developed by this team for secondary treatment. This flow rate along with the volume of gases being produced over a 20-day solids retention time (SRT) was calculated to determine the needed volume of the digesters (V_{DIG}). It was found that the volume of the existing aerobic digestors could only treat about 225 m^3/day of the sludge being wasted. Biomethane generated per day (Q_{CH4}) from the 225 m^3/day of sludge being digested was calculated and it was determined that 340.03 m^3/day of biomethane could be produced. Refer to *Table 3-6* for all values calculated for the feasibility analysis to treat all sludge being produced. *Table 3-7* contains gas production values. A more in-depth explanation of equations and parameters used in this analysis can be found in *Section 4.1.3: Solids Handling Design*.

Table 3-6: Feasibility Analysis to Treat all Solids being Produced

Feasibility Analysis	
Q_w (m^3/d) (All Sludge Produced)	726
V_{DIG} (m^3) (Needed)	24,733
V_{DIG} (m3) (Available)	7,665
Vol Available: Vol Needed	31%
Q_w (m^3/d) (Able to be Treated)	225

Table 3-7: Gas Production

Gas Production	
Total Gas Production (m^3/d)	523
Q_{CH4} (m^3/d)	340

It was determined it would be more cost effective to sell the biomethane being produced instead of building an on-site combined heat and power (CHP) generation station due to high capital costs. To estimate the potential savings of selling the biomethane, the energy generation rate of 9.36kWh/m^3 of biomethane was used in combination with Q_{CH4} and the average price of a kWh in Arizona [12]. With an average price of kWh being 0.1356 $/kWh, the savings came out to $157,000/year [13]. A 20-year life cycle cost analysis was created, refer to *Appendix H: Life Cycle Cost Analysis of Anerobic Digesters.* In the end, the digesters will pay for themselves after 13 years and after 20 years the digester will generate a total of $550,000, refer to *Table 3-8* for a concise representation of the 20-year life cycle cost analysis.

Table 3-8: Concise Representation of the 20-year Life Cycle Cost Analysis of Anaerobic Digestion

Life Cycle Cost of Anerobic Digestion and Biogas Production				
	Capital Cost ($)	O & M ($)	Savings ($)	Total ($)
Year 0	$1,000,000	$-	$-	$(1,000,000)
Year 13	$-	$80,000	$157,523	$7,797
Year 20	$-	$80,000	$157,523	$550,457

3.6.3 Solids Handling Selection

The criteria used to evaluate solids handing selection were capital cost, operation and maintenance, ability to meet permit limits, minimizing construction time, and environmental and societal impacts. Refer to *Appendix E: Criteria for Scoring Decision Matrices* for the scoring criteria of each criterion. Operation and maintenance was weighted the highest to favor whichever alternative would save the facility the most amount of money on an annual basis. In the end, Alternative 2 was chosen due to added benefit of revenue generation from selling the biomethane produced. *Table 3-9* shows a simplified decision matrix with the scores of each alternative, while *Appendix F: Detailed Decision Matrices* shows a decision matrix with supporting details as to why each score was assigned.

Table 3-9: Solids Handling Simplified Decision Matrix

Criterion	Weight	Alternative 1: No Change to Treatment Technology	Alternative 2: Retrofit Aerobic Digesters to Anaerobic to be used with Solar Drying Beds
Capital Cost	20%	5	3
O & M and Life Cycle Cost	25%	3	4
Ability to meet permit limits	20%	1	3
Minimizing construction time	15%	5	2
Environmental and Societal Impacts	20%	1	4
Total	100%	2.9	3.3

4 Recommended Design

The following subsections include a description of the recommended designs.

4.1 Recommended Alternative

Based on the analysis described in *Section 3: Evaluation of Alternatives*, the design team recommends the following changes and improvements.

4.1.1 Preliminary Treatment Design

It is recommended that the three existing JWC Environmental Finescreen Monsters at SPA 1 WRF are replaced with three JWC Environmental Bandscreen Monsters. Band screens will eliminate the possibility of rags caught by the screen to be reintroduced into downstream flow [14]. The head loss and O&M difference between the existing fine screens and proposed band screens are assumed to be negligible. The proposed Bandscreen Monsters will have 1/8-inch (3-millimeter) perforations, and each will have a capacity of 7 MGD. The total capacity of the preliminary treatment process with the proposed improvements is 21 MGD. The proposed band screens are compatible with the existing screenings wash system, compaction, and disposal system at the SPA 1 WRF headworks [4].

4.1.2 Secondary Treatment Design

The following subsections detail the recommended secondary treatment design.

4.1.2.1 Design Assumptions

To complete the design of starting operational parameters of Plants 4 and 5 oxidation ditches, several assumptions were made. Firstly, many values were assumed from typical values found in *Water and Wastewater Engineering Design Principles and Practice 2nd Edition* [15] and *Environmental Biotechnology: Principles and Applications* [16]. These values included microbial kinetics coefficients and oxidation ditch operational parameter ranges, among others. One such assumption was to start with a MLSS concentration of 3000 mg TSS/L. This value is on the lower end of typical values for oxidation ditches [15] and was selected to reduce solids loading on the secondary clarifiers. Secondly, each tank was divided into two passes, and each pass was assumed to act like a completely stirred tank reactor (CSTR) due to the high level of internal recycle within each ditch and the fact that flow makes several full passes of the tank during its hydraulic residence time (HRT). It was also assumed that nitrification is the rate limiting process due to the estimated low fraction of nitrifying organisms in the mixed liquor. Therefore, it was determined that the return activated sludge system (RAS) and waste activated sludge (WAS) should be designed around providing nitrification with a sufficient solids retention time (SRT) to completely convert ammonia to nitrate. It was also assumed that the internal velocity within the ditch was the recommended 0.3 m/s which was used to determine the internal flowrate within the ditch [15]. A full list of assumptions can be found in the oxidation ditch hand calculations found in *Appendix I: Secondary Treatment Hand Calculations.*

4.1.2.2 Design Calculations

A full set of hand calculations including equations used and assumptions made can be found in *Appendix I: Secondary Treatment Hand Calculations.* Firstly, it was determined that the influent BOD (using the conservative assumption that no VSS is used as substrate for denitrification) was not sufficient to completely denitrify, leaving a predicted effluent nitrate concentration of 5.83 mg NO_3-N/L. This is likely higher than reality since some VSS can be used as substrate and nitrifying organisms produce some BOD [16]. This informed the decision to place the anaerobic pass first so it could utilize the BOD for denitrification before it was utilized aerobically. Next the effluent ammonia concentration was calculated assuming all ammonia is nitrified in the second pass, and therefore the effluent ammonia exists solely in the first pass which is being fed by the raw influent. This, combined with the predicted effluent nitrate concentration, resulted in a conservative estimate of the effluent's total nitrogen concentration to be 6.6 mg N/L, which is below the facilities average discharge limit of 8 mg/L. The peak discharge allowed for TN is 10 mg/L, which this value is also below.

With the assumption that all ammonia should be converted to nitrate, a solids retention time was determined using Monod kinetics for CSTRs and a safety factor of 2.5 (typical of oxidation ditches) was applied [15]. This resulted in a solids retention time of 22.3 days which again is within the normal range for oxidation ditches [15]. This solids retention time was then used to calculate the predicted RAS TSS concentration and the volume of WAS produced per day.

4.1.2.3 Recommended Operational Parameters

A summary of the recommended operational parameters can be found in *Table 4-1*. These values include expected flow rates for RAS and WAS, as well as the number of rotors that should be used for aeration and how many should be submerged so that they just move flow within the ditch. Because the existing brush aerators will be utilized, a new blower building will not be required. The location of rotors operating each way can be seen in the full visual operational parameter summary found in *Appendix J: Plants IV and V Operational Parameters.*

Table 4-1: Summary of Recommended Operational Parameters

Recommended Operational Parameters for Plants 4 and 5		
	Plant 4	Plant 5
Design Flow (ADMM), MGD	5.75	5.75
Influent TKN, mg/L	72	72
Influent BOD, mg/L	300	300
Influent TSS, mg/L	300	300
MLSS, mg/L	3000	3000
Estimated Effluent TN, mg/L	6.6	6.6
Solids Retention Time, days	22.3	22.3
Return Activated Sludge, MGD	4.31	4.13
Waste Activated Sludge, MGD	0.034	0.034
Suspended Solids in RAS and WAS, mg/L	6600	6660
Number of Rotors Aerating per Tank	4	4

4.1.2.4 Adjusting Operational Parameters

These recommended parameters are based on typical and expected values and should serve as a starting point for converting the oxidation ditches to the new operation style. They should be adjusted by experienced and licensed wastewater operators based on the effluent being produced. After the initial transition to the new operation style, the RAS/WAS flowrates will need to be regularly adjusted to account for seasonal changes and changes to the influent characteristics. The amount of aeration may also need to be adjusted. The predicted number of rotors being operated to aerate in order to provide enough oxygen for full nitrification (using conservative assumptions) was only slightly over 3 rotors (3.08 to be exact.) If effluent nitrate concentrations start to creep up it may be due to more BOD being consumed aerobically than desired. This may require reducing the number of rotors aerating from 4 to 3, submerging the rotor closest to where influent is added. During peak flow, the system may require more oxygen to nitrify. If effluent ammonia concentrations are high during peak flow conditions, it may be necessary to use an additional rotor as an aerator. This rotor should be the one upstream of where the aerobic zone starts.

4.1.2.5 Secondary Clarifiers

Existing conditions of Plants 4 and 5 secondary clarifiers were investigated to determine if the increased flow rate effected operation and removal efficiency. The parameters investigated were the average solids overflow rate (SOF), average overflow rate (v_0), peak solids overflow rate (Peak SOF), peak overflow rate (Peak v_0), and hydraulic retention time (HRT). *Equation 4-1* was used to calculate v_0 and Peak v_0 [15]. *Equation 4-2* was used to calculate SOF and Peak SOF [15]. To calculate HRT, *Equation 4-3* was used. Refer to *Table 4-2* for key values and *Appendix K: Parameters and Intermediate Values for Secondary Clarifier* contains all parameters and intermediate values for secondary clarifier analysis.

Values were determined to be overloaded, underloaded, or within limits using parameters given from *Water and Wastewater Engineering Design Principles and Practice 2nd Edition* [15]. Refer to *Appendix L: Acceptable Parameters for Secondary Clarifiers* for the figures depicting acceptable values.

Table 4-2: Secondary Clarifier Design Parameters

Limiting Parameters of Plant 4 and 5 Secondary Clarifiers		
v_0 (m/h)	1.39	Overloaded
SOF (kg/m^2*h)	4.18	Underloaded
Peak v_0 (m/h)	3.48	Overloaded
Peak SOF (kg/m^2*h)	10.44	Underloaded
HRT (hr)	3.94	Too High

Equation 4-1: Overflow Rate Equation [15]

$$v_0 = \frac{Q + Q_R}{A}$$

Where: V_0= Overflow rate (m/h), Q= Average design capacity (m^3/day), Q_R= Return activated sludge flow rate (m^3/day), and A= Surface area of clarifier (m^2).

Equation 4-2: Solids Overflow Rate Equation [15]

$$SOF = \frac{(Q + Q_R) * X}{A}$$

Where: X= MLSS concentration (kg/m^3) and SOF= Solid overflow rate (kg/m^2*h).

Equation 4-3: HRT Equation [15]

$$HRT = \frac{V}{Q + Q_R}$$

Where: HRT= Hydraulic retention time (hr) and V= Volume of clarifier (m^3).

It was determined that peak and average overflow rates were too high making the clarifiers overloaded. This meant removal efficiency of TSS in the clarifier would be negatively impacted. Disk filters are anticipated to reduce the effluent TSS to permit levels. However, with increased solids loading, the disk filters will need to be replaced more frequently. In addition, it was determined that the HRT in the clarifier is too high, meaning a slight decrease in TSS removal [17]. Adding to the notion that the disk filters will need to be monitored and maintained more consistently.

In conclusion, the current secondary clarifiers for Plants 4 and 5 will be overloaded with the increased flow rate. However, no changes are recommended, due to the high capital cost of building a new clarifier or modifying the clarifier. Instead, an intentional decision has been made to delegate more stress on disk filters, increasing the rate at which they will need to be maintained.

4.1.3 Solids Handling Design

The following subsections describe the solids handling design for the SPA 1 WRF.

4.1.3.1 Digester Design Assumptions

A set of assumptions were made to confirm the digesters had sufficient time to properly digest the influent sludge and produce biomethane. All assumptions relate to parameters of the digester and microbial kinetics: solids yield (Y), influent bCOD (S_0), decay coefficient (k_d), bsCOD removal, maximum specific growth rate (μ_m), half velocity constant (K_s), effluent soluble COD (S_e), and the safety factor (SF). All the assumptions were made under the condition that the digester would have an operating temperature of 35 °C and reach a methanogenic digestion process. Refer to *Appendix M: Anaerobic Digestion Assumptions* for supporting documentation of assumptions [15]. Refer to *Table 4-3* for all assumptions.

Table 4-3: Anaerobic Digestion Table of Assumptions

Anerobic Digestion Assumptions			
S_0 (g/m^3)	5000	u_m (g/g*d)	0.35
Y (g VSS/ g COD)	0.04	K_s (g/m^3)	160
bsCOD removal (%)	0.95	S_e (g/m^3)	500
k_d (g/g*d)	0.02	SF	5

4.1.3.2 Digester Design Calculations

Determining if the solids retention time (SRT) is adequate was the first step of calculations, refer to

Equation 4-4. Mass of biological solids synthesized (P_x) was calculated as an intermediate value to ultimately obtain the volume of biomethane (Q_{CH4}) and total biogas (Q_{BG}) produced per day. *Equation 4-5* was used to calculate P_x. *Equation 4-6* was used to calculate Q_{CH4}. Assuming that 65% of all biogas production is biomethane, total biogas production (Q_{BG}) can be determined, by dividing Q_{CH4} by 0.65 [15].

Equation 4-4: Solids Retention Tine Equation [15]

$$\frac{1}{SRT} = \frac{\left(\frac{\mu_m * S_e}{K_s + S_e} - k_d\right)}{SF}$$

Equation 4-5: Mass of Biological Solids Synthesized Equation [15]

$$P_x = \frac{Y * Q_w * (S_0 - S_e)}{1 + (k_d * SRT)}$$

Where: Q_w= Influent sludge flow rate (m^3/d).

Equation 4-6: Biomethane Volume Production [15]

$$Q_{CH4} = (0.35) * [(S_0 - S_e) * Q_w - (1.42 * P_x)]$$

Where: Q_{CH4}= Flow rate of biomethane produce (m^3/d).

Once SRT and Q_{BG} were found, the needed volume for the digesters was calculated using *Equation 4-7*; refer to *Table 4-4* for all calculated design values. A set volume is already in place from the existing aerobic digesters. Meaning iterations of influent sludge (Q_w) was adjusted to align with the available digester volume. Refer to *Table 4-5* for the dimensions and available volume of the existing infrastructure.

Equation 4-7: Volume Needed for Proper Anerobic Digestion [15]

$$V_{S\&G} = ((SRT * Q_w) + (Q_{BG} * 1day)) * SF_{DIG}$$

Where: $V_{S\&G}$= Volume needed for sludge and gas production (given gas is collection daily) (m^3) and SF_{DIG}= Safety factor of the digester volume.

Table 4-4: Design Calculations of Anerobic Digestion

Design Values			
Q_w (m^3/d)	225	Q_{BG} (m^3/d)	523
SRT with SF (days)	20.4	SF_{DIG}	1.50
P_x (kg/d)	28.8	$V_{S\&G}$ (m^3)	7667
Q_{CH4} (m^3/d)	340		

Table 4-5:Dimensions of Existing Digesters

Dimensions of Rectangular Digestor	
# of Digestors	2
Height (m)	7.77
Width (m)	17.98
Length (m)	27.43
Total Volume of Digesters (m^3)	7665

It should be noted that Q_w had already been adjusted to align with the available volume the existing digestors. The total amount of sludge being produced by the facility is 726 m^3/day, yet only 225 m^3/day (31%) of the sludge is being directed to the anerobic digestors.

4.1.3.3 Digestor Recommended Parameters

Using all calculations and assumptions from the previous two sections, a set of recommended parameters for anerobic digestion are provided in *Table 4-6*. A simple schematic of the proposed anaerobic digesters can be found in *Appendix N: Schematic of Anerobic Digesters*.

Table 4-6: Recommended Parameters of Anerobic Digestion

Recommended Parameters			
Q_w (m^3/d)	225	u_m (g/g*d)	0.35
pH	7	K_s (g/m^3)	160
SRT with SF (days)	20.40	S_e (g/m^3)	500
Temperature of Sludge (C)	35	# of Digestors	2
S_0 (g/m^3)	5000	Height (m)	7.77
Y (g VSS/ g COD)	0.04	Width (m)	17.98
bsCOD removal (%)	95%	Length (m)	27.43
k_d (g/g*d)	0.02	Total Volume of Digesters (m^3)	7665
Mixing Style	Unconfined Gas Diffusers	Mixing Rate (m^3/m^3*h)	0.28

4.1.3.4 Heat Exchanger Design

In order to maintain efficient digestion a constant temperature of 35 °C inside the digester is needed. A typical method of temperature regulation is the utilization of a heat exchanger. A heat exchanger works by boiling water using the biomethane produced, and the heat from the boiled water is transferred to sludge entering the digesters. Analysis was preformed to determine if enough biomethane is being produced during the winter months to fuel heat exchange.

First, the amount of heat energy required (q_r) for the sludge in digesters was calculated using *Equation 4-8*. Next, heat loss (q_L) of the digesters was calculated using *Equation 4-9*. A coefficient of heat transfer (U) was assumed for the roof, side walls, and floor. The assumptions were taken from *Water and Wastewater Engineering Design Principles and Practice 2nd Edition*, refer to *Appendix O: Heat Exchanger Design Assumptions* for supporting documentation [15]. The required total capacity from the heat exchanger (q_{cap}) is calculated by adding q_r with q_L. The lower heating value (LHV) of the fuel supplied to the exchanger was calculated by dividing q_{cap} by the heat exchanger efficiency. The heat exchanger efficiency was assumed from *Water and Wastewater Engineering Design Principles and Practice 2nd Edition* [15]. Lastly, LHV of the biomethane being produced was calculated by multiplying (Q_{CH4}) by the LHV of methane in general. The LVH of methane was assumed, refer to *Appendix O: Heat Exchanger Design Assumptions*. All key values are found in *Table 4-7*.

Equation 4-8: Heat Energy Required Equation [15]

$$q_r = M_{sl} * C_p * (T_2 - T_1)$$

Where: M_{sl}= Mass of sludge being digested (kg/d), C_p= Specific heat of water (kJ/kg*K), T_1= Temperature of sludge entering digester (K), and T_2= Temperature of sludge in digester (K)

Equation 4-9: Heat Loss Equation [15]

$$q_L = U * A * \Delta T$$

Where: A= Cross sectional area where heat is host (m^2) and ΔT= Temperature change across surface (K)

Table 4-7: Key Values of Heat Exchanger

Key Values from Heat Exchanger Design	
M_{sl} (kg/d) being digested	5894.34
Heat Exchanger Efficiency	80%
q_r (MJ/d)	616.84
q_L (MJ/d)	0.85
q_{cap} (MJ/d)	618.54
LHV of Fuel Needed (MJ/d)	773.17
LHV of Biomethane Produced (MJ/d)	7616.64

As seen in *Table 4-7*, LHV of fuel needed is smaller than the LHV of biomethane produced, meaning the heat exchanger can run solely off biomethane produced on site.

A scraped surface heat exchanger (SSHE) has been selected to be the heat exchanger of solids handling. SSHE's are well suited for viscous material like sludge [18]. A biomethane fueled boiler will be utilized in combination with the SSHE to heat the sludge. Specifications of the boiler and SSHE were not investigated. However, it is known that the SSHE will need to warm 340 m^3/day of sludge to 35°C and sufficient biomethane will be produced to fuel the boiler.

4.1.3.5 Mixing System

Mixing of the digesters is needed to maintain a homogeneous environment and optimize biogas production. The mixing system of choice will be unconfined bottom diffusors. The diffusors release gas from the bottom of the digestor, the gas bubbles through the sludge keeping it mixed. The mixing rate will be 0.28 m^3/m^3*hr per recommendations found in *Water and Wastewater Engineering Design Principles and Practice 2nd Edition* [15].

4.1.3.6 Gas Collection System Design

A gas collection system will be installed to collect the produced biogas. An inline flow meter will be installed to monitor digester operations to avoid water vapor from cooling and condensing on the gas collection system (Rivera, 2016). Gas storage tanks will hold the gas at a stable pressure and temperature with a medium pressure vessel below 100 psi. Safety will be maintained by pressure, temperature, and sealing monitors and regular inspections. Specifications of a gas collection system were not investigated; however, it is known that the system will need to handle an influent of 523 m^3/day of biogas.

4.1.3.7 Air Scrubber Design

The purpose of an air scrubber is to purify the biogas produced from anerobic digestion. Biogas is comprised of 55-65% methane (CH_4), 30-35% carbon dioxide (CO_2), and 0.1-5% hydrogen sulfide (H_2S). H_2S and the CO_2 is removed from the biogas to produce biomethane which can be used as a renewable energy source. A physical absorption scrubber is recommended for the system. Physical scrubbers rely on the solubility of H_2S and CO_2 since the biogas is passed through a water column where both compounds are dissolved [19]. Specifications of the air scrubber were not investigated; however, it is known the equipment will need to handle an influent of 523 m^3/day of biogas.

4.1.4 Other Treatment Systems

All remaining treatment systems including, grit chambers, disk filters, chlorine contact chambers, etc. are currently rated to handle a capacity of at least 16.3 MGD and no alternative was scored high enough to justify their replacement.

4.2 Equipment Sizing

The proposed band screens have the same manufacturer as the fine screens and can be interchanged into the facility's existing systems, given similar rated capacities [20]. Furthermore, band screens are easy to retrofit into existing channels [4]. The proposed band screens with 3-millimeter perforations are assumed to be a similar size to the existing fine screens which are approximately 5 feet 10 inches wide and 3 feet long.

The size of all troughs, oxidation ditches, clarifiers, disk filters, and contact basins are not changing as they currently exist. Pipe and pump sizing was analyzed in order see if they could convey the increased average and peak design flow of 16.3 MGD and 40.75 MGD. Pipes were also analyzed to see if flow velocity stayed below the maximum velocity of 10 cubic feet per second (cfs) [21]. Analyzed infrastructure include RAS pipes, RAS pumps, and pipes downstream from secondary treatment. RAS pumps and pipes adequately conveyed the average design flow, but neither the RAS pumps nor RAS pipes could adequately convey the peak design flow. It is acceptable that RAS infrastructure could not convey the peak flow because returned activated sludge can be reduced during peak events. The open channel troughs can contain the peak flow without overtopping and while maintaining a safe freeboard. The disk filters have a rated capacity that can convey the average design flow even with the secondary clarifiers being overloaded. The analysis that led to the previously mentioned conclusions are found in *Appendix P: Hydraulics Calculations*.

4.3 Site Layout

The footprint of the site will not be changed. Most systems will be left unchanged. There will be modifications to preliminary treatment, secondary treatment, and sludge digestion, but these will exist within the existing footprint. Additional changes include decommissioning of Plants 1 and 2 but the land they occupy is not needed and so there is no reason to demolish them. An existing site layout and proposed site layout with changed infrastructure marked can be found in *Appendix Q: Existing Site Layout* and *Appendix R: Proposed Site Layout*. Both layouts were created in ArcGIS Pro [22].

4.4 Process Flow Diagrams

The existing process flow diagram of the existing infrastructure is found in *Appendix C: Existing Process Flow Diagram*; this diagram was created using Civil 3D software [23]. The existing infrastructure uses gravity flow to direct each process to the next.

The proposed flow diagram shows the changes to preliminary treatment (switching fine screens to band screens) and flow redirection from the centrifuge to the anaerobic digestors for biomethane collection. It can be found in *Appendix S: Proposed Process Flow Diagram.*

4.5 Hydraulic Profile

A new hydraulic profile was developed using Civil 3D [23] to reflect the recommended design; it is shown in *Appendix T: Proposed Hydraulic Profile.* The proposed hydraulic profile shows the peak hour and average hour water surface elevation at various points from preliminary treatment to disinfection. Open channel head loss was calculated using Manning's Equation, and head loss in pipes was calculated using the Hazen-Williams Equation; refer to *Equation 4-10* and *Equation 4-11.*

Equation 4-10: Hazen-Williams Equation [24]

$$S = \left(\frac{Q}{A * k * C * {R_h}^{0.63}}\right)^{\frac{1}{0.54}}$$

Where: S= Slope (ft/ft), Q= Flow rate (ft^3/day), A= Area of pipe (ft^2), k= unit conversion factor, C= Hazen-Williams roughness coefficient, and R_h= Hydraulic Radius (ft).

Equation 4-11: Manning's Equation [24]

$$S = \left(\frac{Q * n}{1.49 * A * R^{\frac{2}{3}}}\right)^2$$

Where: S= Slope (ft/ft), Q= Flow rate (ft^3/day), A= Area of pipe (ft^2), n= Mannings roughness coefficient, and R_h= Hydraulic Radius (ft).

A manning's n-value of 0.013 was used for finished concrete [25], and a Hazen-Williams friction loss coefficient value, C, of 130 was used for the existing ductile iron pipe (DIP) [24].

The continuity equation, *Equation 4-12,* was used to verify that pipes throughout the treatment process could convey the average and peak flows at the maximum specified velocity of 10 feet per second [21].

Equation 4-12: Continuity Equation [24]

$$Q = V * A$$

Where: Q= Flow rate (ft^3/day), V= Velocity of Flow (ft/s), and A= Area of pipe (ft^2).

Head loss through the disk filters was obtained through consulting Aqua-Aerobic Systems, Inc [26]. The water surface elevations in the disinfection contact basins were found from information supplied through the AZWA SDC [3]. The water surface elevations within the troughs leading to the oxidation ditches were obtained utilizing Microsoft Excel's "goal-seek" function [27] to iteratively solve for the critical depth term in Manning's equation until the flow matched peak and average flow values. The remaining water surface elevations were found by adding or subtracting head loss values from known water elevation levels. *Table 4-8*, below, summarizes the average and peak design flow water surface elevations throughout the wastewater treatment process at SPA 1 WRF.

Table 4-8: Water Surface Elevations

Location	WSE (ft)	
	Average Flow	Peak Flow
Screening Influent Box	1144.17	1144.26
Screening Effluent Box	1143.97	1144.06
Trough to Oxidation Ditches	1141.86	1143.00
Parshall Flume	1140.61	1140.75
Selector Basins	1140.42	1140.56
Oxidation Ditches	1139.75	1139.83
Mixed Liquor Flow Splitter Box	1136.39	1136.84
Secondary Clarifier	1136.25	1136.30
Entering Disk Filters	1136.12	1136.17
Exiting Disk Filters	1134.62	1134.67
Disinfection Basin	1119.50	1120.50

The technical work required for creating the proposed hydraulic profile and analyzing existing infrastructure is shown in *Appendix P: Hydraulics Calculations*.

5 Cost Analysis

Details regarding the capital and O&M costs for the recommended design are included below.

5.1 Engineers' Opinion of Probable Construction Cost (EOPCC)

The Engineers' Opinion of Probable Construction Cost (EOPCC) for the proposed improvements to the SPA 1 WRF is found in *Appendix U: Engineers Opinion of Probable Construction* Cost. Capital costs for each respective treatment process were estimated through analysis of RSMeans [7], similar engineering projects, and consultation from experienced engineers [28] [29]. The total construction cost is estimated to be $1,455,000.

5.2 Estimate of Annual Operation and Maintenance Costs (O&M)

An estimate of annual operation and maintenance costs (O&M) for existing conditions of the SPA 1 WRF is found in *Appendix V: Estimate of Annual Operation and Maintenance Costs for Existing Conditions*. Supporting calculations for the total O&M costs can be found in *Appendix W: Cost Analysis*. Items included in O&M costs include energy consumption costs, routine equipment replacement, and labor costs to operate the WRF. Labor costs were calculated based on three typical WRF operator hourly wage values [11]. The facility is assumed to use three low and middle tier operators each and two senior rank operators for a total of eight operators. RSMeans [7] and similar engineering projects were used to estimate the costs associated with routine equipment replacement. Energy was assumed to cost $0.15 per kilowatt hour [30]. The energy consumption was estimated for secondary and preliminary treatment processes and pumps with the amount of energy consumed per day for one year. For secondary, the horsepower used for the individual rotor and mixer of each plant per day in kWh is assumed. Pump energy costs were also estimated using their horsepower assuming that they operate at 75% capacity on average over a year. The routine replacement of disk filters and brush aerators were considered in O&M costs; the typical design life of a brush aerator was obtained at a site tour of the facility [11]. The O&M costs and design life of equipment associated with advanced treatment were found from a similar engineering project in Riverside, California [31]. The annual amount of money saved from brush aerators not being damaged as frequently and not needing replacement as often were considered as offsetting annual costs. Disinfection treatment costs were estimated to be 60% cheaper than conventional disinfection [32]. The O&M costs related to solids handling were made on assumptions taken from a case study of anerobic digestion on a dairy farm [33]. Considering annual savings from biomethane production and annual O&M costs, a break-even analysis was performed for solids handling. A full breakdown of the break-even analysis can be found in *Section 3.6.2* with the table of the analysis being found in *Appendix H: Life Cycle Cost Analysis of Anerobic Digesters*.

The existing O&M costs for SPA 1 are $4,219,685. The total yearly O&M costs for the proposed facility after the proposed changes are estimated to be $3,102,685, refer to *Appendix X: Estimate of Proposed Annual Operations and Maintenance Costs*. For yearly savings of $1,117,000, largely due to decreased brush aerator maintenance and replacements.

6 Construction Sequencing

Preliminary construction sequencing will be performed sequentially. One screen will be shut down at a time, ideally during non-peak flow hours of 5 am to 10 am. If necessary, the redundancy of the overflow channels will be utilized to redirect flow around screening, and operators will monitor for any negative impacts on the downstream systems during construction. This process will be repeated for the second and third screens.

Secondary design will require minimal construction phasing. To adjust the ditches, their digital controls will need to be reprogrammed. During non-peak hours it is recommended to batch the plants with the new operation style one at a time for an estimated 30 minutes (or until the microbiome adjusts and produces effluent within permit limits). Each ditch (half plant) will act as an independent oxidation ditch. They will have an anaerobic zone first, followed by an aerobic length, before the flow returns to the anaerobic section. This will be controlled by the brush aerators in each ditch being submerged for the first pass and at the surface for the second. These aerators have an adjustable submersion depth and can be changed via digital controls. The RAS will be set at 75% of the design flow. Upgrades to preliminary treatment should be completed before the secondary treatment systems are changed to the new operation style to prevent rags from damaging an oxidation ditch while a different ditch is batching.

The basins and piping involved in the anaerobic digestors will require refurbishment as part of the construction scope. However, these refurbishments will not impact ongoing sludge processing during construction, as the existing centrifuges and solar drying facilities will not be changed or affected. Construction of the gas collection lines will be placed above the anaerobic digestors and direct to a gas storage tank. An inline direct mass flow meter will monitor collection to prevent leakage or explosions of pressurized gas. The pipe will contain a membrane separator to transfer the waste gas to a waste piping system and the biomethane to a storage tank. The waste gas will be collected with the current scrubbed gas in preliminary for odor control and be disposed of with the sulfur gas.

Pedestrian and vehicular phasing plans are not anticipated to be needed due to the proposed improvements having a minor effect on the operations and movement throughout the rest of the facility. Any large equipment delivered to the facility should not be placed in an area that interferes with heavy vehicular traffic or areas that disturb frequent travel such as solid waste leaving the facility; consideration should be given to proper laydown areas if required.

Further construction sequencing details can be found in the Manual of Permitted Operations (MOPO) found in *Appendix Y: Manual of Permitted Operations*. This includes matrices that show what construction and maintenance activities can and cannot be completed at the same time, as well as which can and cannot be completed during inclement weather and high influent flow.

7 Project Impacts

This project will have direct external impacts on social, environmental, and economic aspects. The increased capacity for SPA 1 WRF will allow the surrounding area to facilitate the population growth expected in the coming years near Surprise, Arizona. Furthermore, the proposed upgrades will create more work for the construction industry.

From an environmental perspective, the recommended design solution does not change the footprint of the existing facility; this means that no additional energy and resources will need to be spent on constructing infrastructure such as concrete for oxidation ditches or clarifiers. The design team approached the problem with sustainability in mind so that existing infrastructure could be retrofitted and repurposed to accommodate the additional treatment capacity and solve existing challenges that arose during the project. More material resources will be used to produce disk filters, however, as they will need to be replaced more frequently. Converting the aerobic digestors to anaerobic digestors will reduce the volume of solids taken to the landfill; this reduces the carbon emissions and energy required to transport and handle the solids at both the SPA 1 WRF and landfill. Additionally, the biomethane produced will serve as an alternative to natural gas for a local business once sold, providing them with a climate friendlier energy source.

Expanding the capacity of the facility allows the City to expand which opens up more potential for economic opportunities in the surrounding area. Decreasing the yearly O&M costs for the SPA 1 facility positively impacts both the facility and surrounding community. The facility is publicly owned and operated, so a change in cost directly affects the taxpayers and end users who help fund the facility. The recommended design changes keep required capital and O&M costs very low which means taxpayers are not affected or very minimally affected by the improvements at SPA 1 WRF.

8 Summary of Engineering Work

The work provided created plans to implement the change in capacity of the SPA 1 WRF from 12.8 to 16.3 MGD.

8.1 Summary of Work Completed

The work completed can be found in *Section 4: Recommended Design* where analysis and sizing of existing equipment was first performed to understand the plant. Following this process, design alternatives were generated and compared in decision matrices. For each highest scoring design alternative, a final design was developed and the capital cost and operations and maintenance costs were calculated.

8.2 Revised Schedule

Using Microsoft Project, a proposed Gannt chart, found in *Appendix Z: Proposed Gantt Chart*, was compared to a revised chart, found in *Appendix AA: Actual Gantt Chart*. Major changes to the work included the site visit being pushed back causing alternative selection to be delayed past the 30% deliverable. This required work pace to increase between the 30% and 60% deliverable deadlines. Allocation of different treatment final designs being completed individually and collaboratively depending on difficulty of task. Additionally, minimal design work was completed for primary, advanced, and disinfection as these processes were not changed, allowing for additional time to make up for work not completed at the 30%.

9 Summary of Engineering Costs

Engineering work was completed by four core team members, a senior engineer (SENG), a design engineer (DENG), a civil engineer intern (CINT), and an environmental engineer intern (EINT).

Predicted work hours needed to complete the project came to 784 hours. The majority of the work comes from the design engineer, and the most time-consuming task was Task 4: Final Design. Refer to *Table 9-1* for the proposed summary of work table.

Table 9-1: Proposed Summary of Work Table

Task	SENG	DENG	CINT	EINT	Total Work Hours
Task 1: Research Preparation	1	0	12	12	25
Task 2: Site Assessment	8	8	12	12	40
Task 3: Treatment Process Selection	25	40	70	70	205
Task 4: Final Design	13	174	46	44	277
Task 5: Project Impacts Analysis	4	16	2	2	24
Task 6: Project Deliverables	17	14	46	46	123
Task 7: Project Management	30	30	15	15	90
TOTAL HOURS	98	282	203	201	784

The actual work provided by the team totaled 533.5 hours. The majority of the work came from the design engineer, civil engineer intern, and environmental engineer intern. The most time-consuming tasks being Task 3: Treatment Process Selection, Task 4: Final Design, and Task 6: Project deliverables. Refer to *Table 9-2* for the actual summary of work table.

Table 9-2: Actual Summary of Work Table

Task	SENG	DENG	CINT	EINT	Total Work Hours
Task 1: Research Preparation	**1**	**1**	**0**	**4.5**	**6.5**
Task 2: Site Assessment	**8.5**	**24.5**	**17**	**19**	**69**
Task 3: Treatment Process Selection	**3**	**20**	**36**	**48**	**107**
Task 4: Final Design	**14**	**45**	**18**	**27**	**104**
Task 5: Project Impacts Analysis	**1**	**2**	**1**	**0**	**4**
Task 6: Project Deliverables	**3**	**21**	**71**	**50**	**145**
Task 7: Project Management	**25**	**30**	**18**	**25**	**98**
TOTAL	**55.5**	**143.5**	**161**	**173.5**	**533.5**

In the end, the difference between the proposed and actual summary of engineering work was 250 hours. This was largely because Task 3: Treatment Process Selection and Task 4: Final Design did not take as long as expected. This time decrease was due to the fact a design was not needed for the primary, advanced, and disinfection treatment processes. Time spent on project deliverables did increase by about 15 hours due to the extra time required to discuss and practice the final presentation for the AZWA student design competition.

The cost of engineering services is broken out in three categories of personnel, supplies, and travel. The estimated cost of engineering services came out to $89,213, refer to *Table 9-3*.

Table 9-3: Proposed Cost of Engineering Services

Category	Sub-Category	Classification	Quantity	Unit	Rate	Unit	Cost ($)
1.0 Personnel		SENG	98	hours	250	$/hour	$24,500
		DENG	282	hours	150	$/hour	$42,300
		CINT	203	hours	50	$/hour	$10,150
		EINT	201	hours	50	$/hour	$10,050
						Subtotal:	**$87,000**
2.0 Supplies		Membership	4	memberships	20	$/subscription	$80
		Computer Lab Rental	10	days	100	$/day	$1,000
						Subtotal:	**$1,080**
3.0 Travel	**3.1 Site Visit**	Car	1	day	38.93	$/day	$39
		Gas	286	miles	0.455	$/mile	$127
	3.2 Competition	Car	2	day	38.93	$/day	$78
		Gas	286	miles	0.455	$/mile	$127
		Per Diem	8	day-person	36.75	$/day-person	$294
		Hotel	3	night-room	156	$/night-hotel	$468
						Subtotal:	**$1,133**
						Total Cost of Engineering Services:	**$89,213**

The actual cost of engineering services only changed because of a decrease in personnel hours. All other expenses were correct, bringing the actual cost of engineering services to a total of $54,338.

Table 9-4: Actual Cost of Engineering Services

Category	Sub-Category	Classification	Quantity	Unit	Rate	Unit	Cost ($)
1.0 Personnel		SENG	55.5	hours	250	$/hour	$13,875
		DENG	143.5	hours	150	$/hour	$21,525
		CINT	161	hours	50	$/hour	$8,050
		EINT	173.5	hours	50	$/hour	$8,675
						Subtotal:	**$52,125**
2.0 Supplies		Membership	4	memberships	20	$/subscription	$80
		Computer Lab Rental	10	days	100	$/day	$1,000
						Subtotal:	**$1,080**
3.0 Travel	**3.1 Site Visit**	Car	1	day	38.93	$/day	$39
		Gas	286	miles	0.455	$/mile	$127
	3.2 Competition	Car	2	day	38.93	$/day	$78
		Gas	286	miles	0.455	$/mile	$127
		Per Diem	8	day-person	36.75	$/day-person	$294
		Hotel	3	night-room	156	$/night-hotel	$468
						Subtotal:	**$1,133**
						Total Cost of Engineering Services:	**$54,338**

The cost of engineering services was overestimated by about $35,000 due to the large decrease in personnel hours.

10 Conclusions

The City of Surprise requires an increase in capacity from 12.8 to 16.3 MGD as the city is rising in population. Per the client's request, alternatives with no change to the footprint were considered alongside the implementation of a sixth plant. Alternatives were weighed and considered using decision matrices to best meet the needs of the City of Surprise. The best alternatives were selected and a full design was made.

Through the final design recommendations, found in *Section 4:Recommended Design*, the final capacity of the SPA 1 WRF will be increased from 12.8 to 16.3 MGD. Additional changes will meet objectives not originally present in the problem statement that were identified during the course of the project. Through the replacement of fine screens to band screens, annual O&M will drastically decrease in the secondary treatment process by preventing rags from bypassing screens and damaging brush aerators. Additional savings will occur from the recommended sludge design through the selling of generated biomethane from the anaerobic digestors.

The SPA 1 WRF is estimated to hit the maximum capacity of 16.3 MGD within the next 10-15 years based on their water master plan [34]. This design allows for the expansion of a sixth and seventh plant onsite by not changing the current footprint of the facility. The recommended design is a cost-effective alternative to allow for ongoing treatment until additional capacity is needed within the next 15 years. The total cost to implement the project will be $1,509,338 for capital and cost of engineering services. This will take 1-2 years to implement all changes, mainly due to the refurbishment of the aerobic digestor to anaerobic digestors. The construction sequencing plan and manual of permitted operations will help ensure that the facility remains operational throughout construction.

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12 Appendices

Appendix A: Site Visit Photo Log

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Photo 1: Headworks Facility

Photo 2: 3mm Finescreen

Photo 3: Open channel flow after headworks

Photo 4: Plant 3 Oxidation Ditch

Photo 5: Plant 3 Secondary Clarifier

Photo 6: Flow entering Plant 5 over a weir

Photo 7: Disk Filtration used in Advanced Treatment

Photo 8: Microchlor onsite hypochlorite generator

Photo 9: Contact Basin

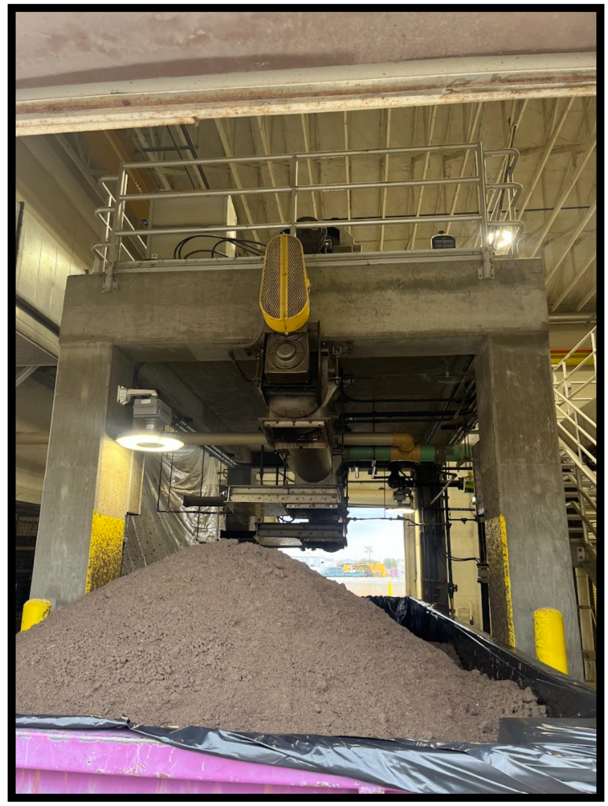

Photo 10: Solids Production

Photo 11: Drying Beds

Photo 12: Recharge Basin

Appendix B: Existing Hydraulic Profile

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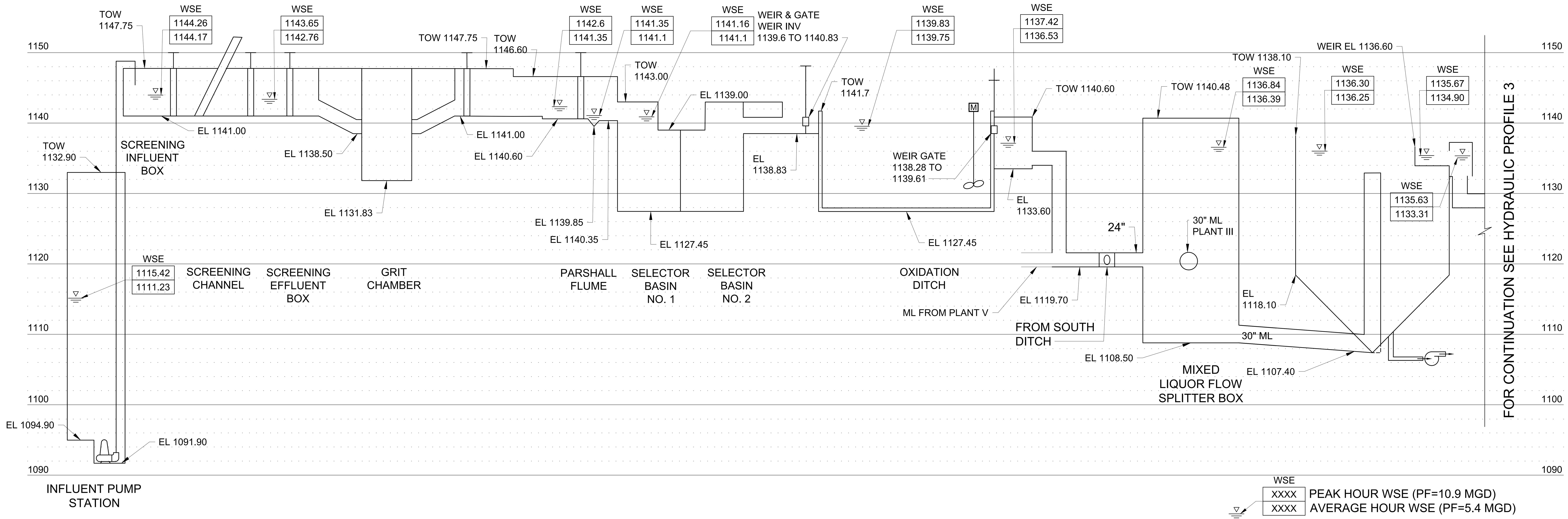

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90% SUBMITTAL
NOT FOR CONSTRUCTION

DESIGN DB
DRAWN MTP/JNE
CHECKED JHM
APPROVED JHM

WATERWORKS ENGINEERS

SURPRISE ARIZONA

CITY OF SURPRISE
SPA 1 WRF CAPACITY EXPANSION

GENERAL

HYDRAULIC PROFILE 2
PLANTS IV AND V PRELIMINARY
AND SECONDARY TREATMENT

DATE APRIL 2023
PROJECT NUMBER 20-018
DRAWING NUMBER G-00-007
SHEET
SHEET NO. ###

FILENAME: C:\USERS\ANGELF\ACCDOCS\WATER WORKS ENGINEERS\20-018 SPA 1 WRF CAPACITY EXP FACILITY RERATING\PROJECT FILES\07 DRAWINGS\2018-G-00-007.DWG

Appendix C: Existing Process Flow Diagram

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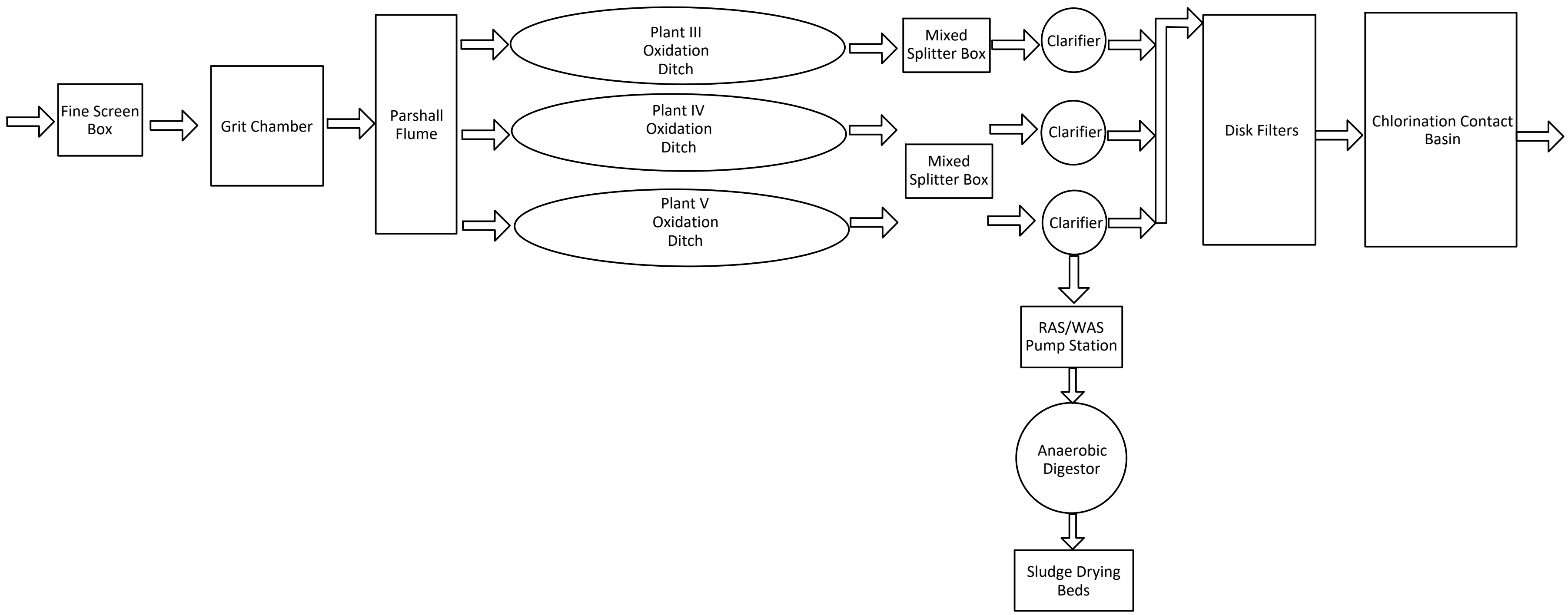

Company: WALNUT CANYON WASTEWATER
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REVISIONS

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EXISTING FLOW DIAGRAM

Appendix D: Vendor Information

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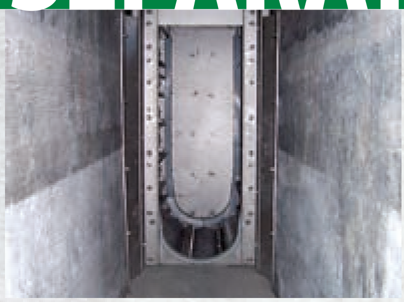
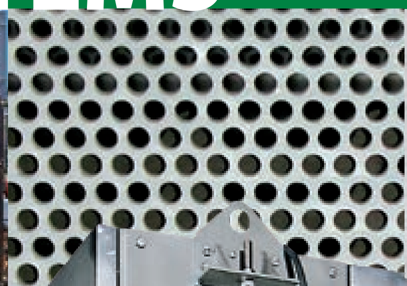
JWC Environmental®
A Sulzer Brand

MONSTER SEPARATION SYSTEMS®





Bandscreen Monster®

This system offers incredibly high capture rates and is able to remove a wider variety of solids, particularly small debris, better than traditional screens. It can also be used to protect high-tech Membrane Bioreactors.

The rotating panels are positioned parallel to the flow and as wastewater enters the screen it flows through the perforated screening panels. Easy to retrofit into existing channels and installs at a 90º inclination.

Unique Flow Design

- Zero carryover and the highest capture rate of all screens*.
- Perforated openings capture twice as much debris as bar screens.
- Perforated UHMW inserts limit hair pinning (replaceable) with stainless steel frame.

Enhanced Cleaning System

- Spray bar keeps the screen's panels clear.

Heavy-Duty Stainless Steel Roller Chains

- Stainless steel construction ensures long life.
- Roller chains track smoothly in UHMW guides.

EXCELLENT PROTECTION FOR MBRs

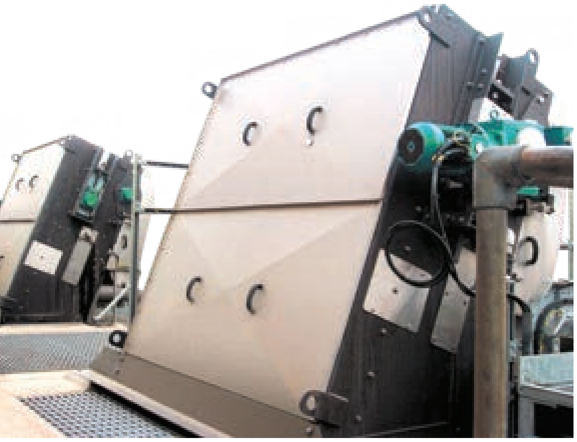
Equipment Sizing

Screen panel hole size: ø 5/64", 1/8" or 1/4" (2, 3 or 6mm) Perforations
Minimum Wash Water Head at Spray Jets: 55 PSI (3.8 bar)

Materials of Construction

Screen Structure: 304 or 316 Stainless Steel
Screen Panels: UHMW Plates, 1/4" or 3/8" Thick

Monster Separation Systems®



Finescreen Monster®

The Finescreen Monster incorporates a continuous band of stainless steel panels or optional StapleGuard ultra high molecular weight (UHMW) polyethylene perforated panels attached to heavy-duty stainless steel roller chains. Panels available with 1/8″ or 1/4″ (3 or 6 mm) openings. Stainless steel rollers track in UHMW guides at the bottom of the screen, thus eliminating the need for sprockets or bearings submerged in the wastewater flow.

Advanced Design

- Completely stainless steel.
- UHMW side seals and bottom sealing strip prevent debris from passing around the screen.

Enhanced Cleaning System

- Brushless cleaning system using water spray.
- Lower Maintenance and better panel cleaning.

Ease of Maintenance

- Easy to lift access covers and easy to reach assembly allows simple fine tuning.

Staple Guard UHMW Polyethylene Perforated Panels (optional)

- Reduces stapling (or hair pinning) on the panels.
- Highly abrasion, wear and corrosion resistant.

Equipment Sizing

Screen Panel Hole Size: ø 1/8″ or 1/4″ (3 or 6mm) Holes
Depth: up to 20′ (6m) with a max 5′ (1.5m) Discharge Height
Width: 2′ to 8′ (.6 to 2.4m)
Angle: 60° to 85° Inclination; 70° Standard

Since its founding in 1973, JWC Environmental has become a world leader in solids reduction and removal for the wastewater industry with its Muffin Monster grinders and Monster Separation Systems. JWC also solves challenging size reduction and processing problems in commercial and industrial applications through its Monster Industrial division. JWC Environmental is headquartered in Santa Ana, California, and has a global network of representatives, distributors and regional service centers to provide customer support.

For more information, visit JWC Environmental at www.jwce.com.

JWC Environmental®
A Sulzer Brand

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fax: **949.833.8858**
email: **jwce@jwce.com**

www.jwce.com

(MSSMonster-NA-JWCE-0122)

CHANNEL MONSTER®
FLEX

Overview

The patent pending Channel Monster FLEX consists of a FLEX grinder and a solids diverter with perforated screen connected by a FLEX frame. This modular design allows for the flexibility of servicing the FLEX grinder and solids diverter separately while maintaining the best-in-class technology for wastewater solids reduction. An exacta-lock adjuster mechanism allows for fine-distance adjustment between the grinder and screen to minimize solids bypass. The Channel Monster FLEX continues the Channel Monster legacy of high flow capacity while capturing and shredding rags, rocks, wood, and other solids into small pieces to pass harmlessly through pumps, pipes, and processing equipment.

Benefits

Equipment protection

- Protect pumps and other critical equipment from costly clogs and damage from tough solids

Efficient treatment operations

- Grinding separates organic from inorganic materials in the waste stream keeping organics in the treatment process, and removed screenings are cleaner

Lower operating cost

- Grinding solids into smaller pieces keeps pipes and pumps clear, resulting in shorter pump run cycles and lower electrical costs
- Reduced unit maintenance expense with modular FLEX grinder and solids diverter

Features

1 FLEX grinder

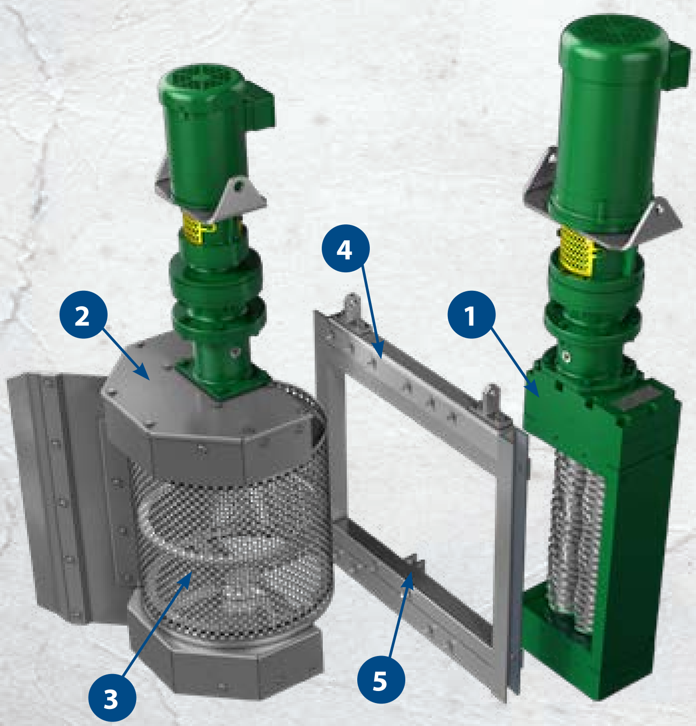
- Dual-shafted, slow-speed, high-torque to grind a wide variety of solids
- Modular for easy field replacement
- Optional 10 hp motor for the highest cutting force for grinder in its size class

2 Solids diverter with perforated screen

- Allows higher flow while capturing solids and directing them into the grinder
- 3 304 stainless steel 1/2-inch (12.7 mm) perforated drum
- Modular assembly for easy field replacement

4 FLEX frame

- Connects FLEX grinder and solids diverter
- 5 Exacta-lock adjuster precisely locks distance between grinder and drum to optimize solids capture and shredding of solids



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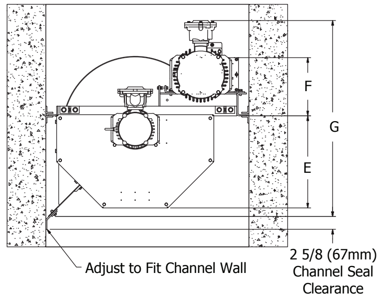
Channel Monster® FLEX



Materials of construction

Solids diverter perforated screen: 304 stainless steel
Solids diverter cover: 304 stainless steel
Solids diverter end housings: Gray iron
FLEX grinder cutter and spacers: Hardened alloy steel standard, other metals optional
FLEX grinder shafts: Hardened alloy steel
FLEX grinder end housings, covers and side rails: Gray iron
Mechanical seal faces: Tungsten carbide
FLEX and channel frames: 304 stainless steel

Cutter Stack Height
18, 24, 36 or 60 inches

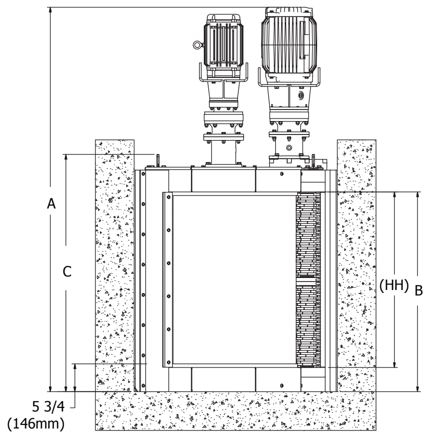
Drive Configuration
M - Multi-drive

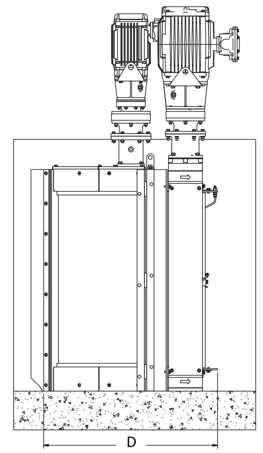
Duty Rating
2.0 - Standard duty

CMF HHDD-M2.0T

Drum Diameter
12, 18, 24, 30, 36 or 60 inches

Drive Type
E - Electric motor
H - Hydraulic drive
HE - Hydraulic grinder / Electric drum drive



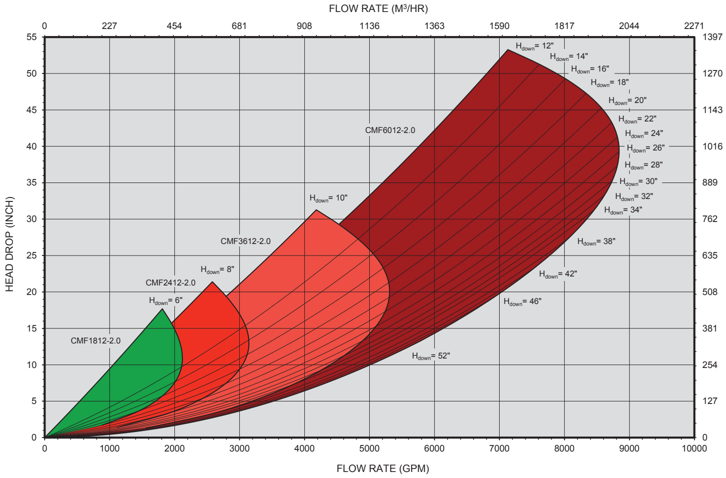


Model	A[1] - inches (mm)	B - inches (mm)	C - inches (mm)	D - inches (mm)	E - inches (mm)	F - inches (mm)	G - inches (mm)	Min/Max Channel Width - inches (mm)	Max Flow – mgd (m3/hr)	Approximate Net Weight - lbs (kg)[1]
CMF1812-M2.0E[2]	90-7/16 (2297)	23-3/4 (603)	31-1/2 (800)	23 (586)	10-1/4 (260)	8-3/4 (222)	27-5/8 (700)	22 / 36 (559) / (914)	3.0 (479)	1085 (492)
CMF2412-M2.0E[2]	96-3/16 (2443)	29-1/2 (749)	37-1/8 (943)	23 (586)	10-1/4 (260)	8-3/4 (222)	27-5/8 (700)	22 / 36 (559) / (914)	4.6 (720)	1146 (520)
CMF3612-M2.0E[2]	108-1/16 (2745)	41-1/4 (1048)	49 (1245)	23 (586)	10-1/4 (260)	8-3/4 (222)	27-5/8 (700)	22 / 36 (559) / (914)	7.6 (1206)	1311 (595)
CMF6012-M2.0E[2]	132-1/16 (3354)	65-1/4 (1657)	73 (1854)	23 (586)	10-1/4 (260)	8-3/4 (222)	27-5/8 (700)	22 / 36 (559) / (914)	12.7 (2008)	1813 (822)
CMF1818-M2.0E	61-13/16 (1570)	23-3/4 (603)	31-1/2 (800)	27-3/4 (706)	13 (330)	9-1/4 (235)	32-3/8 (821)	30 / 44 (762) / (1118)	4.6 (723)	1048 (475)
CMF2418-M2.0E	67-1/2 (1715)	29-1/2 (749)	37-1/8 (943)	27-3/4 (706)	13 (330)	9-1/4 (235)	32-3/8 (821)	30 / 44 (762) / (1118)	6.6 (1036)	1116 (506)
CMF3618-M2.0E	79-3/8 (2016)	41-1/4 (1048)	49 (1245)	27-3/4 (706)	13 (330)	9-1/4 (235)	32-3/8 (821)	30 / 44 (762) / (1118)	10.4 (1647)	1284 (582)
CMF6018-M2.0E	103-3/8 (2626)	65-1/4 (1657)	73 (1854)	27-3/4 (706)	13 (330)	9-1/4 (235)	32-3/8 (821)	30 / 44 (762) / (1118)	16.5 (2596)	1587 (720)
CMF1824-M2.0E	61-13/16 (1570)	23-3/4 (603)	31-1/2 (800)	31-5/8 (805)	16 (406)	10-7/8 (276)	36-1/4 (919)	36 / 50 (914) / (1270)	6.0 (945)	1088 (493)
CMF2424-M2.0E	67-1/2 (1715)	29-1/2 (749)	37-1/8 (943)	31-5/8 (805)	16 (406)	10-7/8 (276)	36-1/4 (919)	36 / 50 (914) / (1270)	8.5 (1334)	1160 (526)
CMF3624-M2.0E	79-3/8 (2016)	41-1/4 (1048)	49 (1245)	31-5/8 (805)	16 (406)	10-7/8 (276)	36-1/4 (919)	36 / 50 (914) / (1270)	13.1 (2059)	1344 (609)
CMF6024-M2.0E	103-3/8 (2626)	65-1/4 (1657)	73 (1854)	31-5/8 (805)	16 (406)	10-7/8 (276)	36-1/4 (919)	36 / 50 (914) / (1270)	20.0 (3160)	1674 (759)
CMF2430-M2.0E	67-1/2 (1715)	29-1/2 (749)	37-1/8 (943)	35-7/8 (912)	19 (482)	12 (305)	40-3/8 (1026)	42 / 56 (1067) / (1422)	10.5 (1658)	1287 (584)
CMF3630-M2.0E	79-3/8 (2016)	41-1/4 (1048)	49 (1245)	35-7/8 (912)	19 (482)	12 (305)	40-3/8 (1026)	42 / 56 (1067) / (1422)	16.9 (2660)	1486 (674)
CMF6030-M2.0E	103-3/8 (2626)	65-1/4 (1657)	73 (1854)	35-7/8 (912)	19 (482)	12 (305)	40-3/8 (1026)	42 / 56 (1067) / (1422)	26.6 (4191)	1838 (834)
CMF3636-M2.0E	79-3/8 (2016)	41-1/4 (1048)	49 (1245)	40 (1015)	22 (559)	13 (330)	44-1/2 (1129)	48 / 62 (1219) / (1575)	23.5 (3709)	1601 (726)
CMF6036-M2.0E	103-3/8 (2626)	65-1/4 (1657)	73 (1854)	40 (1015)	22 (559)	13 (330)	44-1/2 (1129)	48 / 62 (1219) / (1575)	40.4 (6378)	1977 (897)

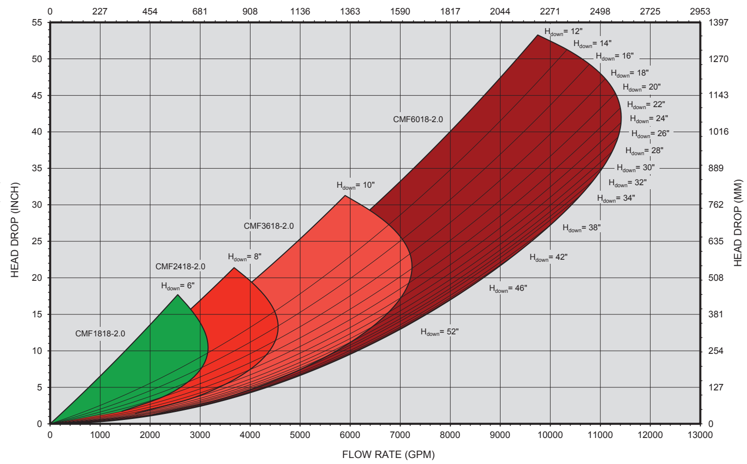
1. Estimated height and weight based on 1 hp XPNV solids diverter motor and 5 hp XPNV FLEX grinder motor.
2. Dimension "A" height is for solids diverter motor height with extended shaft. If extended shaft is required for FLEX grinder drive, solids diverter extended shaft must always exceed by a minimum of 30 inches (752mm).

Channel Monster® FLEX

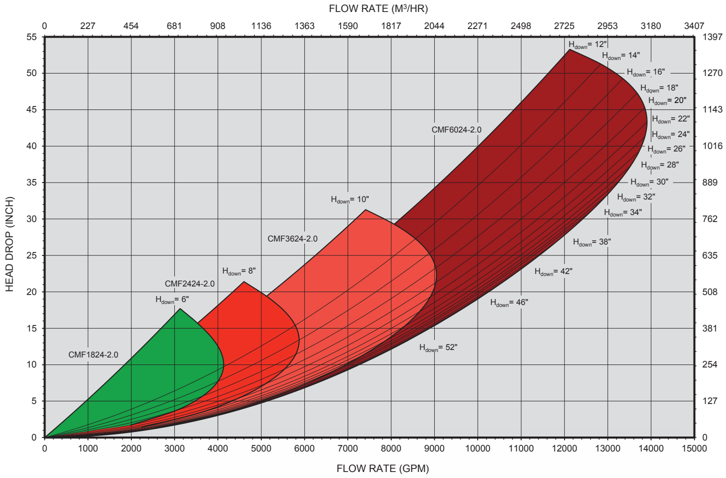
HEAD DROP
MODEL CMFHH12-2.0 Ø1/2" PERFORATIONS



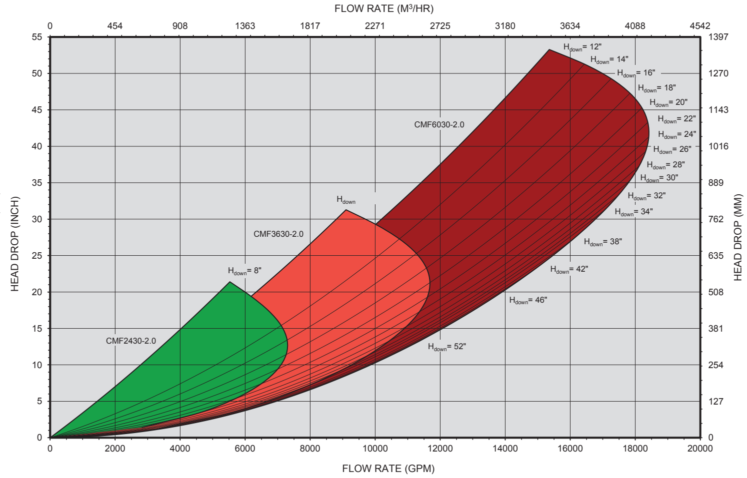
HEAD DROP
MODEL CMFHH18-2.0 Ø1/2" PERFORATIONS



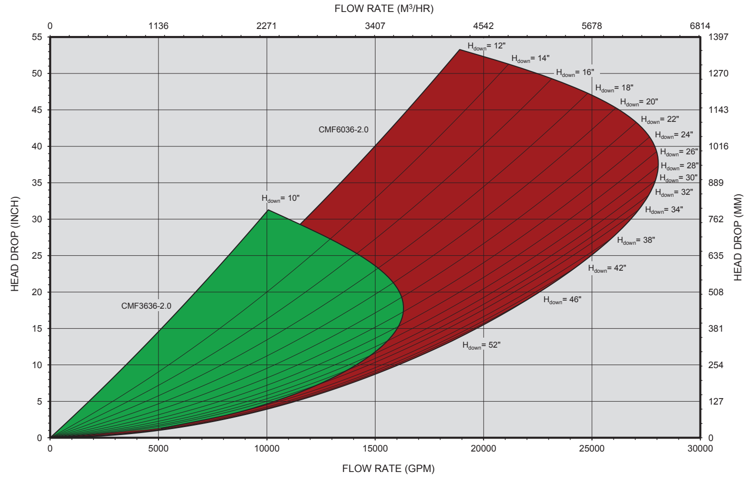
HEAD DROP
MODEL CMFHH24-2.0 Ø1/2" PERFORATIONS



HEAD DROP
MODEL CMFHH30-2.0 Ø1/2" PERFORATIONS



HEAD DROP
MODEL CMFHH36-2.0 Ø1/2" PERFORATIONS



7-tooth:
Heavy Solids

11-tooth:
Typical Solids Loading

17-tooth:*
Rags & Stringy Materials

Extended Motor Shaft

Extended motor shaft

- Places motor above highest water level
- Available in 1-foot (305 mm) increments
- Maximum: 15 feet (4570 mm)

Model PC2200 Standard Enclosure

FLEX grinder cutters

- 7- and 11-tooth cutters in alloy steel or stainless steel
- 17-tooth serrated Wipes Ready® cutters in alloy steel

Custom wall and channel frames

- Custom-built to meet site requirements: may include guide rails, grinder support base, overflow bar racks and more
- Guide rails for easy installation and maintenance of unit
- Stainless steel construction

Smart controller

- Load-sensing control system automatically reverses to clear jams
- Standard: NEMA 4X FRP enclosure with 3-position switch and status indicators
- Optional: NEMA 4X stainless steel or NEMA 7 enclosures
- Customized control configurations for any installation
- UL registered

Electric Motor

Exclusive: JWC-Designed
Immersible Motor
(NEMA 6P)

Electric motors

FLEX grinder 5 hp standard, 10 hp optional
solids diverter 1 hp

- TEFC: Totally enclosed fan-cooled
- XPFC: Explosion-proof fan-cooled
- XPNV: Exclusive immersible

Hydraulic Drives for 10 hp
Power Pack

Hydraulic Drives with 15 hp
Power Pack

Hydraulic drive motor assembly

- 10 hp hydraulic power pack: 3 hp equivalent on FLEX grinder, 1 hp equivalent on solids diverter
- 15 hp hydraulic power pack: 5 hp equivalent on FLEX grinder, 1 hp equivalent on solids diverter

JWC Environmental® SERVICE SOLUTIONS

JWC Service Solutions

Monster Renew: Replace the worn Channel Monster FLEX grinder or solids diverter with a factory-new module. Renew module minimizes downtime and comes with a 1-year factory warranty.

JWC Environmental®
A Sulzer Brand

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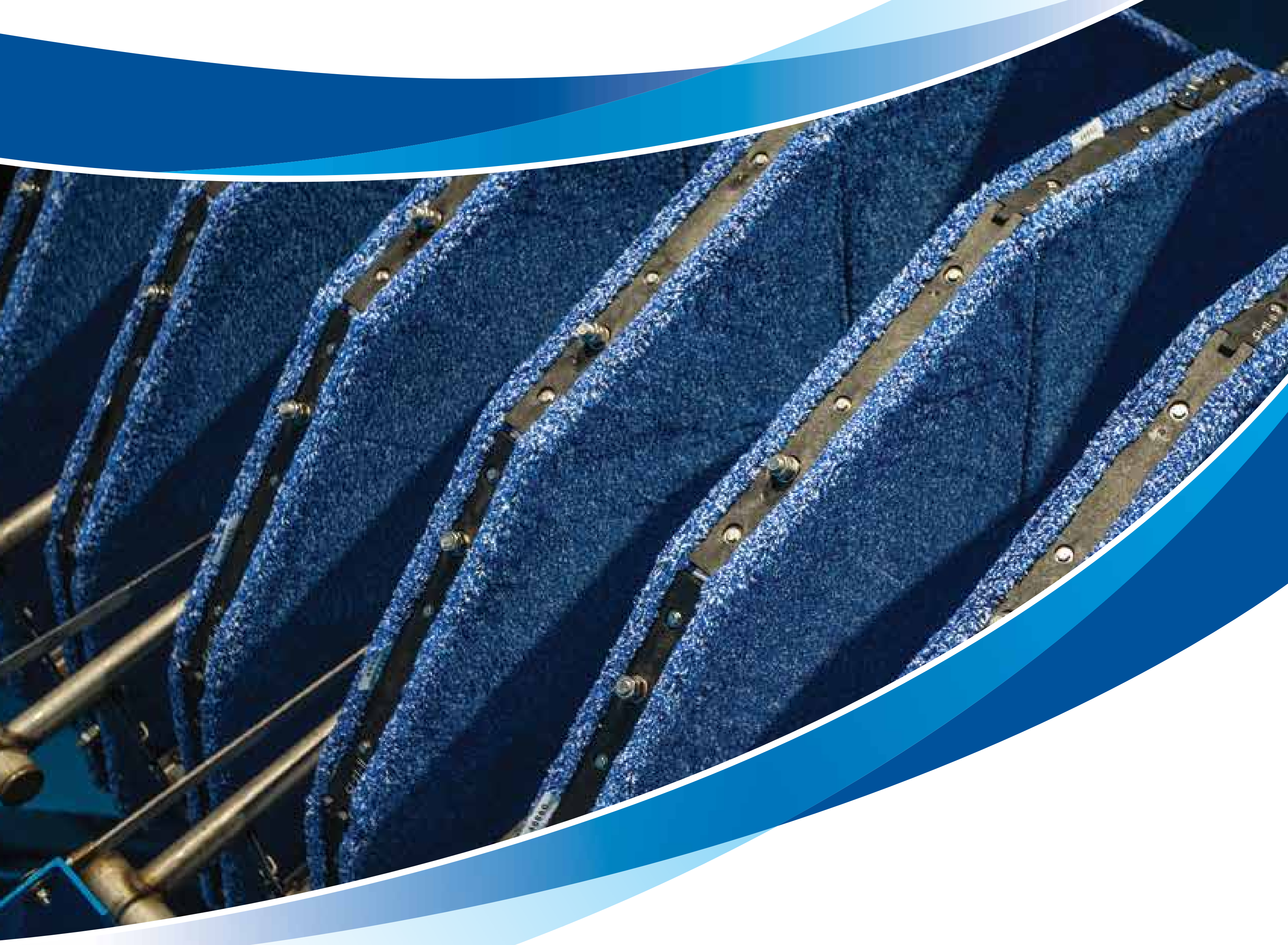
Santa Ana, CA 92705 USA
toll free: **800.331.2277**
phone: **949.833.3888**
fax: **949.833.8858**
email: **jwce@jwce.com**

Muffin Monster® MONSTER INDUSTRIAL® IPEC®

www.jwce.com

Cloth Media Filtration

Featuring OptiFiber® Pile Cloth Media



AQUA-AEROBIC SYSTEMS, INC.
A Metawater Company

Aqua-Aerobic®

Cloth Media Filter Featuring OptiFiber® Pile Cloth Media

In the early 1990s, Aqua-Aerobic Systems revolutionized tertiary treatment by introducing Pile Cloth Media Filtration utilizing a disk configuration. Since then, over 3,000 pile cloth media filtration units have been installed worldwide, and hundreds of different media have been researched and tested with a select few that are currently being applied to six mechanical configurations in a variety of applications including: water reuse, low level phosphorus, stormwater and primary treatment.

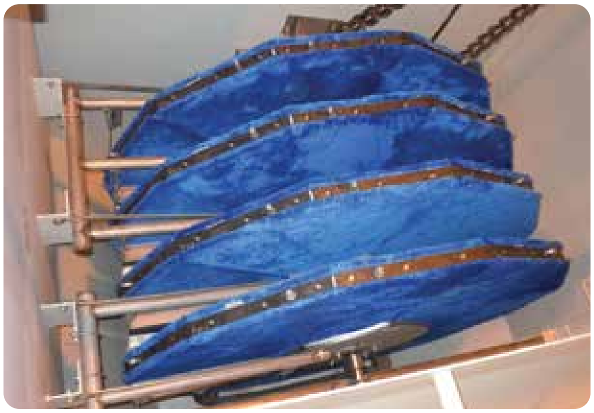
Effective Depth Filtration

The original OptiFiber® pile cloth media is specifically engineered for water and wastewater applications and designed to maximize solids removal over a wide range of particle sizes. Deep, thick, pile fibers capture particles for the most effective depth filtration. Perhaps as important, the media is engineered to backwash effectively and last over time. OptiFiber media is exclusive to the entire line of cloth media filter configurations including:

- **AquaDisk®**
- **Aqua MegaDisk®**
- **AquaPrime®**
- **AquaStorm™**
- **AquaDiamond®**
- **Aqua MiniDisk®**

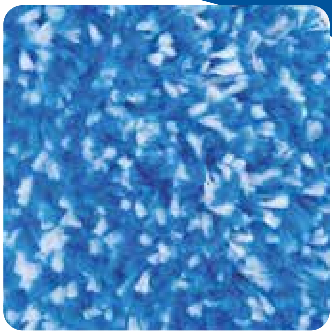
OptiFiber® Media Advantages

- Woven, precision fibers provide strength and durability
- Discrete pile fibers effectively release solids during backwash
- Open backing minimizes potential for biofouling
- Low backwash volume results in water savings and energy reduction
- Variety of application-specific cloth including 2, 5 & 10 μm nominal pore size media
- Phosphorus removal to 0.075 mg/l or less
- Ability to handle high solids conditions

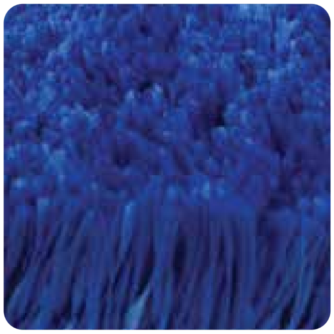


An AquaDisk® filter with OptiFiber PES-14® treats cooling tower blow-down.

OptiFiber PA2-13®



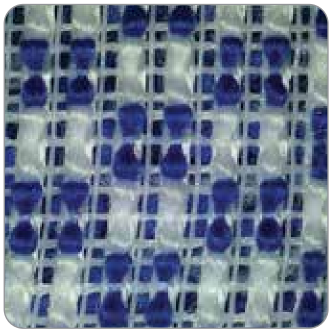
OptiFiber PES-13®



OptiFiber PES-14®

OptiFiber UFS-9™

OptiFiber PF-14®



Unique Backing Design

OptiFiber® Cloth Filtration Media
Awarded BlueTech® Research Innovation Badge

Engineered Cloth Media

The media is the most important aspect in any filter design. Today's OptiFiber® pile cloth filtration media is the result of over 30 years of continuous engineering and improvement. Each aspect of the pile cloth is design is engineered to provide an optimal design to maximize particle removal, allow for effective backwash, and maximize media life.

Hundreds of media options have been tested as part of this continuous development process. Only five of these options have made it through the rigorous testing process and met the quality standards set forth by Aqua-Aerobic Systems, Inc.

A cloth media display showcases samples of tested media with the far left panel featuring OptiFiber® media.

OptiFiber® Cloth Media Technology Timeline

1993	2000	2004	2006	2011	2013	2016	2017	Continued Innovation
AquaDisk® Filter	OptiFiber PA2-13® Media	AquaDiamond® Filter	OptiFiber PES-13® Media	OptiFiber PES-14® Microfiber Media	Aqua MegaDisk® Filter	AquaPrime® & AquaStorm™ Filter with OptiFiber PF-14® Media	OptFiber UFS-9® Ultrafiber Media	

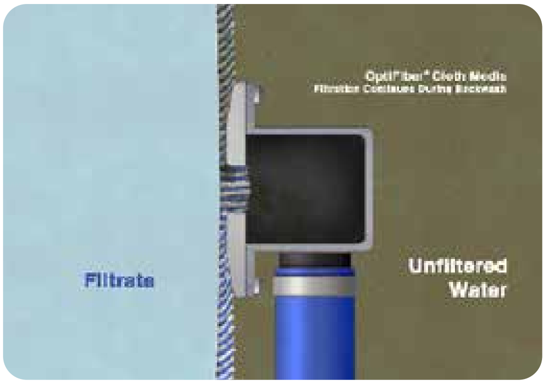
Backwash System

Effective Cleaning With Less Water and Energy

Maximum cleaning of the OptiFiber® cloth media is accomplished with a unique backwash system. The backwash shoe makes direct contact with the cloth media and solids are vacuumed from the surface. During backwash, fibers fluidize to provide an efficient release of stored solids deep within the fiber depth.

Backwash System Advantages

- Filtration continues during backwash
- Initiated at a pre-determined liquid level or time
- Low backwash rates
- Less water volume required
- Low energy consumption



Backwash shoe makes direct contact with the media.

Shown is pile cloth media in its natural state (left) and its conditioned state (right).

Configurations

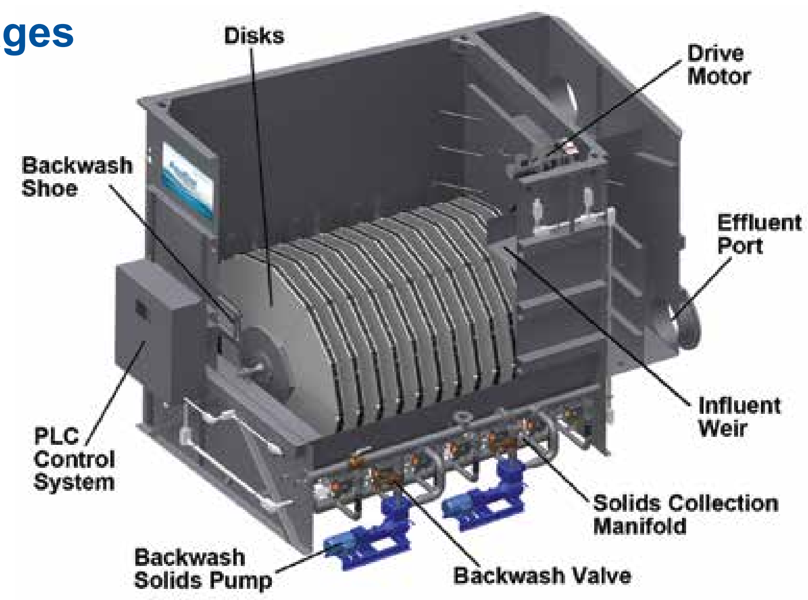
AquaDisk®

Cloth Media Filter

The cloth media "Disk" configuration was the first to enter the marketplace as an alternative to conventional granular media filtration technologies. This original configuration comprises the majority of Aqua-Aerobic cloth media filters installed today. A history of exceptional operating experience in a variety of municipal and industrial applications continues to make the AquaDisk® the tertiary filter of choice.

System Features and Advantages

- Vertically oriented cloth media disks reduce required footprint
- Each disk has six lightweight, removable segments for ease of maintenance
- Low hydraulic profile
- Higher solids and hydraulic loading rates
- Low backwash rate
- Available in painted steel, stainless steel or concrete tanks
- Fully automatic PLC control system with color touchscreen HMI
- Low cost of ownership



Modes of Operation

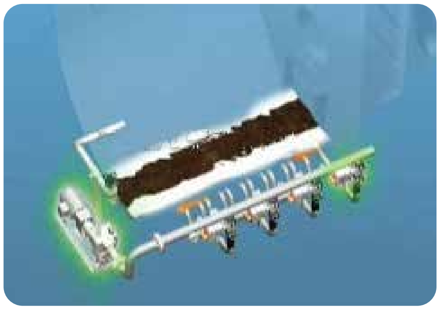
Aqua-Aerobic cloth media filter configurations operate on the same (3) modes of operation: **FILTRATION, BACKWASH** and **SOLIDS WASTING.**

Filtration Mode

- Inlet wastewater enters filter
- Cloth media is completely submerged
- Disks are stationary
- Solids deposit on outside of cloth media forming a mat as filtrate flows through the media
- Tank liquid level rises
- Flow enters the filter by gravity and filtrate is collected inside the disks and discharged
- Heavier solids settle to the tank bottom

Backwash Mode

- Solids are backwashed at a predetermined liquid level or time
- Backwash shoes contact the media directly and solids are removed by vacuum pressure using the backwash pump
- Two disks are backwashed at a time (unless a single disk is utilized)
- Disks rotate slowly
- Filtration is not interrupted
- Backwash water is directed to headworks



Solids Wasting Mode

- Heavier solids on the tank bottom are removed on an intermittent basis
- Solids are pumped back to the headworks, digester or other solids collection area of the treatment plant

Aqua MegaDisk®

Cloth Media Filter

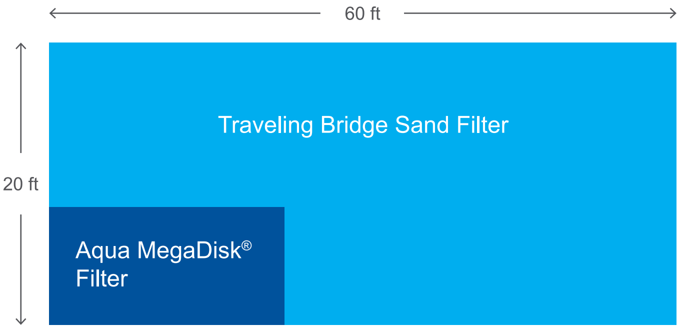
The Aqua MegaDisk® cloth media filter expands on the reliability and exceptional performance of the original AquaDisk filter, but on a larger scale. Each disk is approximately 10' in diameter. The unit features all of the same benefits and (3) modes of operation as the AquaDisk but with larger disks.

Additional Features and Advantages

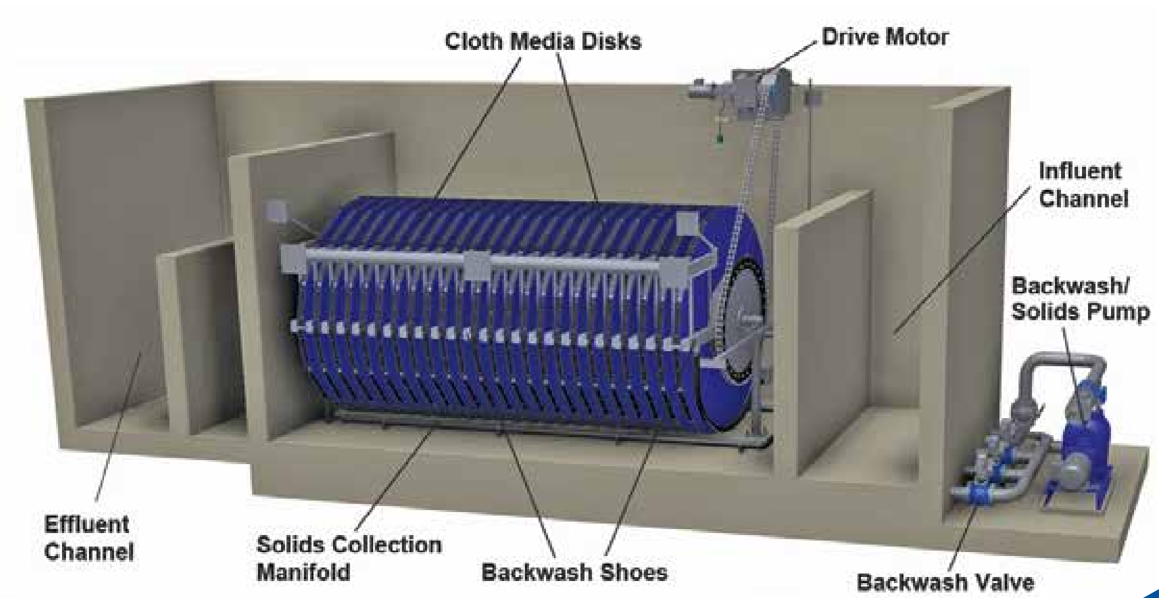
- Smallest footprint, operating in 80% less space than sand filters with comparable hydraulic capacity
- Up to 24 disks in a single filter, capable of treating 24 MGD
- Ideal for deep bed sand filter retrofits, new plants or expansions
- Lightweight segments removable without a crane

Aqua MegaDisk® (left) compared to AquaDisk® (right).

Footprint Savings Compared to Sand Filters



Internal view of the Aqua MegaDisk®



Configurations

Aqua MiniDisk®

Cloth Media Filter

The Aqua MiniDisk® cloth media filter features all of the same benefits and (3) modes of operation as the original AquaDisk. The configurations are designed to provide economical treatment of smaller flows and easily retrofit into existing traveling bridge sand filters.

The Aqua MiniDisk® cloth media filter is available as packaged unit(s) or concrete basin(s).

The modular design of the Aqua MiniDisk® filter retrofits neatly into existing 9 ft. (2.74 m) wide concrete traveling bridge filter basins, providing more than two times the hydraulic capacity of the original sand filters.

Filter IntelliPro®

Filtration Optimization System

Building from a decade of experience in applying advanced process control, Filter IntelliPro® is a control system for cloth media filters that uses real time data to optimize chemical usage for phosphorus removal prior to filtration. Among its many features, the system includes automatic optimal dose selection for metal salt, polymer, and pH control.

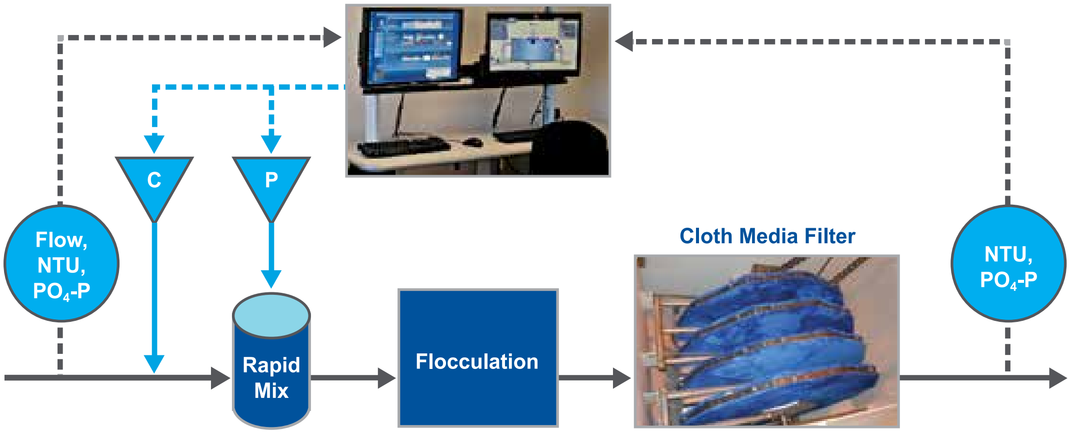
System Features

- PC with IntelliPro software developed by Aqua-Aerobic Systems, Inc.
- Network settings to allow communication between the instruments, the PLC and the PC
- Process, instrumentation and software on-site training

System Advantages

- Chemical savings through load based control
- Automatic chemical dose response curves replace jar testing
- Improved process reliability using real time information
- Multi-point analysis of key process parameters

IntelliPro® System Layout for Ultra-Low Phosphorus



AquaDiamond®

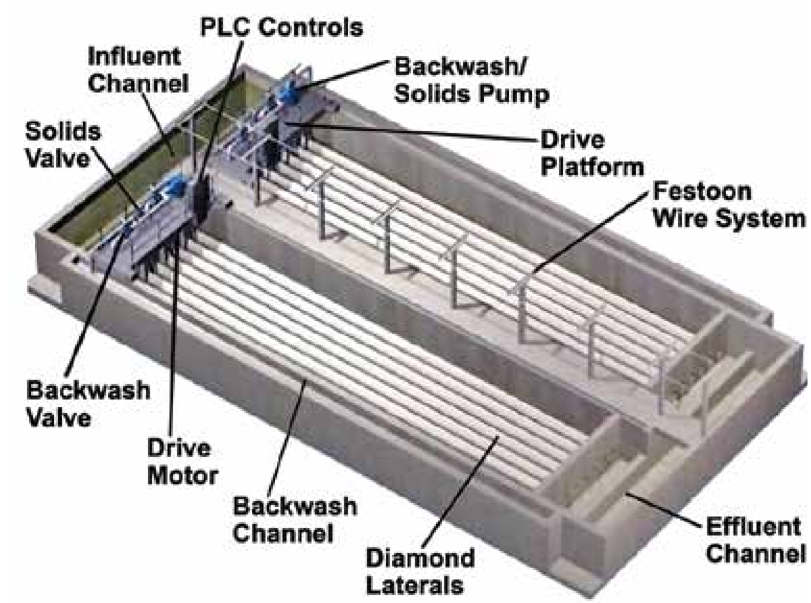
Cloth Media Filter

The AquaDiamond® cloth media filter is a unique combination of two proven technologies: traveling bridge and cloth media filters. The result is two to three times the flow capacity of a traveling bridge filter within an equivalent footprint, making it ideal for sand filter retrofits. The unit features all of the same benefits and (3) modes of operation as the AquaDisk but with vertically oriented diamond laterals and a traveling platform.

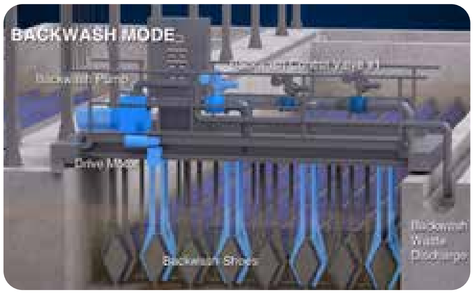
Additional Features and Advantages

- Up to eight diamond laterals per unit
- Fits neatly into existing traveling bridge filter profile with minimal civil work
- Variable speed drive platform and backwash pump provide immediate response to influent solids excursions
- Advanced drive and tracking system prevents misalignment

An AquaDiamond® filter with Microfiber cloth polishes phosphorus to < 0.1 mg/l.



Modes of Operation



Filtration Mode

- Inlet wastewater enters the filter
- Cloth media is completely submerged
- No moving parts
- Solids deposit on outside of cloth media forming a mat as filtrate flows through the media
- Flow enters the filter by gravity and filtrate is collected inside the diamond laterals and discharged
- Heavier solids settle to the basin floor

Backwash Mode

- Periodic backwashing is initiated by increased headloss due to solids deposits
- The platform traverses the length of the cloth media diamond laterals during backwashing
- Backwash shoes contact the media directly and solids are removed by vacuum pressure using the backwash pump
- The platform only operates during backwash and solids collection

Filtrate Collection and Discharge

- Heavier solids on the tank bottom are removed on an intermittent basis
- Small suction headers collect and discharge settled solids
- The backwash pump is utilized for solids removal

High Solids Applications

Primary Filtration and Wet Weather Treatment

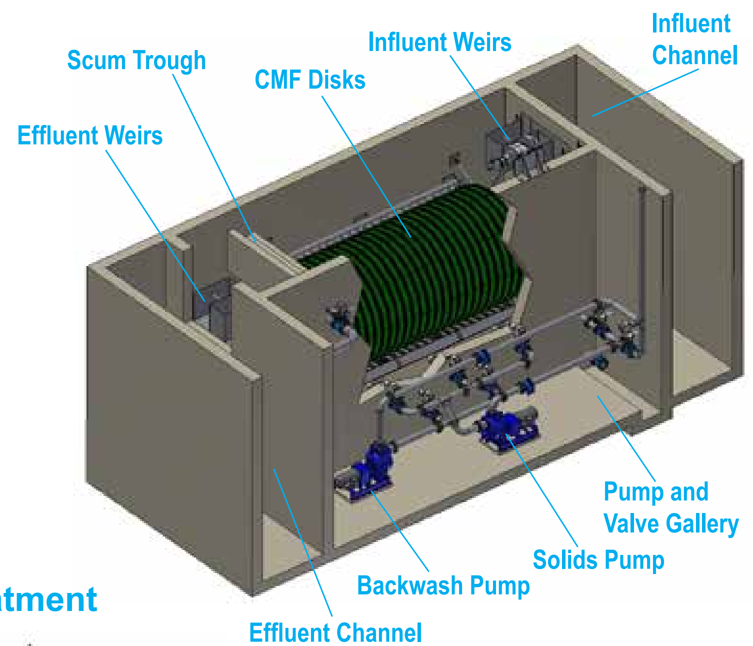
AquaPrime®

Cloth Media Filter

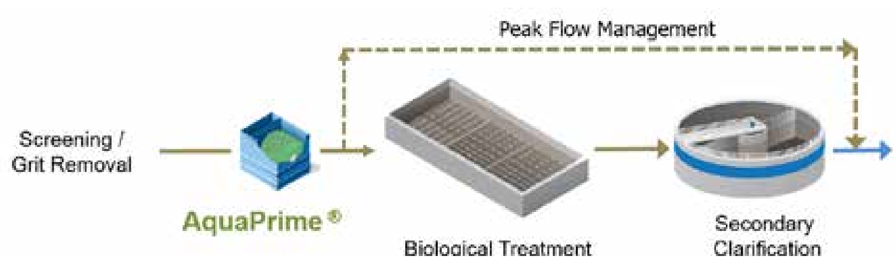
The AquaPrime® cloth media filter is ideal for primary wastewater treatment due to its proven removal efficiencies. The main advantages include extremely small footprint, reduced energy costs in the secondary process due to a reduction in organic loading and more solids for increased gas production in anaerobic digesters for primary applications.

AquaPrime® Features and Advantages

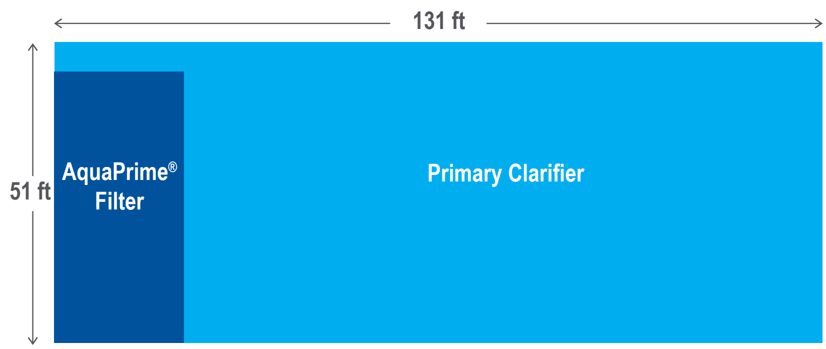
- Vertically oriented cloth media disks reduce required footprint to 15% to 20% of primary clarification
- Provides enhanced solids and BOD removal resulting in:
  - Less aeration energy for secondary process due to reduced organic loading
  - More solids for increased biogas production in anaerobic digesters
  - Increased capacity in existing secondary process basin
- Three methods of solids removal with specifically designed floatable, filtration and solids removal zones
- Dual use applications of advanced primary treatment and wet weather treatment
- Major capital construction savings



Flow Diagram for Advanced Primary Treatment



80-85% Footprint Savings Compared to Primary Clarifier



Linda County Water District, Olivehurst, CA

- Primary Filtration Application
- TSS removal greater than 75%
- BOD removal up to 60%

AquaStorm®
Cloth Media Filter

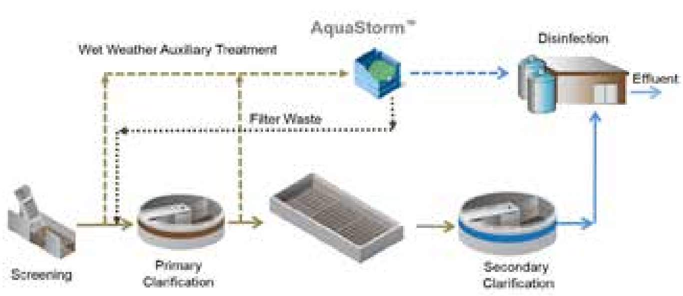
The AquaStorm® cloth media filter features a similar mechanical configuration as the AquaPrime® filter, as well as offers inherent advantages related to wet weather treatment for stormwater, Combined Sewer Overflow (CSO) and Sanitary Sewer Overflow (SSO), including the ability to be configured for dual-use applications for tertiary and wet weather operation. Also, differences in controls specifically designed to handle intermittent operation and need for lower effluent requirements for wet weather applications.

AquaStorm® Features and Advantages

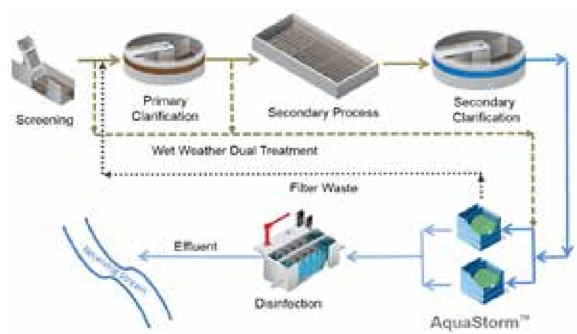
- High quality effluent similar to secondary standards
- Use with or without chemical, depending on site-specific effluent water quality requirements
- Can be configured for dual-use application for tertiary or wet weather operation
- Simple start-up and shutdown with unattended operation for remote locations
- Provides the treatment facility with resiliency during wet weather events
- Maximizes the wet weather flows to be treated
- Protects the biological portion of the facility
- Can be used at remote CSO/SSO sites
- Improves disinfection of wet weather flows

Flow Diagrams for Wet Weather Treatment

Side Stream

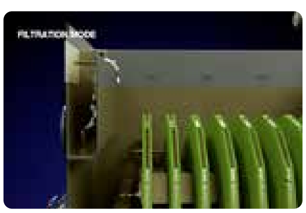


Dual Treatment



Modes of Operation

The AquaPrime® and AquaStorm® cloth media filtration system operates on four (4) modes of operation: **FILTRATION, BACKWASH, SOLIDS WASTING** and **FLOATABLE WASTING**.



Filtration Mode

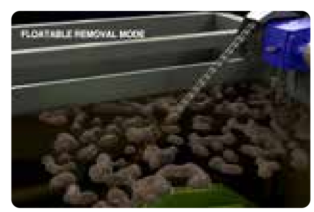
- Inlet wastewater enters filter by gravity
- Cloth media is completely submerged and stationary
- Solids deposit on outside of cloth media forming a mat as filtrate flows through the media
- Filtrate is collected inside the disks and discharged
- Heavier solids settle to the tank bottom
- Tank liquid level rises

Backwash Mode

- Solids are backwashed at a predetermined liquid level or time
- Backwash shoes contact the media directly and solids are removed by vacuum pressure using the backwash pump
- 2 to 8 disks are backwashed at a time
- Disks rotate slowly
- Filtration is not interrupted
- Backwash water is directed to the waste handling facility or headworks (AquaStorm)

Solids Wasting Mode

- Heavier solids are collected in the hoppers and are removed on an intermittent basis
- After a preset number of backwashes, a solids wasting occurs
- Backwash/Solids Pump provides suction to the solids collection manifold for wasting of settled solids
- Solids are pumped back to the waste handling facility or headworks (AquaStorm)



Floatable Wasting Mode

- Floatable scum is allowed to collect on the water surface
- After a preset amount of time, the water level is allowed to rise above the preset floatable setpoint
- As the water level increases, floating scum is removed by flowing over the scum removal weir
- Scum wasting water is directed to the plant's waste handling facility

Cloth Media Filtration
Mobile Pilot Systems

Technology pilot demonstrations can be beneficial to wastewater treatment plants by providing a snapshot of essential process operating conditions and allowing the customer to interact with the technology and Aqua-Aerobic personnel. OptiFiber cloth media filter pilot systems provide customers with the most comprehensive on-site testing and analytical services available. Our unique approach is designed to provide prompt operational feedback, allowing immediate fine-tuning of parameters for the most effective pilot/demonstration experience.

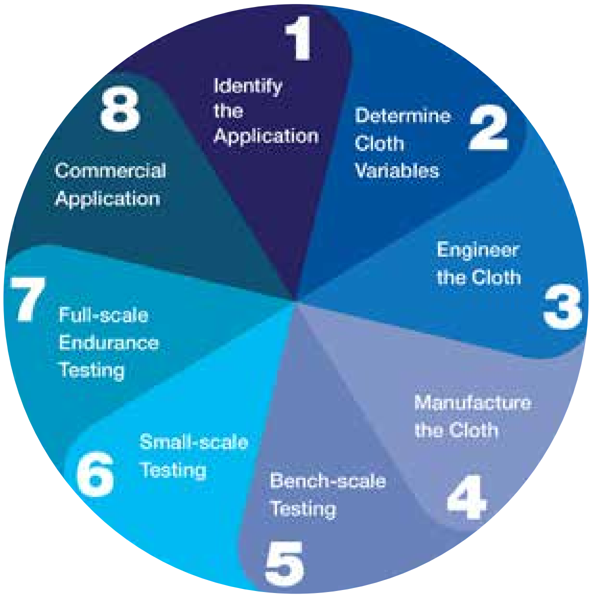
Fully Equipped Laboratory

Mobile Primary Filtration Pilot System

Aqua-Aerobic Research & Technology Center

In 2011, Aqua-Aerobic Systems, Inc. in partnership with the Rock River Water Reclamation District (Rockford, IL) built a new Research & Technology Center at the District's central treatment plant. The facility was constructed for the purpose of conducting applied research and demonstration of new products and processes for treating wastewater. The Center is integral in developing and testing cloth filtration media for future commercialization and application, both domestically and internationally.

Customers visit the R&T Center as part of the technical seminar program.



OptiFiber® media development: an eight step, three year process

All Aqua-Aerobic® cloth media filtration products offer a "green" advantage including lower energy consumption and reduced water usage.

Application Profiles

Municipal Recycle/Reuse

- Hundreds of installations
- Title 22 approved
- Multiple cloths capable of producing effluent below 1.0 NTU

Phosphorus Removal

- Achieve phosphorus removal below 0.075 mg/l
- Depth of filtration means less chemical/ flocculation and energy

Traveling Bridge Filter Retrofits

- 2-3 times hydraulic capacity within existing footprint
- Minimal mechanical components and no civil changes

Deep Bed Filter Retrofits

- 3-4 times hydraulic capacity within existing footprint
- Minimal mechanical components and no civil changes

Industrial

- Robust cloth media handles high industrial solids
- Applied in several industrial applications including: Energy, Food/Beverage, Textile and Pharmaceutical

Large Flows

- Ideal application for Aqua MegaDisk® and AquaDiamond® filters
- Smallest footprint when compared to hydraulic capacity
- Experience in large flow filter designs over 50 MGD

Power and Energy

- Removes coal ash and coal fines from runoff or wastewater streams
- Reduces TSS and NTU for process water
- Provides reuse water for cooling

Stormwater/CSO/SSO

- Effectively removes TSS without chemicals
- Easily accommodates varying flows
- Can provide tertiary treatment between rain events

Primary Filtration

- Reduce organic load to secondary process
- Lower energy consumption
- Replace existing primary clarifiers
- Increased biogas production

Since 1969, Aqua-Aerobic Systems, Inc. has led the industry by providing advanced solutions in water and wastewater treatment. As an applied engineering company serving both municipal and industrial customers, we work collaboratively with consulting engineers, owners, plant managers, and operators to design and manufacture the best treatment solution with the lowest lifecycle cost.

Providing TOTAL Water Management Solutions

Aeration & Mixing

Biological Processes

Filtration

Oxidation & Disinfection

Membranes

Controls & Monitoring Systems

Aftermarket Products and Services

Cloth Media Filtration

Featuring OptiFiber® Pile Cloth Media

Visit our website at www.aqua-aerobic.com to learn more about the Cloth Media Filtration Featuring OptiFiber® Pile Cloth Media and our complete line of products and services.

AQUA-AEROBIC SYSTEMS, INC.
A Metawater Company

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Bulletin #601B 4/22

Appendix E: Criteria for Scoring Decision Matrices

Table E-12-1: Preliminary Treatment Scoring Criteria

	Rating				
Criterion	5	4	3	2	1
Life Cycle Costs (Capital Cost and O&M)	50% less expensive than current costs	25% less expensive than current costs	Within 5% of current costs	25% more expensive than current costs	50% more expensive than current costs
Removal Efficiency	Significant improvement in removal efficiency		Some improvement in removal efficiency		Removes same amount and size of material as current conditions
Minimizing Construction Time	Less than a month	Less than six months	Less than a year	Less than two years	Greater than two years
Adaptable Capacity *	Meets four or more additional criteria	Meets three additional criteria	Meets two additional criteria	Meets one additional criterion	Meets design capacity

*Accomplishes Flow Equalization, Improves Redundancy, Includes Further Scale Up Potential, Particularly Fast HRT, etc.

Table E-12-2: Primary Treatment Scoring Criteria

	Rating				
Criterion	5	4	3	2	1
Life Cycle Costs (Capital Cost and O&M)	50% less expensive than current costs	25% less expensive than current costs	Within 5% of current costs	25% more expensive than current costs	50% more expensive than current costs
Downstream Effects	Significant improvement		Some improvement		Negligible effect
Minimizing Construction Time	Less than a month	Less than six months	Less than a year	Less than two years	Greater than two years

Table E-12-3: Secondary Treatment Scoring Criteria

	Rating				
Criterion	5	4	3	2	1
Capital Cost	No additional costs	Less than 1 million	Less than 2 million	Less than 3 million	Greater than 3 million
O&M and Life Cycle Cost	50% less expensive than current costs	25% less expensive than current costs	Within 5% of current costs	25% more expensive than current costs	50% more expensive than current costs
Ability to Meet Permit Limits	Far exceeds permit requirements		Exceeds permit requirements		Meets permit requirements
Minimizing Construction Time	Less than a month	Less than six months	Less than a year	Less than two years	Greater than two years
Adaptable Capacity *	Meets four or more additional criteria	Meets three additional criteria	Meets two additional criteria	Meets one additional criterion	Meets design capacity

* Accomplishes Flow Equalization, Improves Redundancy, Includes Further Scale Up Potential, Particularly Fast HRT, etc.

Table E-12-4: Advanced Treatment Scoring Criteria

	Rating				
Criterion	5	4	3	2	1
Life Cycle Costs (Capital Cost and O&M)	50% less expensive than current costs	25% less expensive than current costs	Within 5% of current costs	25% more expensive than current costs	50% more expensive than current costs
Water Quality*	Meets four or more additional criteria	Meets three additional criteria	Meets two additional criteria	Meets one additional criterion	No Improvement from current conditions
Minimizing Construction Time	Less than a month	Less than six months	Less than a year	Less than two years	Greater than two years
Downstream Effects	Significant improvement		Some improvement		Negligible effect

*Improves TSS, Improves Turbidity, Prepares Water for Alternate Use, Improves Phosphorous content, etc.

Table E-12-5: Disinfection Scoring Criteria

	Rating				
Criterion	5	4	3	2	1
Life Cycle Costs (Capital Cost and O&M)	50% less expensive than current costs	25% less expensive than current costs	Within 5% of current costs	25% more expensive than current costs	50% more expensive than current costs
Ability to Meet Permit Limits	Far exceeds permit requirements		Exceeds permit requirements		Meets permit requirements
Minimizing Construction Time	Less than a month	Less than six months	Less than a year	Less than two years	Greater than two years
Efficiency Rate	Less than half an hour	Half an hour or more	One hour or more	Six hours or more	Twelve hours or more

Table E-12-6: Solids Handling Scoring Criteria

	Rating				
Criterion	5	4	3	2	1
Capital Cost	No additional cost	Under $500,000	Under $1 million	Under $1.5 million	Greater than $1.5 million
O&M and Life Cycle Cost	50% less expensive than current costs	25% less expensive than current costs	Within 5% of current costs	25% more expensive than current costs	50% more expensive than current costs
Ability to Meet Permit Limits	Far exceeds permit requirements		Exceeds permit requirements		Meets permit requirements
Minimizing Construction Time	Less than a month	Less than six months	Less than a year	Less than two years	Greater than two years
Environmental and Social Impacts*	Meets four or more additional criteria	Meets three additional criteria	Meets two additional criteria	Meets one additional criterion	As good as current conditions

* Reduces sludge volume/water content, reduces odor, energy production, reduces energy consumption, prepares sludge for alternative use, etc.

Appendix F: Detailed Decision Matrices

Table F-1: Preliminary Treatment Detailed Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Band screen with an In-to-Out Screen from JWCE	Alternative 3: Add grinder to the preliminary treatment process
Life Cycle Costs (Capital Cost and O&M)	30%	3	2	1
		No capital cost, 8k annual energy cost	500k estimation for capital cost, 8k annual energy cost	600k estimation in capital cost, 16k energy cost per year
Removal Efficiency	30%	1	3	4
		Commonly gets rags stuck	Minimize the rags getting stuck but does not completely solve the problem	Solves rag problem and improves general removal
Minimizing Construction Time	20%	5	4	3
		No construction time	Less than six months (phasing the replacement of screens, most likely one at a time)	Less than a year
Adaptable Capacity	20%	1	3	2
		No additional benefit	Meets current capacity, further scale up potential, improves redundancy	Meets current capacity, further scale up potential
Total	100%	2.4	2.9	2.5

Table F-2: Primary Treatment Detailed Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Primary Clarifier
Life Cycle Costs (Capital Cost and O&M)	30%	3	1
		No additional capital cost	$1.2M
Downstream Effects	40%	1	2
		5% more sludge to handle	Decreases the BOD of the water by approximately 20% and decreases sludge content by 5%
Minimizing Construction Time	30%	5	2
		No construction needed	Takes less than 2 years
Total	100%	2.8	1.7

Table F-3: Secondary Treatment Detailed Decision Matrix

Criterion	**Weight**	**Alternative 1: Conventional Oxidation with Denitrification**	**Alternative 2: Convert to half and half**	**Alternative 3: Addition of a sixth plant**
Capital Cost	25%	4	4	1
		Minor capital costs related to reprogramming and refurbishing infrastructure	Minor capital costs related to reprogramming and refurbishing infrastructure	$3,041,399.35 capital cost
O&M and Lifetime Cycle Cost	25%	3	2	1
		No significant changes to O&M expected	Requires regular purchasing of BOD feed stock such as methanol	Significant increases relating to energy consumption and maintenance
Ability to Meet Permit Limits	15%	1	2	1
		Meets permit limits	Use of BOD feedstock allows for some further denitrification	Meets permit limits
Minimizing Construction Time	25%	5	5	2
		Less than a month if any construction is required	Less than a month	Probably around 2yrs
Adaptable Capacity	10%	3	1	4
		May reduce sludge volume through digestion in the anaerobic zone, improves redundancy	Meets design capacity	Flow equalization, improves redundancy, has scale up potential
Total	100%	3.45	3.15	1.55

Table F-4: Advanced Treatment Detailed Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Reincorporate antiquated sand filter system	Alternative 3: Add membrane filtration
Life Cycle Costs (Capital Cost and O&M)	40%	3	2	1
		Slight change to current costs due to expanded design capacity	It is significantly cheaper than the RO system. Yet more expensive than not changing the treatment technology	Utilized cost for RO system is $8.7 million. Over 30 years
Water Quality	20%	1	1	5
		No improvement from current conditions	Combination of disk filters and sand filtration will not improve from current conditions	Improves TSS, turbidity, phosphorus content and prepares water for alternative use
Minimizing construction time	25%	5	3	2
		No large infrastructure will need to be added, meaning construction time will be less than month	Due to the need to reroute water flow and hydraulics that construction time will be less than a year	Due to the complicated nature of an RO system construction time will be less than two years
Downstream Effects	15%	1	3	4
		No improvement from current conditions	Improves redundancy of the water reclamation facility	Improves water quality but does not necessarily mean downstream treatment will become 100% efficient
Total	100%	2.8	2.2	2.5

Table F-5: Disinfection Detailed Decision Matrix

Criterion	**Weight**	**Alternative 1: No change to treatment technology**	**Alternative 2: Ultraviolet disinfection**	**Alternative 3: Ozone disinfection**
Life Cycle Costs (Capital Cost and O&M)	40%	3	1	2
		Capital Cost is nonexistent since no new equipment is being bought. Simple operation and maintenance due to the Microclor OSHG having clear and replaceable cells	\$1.1 million capital; \$50,000/year in O&M	Capital Costs include \$245,500 for Oxygen feed gas and compressor \$5,000 for Contact Vessel (500 gpm). Total O&M is \$138,500/year
Ability to Meet Permit Limits	25%	1	2	2
		Will meet permit with increased injection of chlorine	As long as TSS stays within design parameters sufficient disinfection will occur	Through utilization of current contact basin and ozone injection, capacity will be met and has the potential to be scaled up for a higher demand
Minimizing Construction Time	25%	5	2	4
		No additional construction is needed	New facilities will need to be constructed to coincide with existing effluent flow path. Meaning less than two years	The same contact basins used for chlorination can be used, minimizing construction. To add, contact time for ozone disinfection is significantly less that chlorination
Contact Time	10%	2	5	4
		Average contact time is 6 hours.	Average contact time of 20 - 60 seconds	10-30 minutes
Total	100%	2.9	1.9	2.7

Table F-6: Solids Handling Detailed Decision Matrix

Criterion	Weight	Alternative 1: No change to treatment technology	Alternative 2: Repurpose antiquated aerobic digesters to anaerobic used in combination with operational solar drying bed
Capital Cost	20%	5 No capital cost	3 Reactivate anerobic digesters in combination with existing heating bed infrastructure. Capital cost of $400,000 to $800,000. Capital Cost of air scrubber and gas collection system
O&M and Life Cycle Cost	25%	3 No Additional O&M	4 Reactivate anerobic digesters in combination with existing heating bed infrastructure. Annual operation and maintenance costs of $50,000 to $100,000. With a potential annual methane profit margin of $157,000 per year
Ability to Meet Permit Limits	20%	1 Current solids handling system meets permit limits of the new design capacity	3 Combination of anerobic digestion and solar drying beds will enhance the quality of biosolids
Minimizing Construction Time	15%	5 No construction needed	2 18 months to ensure the antiquated digestors are safe and fully operational, as well as construction of gas storage tank
Environmental and Societal Impacts	20%	1 System will bring no environmental or social benefits since the biosolids are being transported to landfill	4 As good as current conditions, produces energy, reduces sludge volume, upgrade of biosolids will provide opportunity for the agricultural industry to buy the biosolids as fertilizer
Total	100%	2.9	3.3

Appendix G: Plant 6 Analysis

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Hazen-Williams Equation:

$$S = \frac{h_f}{L} = \left[\frac{Q}{A K C R_H^{0.63}}\right]^{1/0.54}$$

$$A = \frac{\pi D^2}{4} \qquad R_H = \frac{D}{4}$$

FOR FULL PIPE

RAS lines:

Exisitng Plant 4 (same for 5):			Proposed Plant 6:			
k:	1.318		k:	1.318		
Q:	10.8304	(ft^3)/s	Q:	10.8304	(ft^3)/s	
pipe diameter:	1.333333	ft	pipe diameter:	1.5	ft	DIP material
A:	1.395556	ft^2	A:	1.76625	ft^2	Pipe dia. increased to 18 in. to get same head loss as other RAS lines (iteration)
C:	130		C:	130		
Rh:	0.333333	ft	Rh:	0.375	ft	
L:	512	ft	L:	835	ft	
head loss:	0.79	ft	head loss:	0.72	ft	
Existing Plant 4 and 5 head loss:	1.57	ft				

Pipe from splitter box to plant 6 oxidation ditch:

Plant 3:			Plant 4 (same for 5):			Plant 6:			
k:	1.318		k:	1.318		k:	1.318		
Q:		(ft^3)/s	Q:	10.8304	(ft^3)/s	Q:	10.8304	(ft^3)/s	
pipe diameter:	1.333333	ft	pipe diameter:	2.5	ft	pipe diameter:	3	ft	
A:	1.395556	ft^2	A:	4.90625	ft^2	A:	7.065	ft^2	Upgrade to 36 inch DIP to roughly match headloss
C:	130		C:	130		C:	130		
Rh:	0.333333	ft	Rh:	0.625	ft	Rh:	0.75	ft	
L:		ft	L:	106	ft	L:	296	ft	
head loss:		ft	head loss:	0.007623	ft	head loss:	0.00876	ft	

Manning's Equation

$$S = \left(\frac{Q * n}{1.49 * A * R^{\frac{2}{3}}}\right)^2$$

S is the slope of the energy grade line and $S = h_f/L$ where h_f is energy (head) loss and L is the length

$$R_h = \frac{A}{WP} = \frac{b \cdot y}{b + 2y}$$

L:	228	ft	
Q:	15.472	(ft^3)/s	(equivalent of 10MGD, 4MGD multipied by 2.5 peaking factor)
n:	0.013		(concrete, trowel finish)
b:	8	ft	
y:	2.4	ft	*from excel goal seek analysis for critical depth
R:	1.50	ft	
A:	19.2	ft^2	
head loss due to new ditch:	0.033	ft	

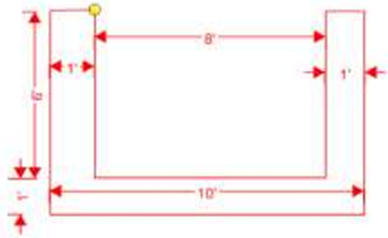
New Trough:

Length:	228	ft
Concrete Area of cross section:		ft^2
Concrete Volume (ft^3)	4800.125	ft^3
Concrete Volume (CY)	177.78	CY

New Clarifier:

Bottom Slab Thickness:	1	ft
Area of bottom slab:	11082	ft^2
Volume of Slab Concrete:	11082	ft^3
Volume of Slab Concrete:	410.44	CY
Wall Height:	20	ft
Wall Thickness:	1	ft
Length of Wall:	385	ft
Volume of Wall Concrete:	7700	ft^3
Volume of Wall Concrete:	285.1852	CY
Total concrete volume for plant 6 clarifier:	695.63	CY

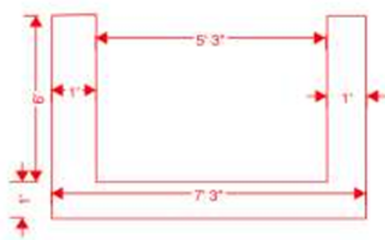
Section 1



Section 1:

Concrete Area of cross section:	22	ft^2
Length:	172.25	ft
Concrete Volume (ft^3)	3789.5	ft^3

Section 2



Section 2:

Concrete Area of cross section:	19.25	ft^2
Length:	52.5	ft
Concrete Volume (ft^3)	1010.625	ft^3

Selector Basin concrete:

Area for slab	1,670	ft^2
Slab Thickness:	1	ft
Volume of Concrete:	1670	ft^3
Volume of Concrete:	61.85	CY
Length of Walls:	240	ft
Thickness of Wall:	1	ft
Height of Wall:	15.5	ft
Volume of Concrete:	3720	ft^3
Volume of Concrete:	137.78	CY
Total Concrete:	199.63	CY

Infrastructure	Unit Cost ($)	Total Cost ($)
parshall flume	$ 3,530.00	$ 3,530.00
selector basins (2) (concrete)	$ 147.00	$ 29,345.56
heavy duty sluice gate assembly	$ 37,900.00	$ 37,900.00
9 meter maxi-rotor	$ 60,000.00	$ 960,000.00
baffle	$ 1,000.00	$ 16,000.00
ditch construction (concrete)	-	$ 570,600.00
ditch construction (wire mesh and rebar)	23 cents per sf for mesh and 40 cents per lf of concrete	$ 14,767.67
Concrete Forms	$6 per sf	$ 141,492.00
DIP (18" dia)	$ 211.00	$ 176,185.00
DIP (36" dia)	$ 222.00	$ 65,712.00
new trough (concrete)	$ 147.00	$ 51,134.01
Structural Fill	$ 25.01	$ 352,326.06
clarifier (concrete)	$ 147.00	$ 127,257.56
clarifier mechanism	-	$ 370,000.00
scum box		$ 39,375.00
glass lined 12" DIP	$ 288.00	$ 80,640.00
DIP (6" scum drain line)	$ 163.00	$ 5,134.50
	Total:	$ 3,041,399.35

List of Cost Sources:

RSMeans

https://www.openchannelflow.com/blog/how-much-do-fiberglass-frp-parshall-flumes-cost

https://lawnlove.com/blog/concrete-cost/#:~:text=Reinforcement%20ensures%20that%20the%20concrete,foot%2C%20to%20reinforce%20the%20concrete.

https://www.homeblue.com/concrete/phoenix-az-concrete-cost-per-yard.htm

https://cms9files.revize.com/grantsvilleut/Document_Center/Department/Public%20Works/Grantsville%20WW%20Study%202022%20FINAL%20DRAFT%202022-11-10%20Final%20Draft.pdf

https://jmsequipment.com/typical-cost-for-a-scum-pipe-system/

Appendix H: Life Cycle Cost Analysis of Anerobic Digesters

Table G-1: Life Cycle cost Analysis of Anerobic Digesters

Life Cycle Cost of Anerobic Digestion and Biogas Production				
	Capital Cost ($)	O & M ($)	Savings ($)	Owed ($)
Year 0	$1,000,000.00	$-	$-	$(1,000,000.00)
Year 1	$-	$80,000.00	$157,522.87	$(922,477.13)
Year 2	$-	$80,000.00	$157,522.87	$(844,954.27)
Year 3	$-	$80,000.00	$157,522.87	$(767,431.40)
Year 4	$-	$80,000.00	$157,522.87	$(689,908.54)
Year 5	$-	$80,000.00	$157,522.87	$(612,385.67)
Year 6	$-	$80,000.00	$157,522.87	$(534,862.80)
Year 7	$-	$80,000.00	$157,522.87	$(457,339.94)
Year 8	$-	$80,000.00	$157,522.87	$(379,817.07)
Year 9	$-	$80,000.00	$157,522.87	$(302,294.20)
Year 10	$-	$80,000.00	$157,522.87	$(224,771.34)
Year 11	$-	$80,000.00	$157,522.87	$(147,248.47)
Year 12	$-	$80,000.00	$157,522.87	$(69,725.61)
Year 13	$-	$80,000.00	$157,522.87	$7,797.26
Year 14	$-	$80,000.00	$157,522.87	$85,320.13
Year 15	$-	$80,000.00	$157,522.87	$162,842.99
Year 16	$-	$80,000.00	$157,522.87	$240,365.86
Year 17	$-	$80,000.00	$157,522.87	$317,888.73
Year 18	$-	$80,000.00	$157,522.87	$395,411.59
Year 19	$-	$80,000.00	$157,522.87	$472,934.46
Year 20	$-	$80,000.00	$157,522.87	$550,457.32

Appendix I: Secondary Treatment Hand Calculations

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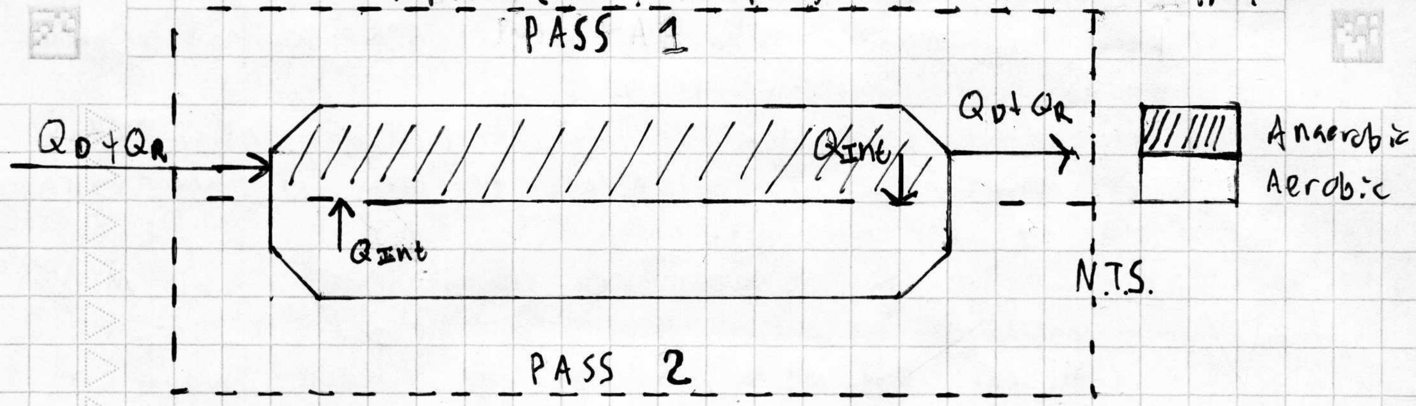
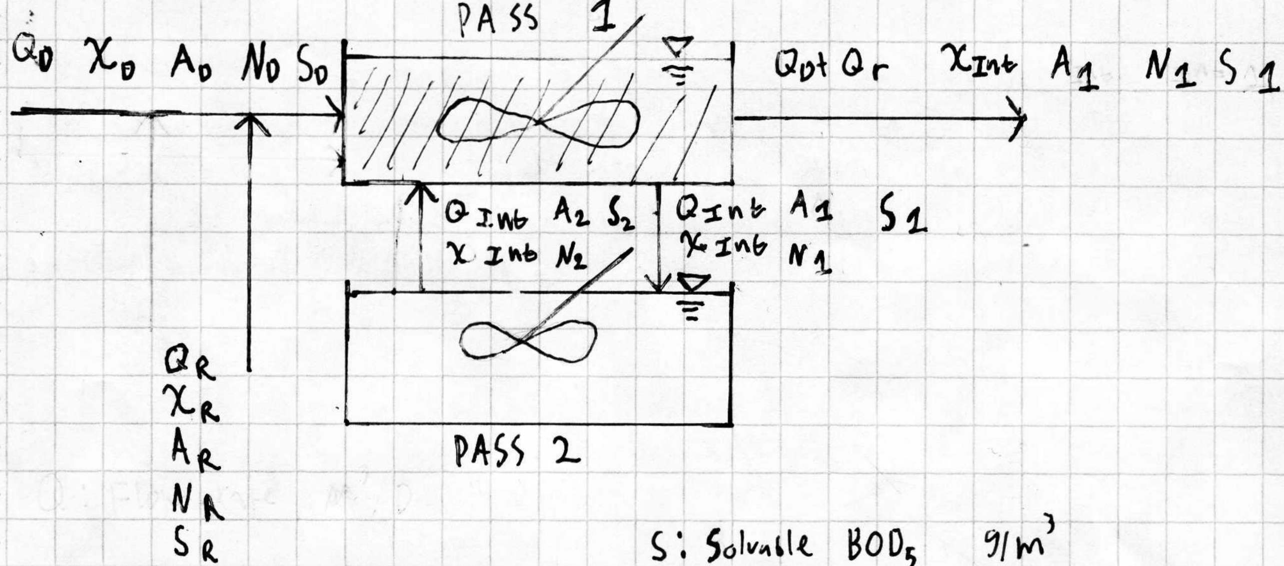
TANK (1/2 PLANT)



Assume each pass operates like a CSTR.



S : Soluble BOD_5 g/m^3

Q : Flow Rate (m^3/D)

A : Ammonia concentration (g/m^3)

X : TSS concentration (g/m^3)

N : Nitrate concentration (g/m^3)

X' : VSS concentration (g/m^3)

$\forall$: Volume (m^3)

θ_r : Hydraulic Retention Time (d)

θ_x : Solids Retention Time (d)

μ_m : Maximum Specific Growth Rate (d^{-1})

Y : Yield (g VSS / g substrate)

K_s : Half Saturation Constant (g/m^3)

k_d : Endogenous Decay Coefficient (g/g-d)

v : Velocity (m/s)

Y_{obs} : Observed Yield (g/g)

P_x : Net waste activated sludge produced (Kg/d)

* : From Competition Rules

** : Water and Wastewater Engineering 2nd Edition, Mackenzie L. Davis

*** : Environmental Biotechnology 2nd Edition, Bruce Rittmann & Perry McCarty

**** : Assumption

Assumptions:

- Each pass operates essentially as a CSTR due to the high amount of internal recycle.
- The design temperature is 20°C. Temperatures below this are not anticipated, and higher temperatures are expected to only be more efficient
- Biomass growth is negligible compared to internal MLVSS concentrations resulting from cell recycle.
  ∴ MLVSS and MLSS are assumed constant through the tank.
- MLVSS / MLSS = 0.75 **
- Nitrification is the slowest process, and therefore is the design process
- Nitrification is complete in Pass 2
- All TKN is either Ammonia or rapidly converted to Ammonia
- Aerators move the internal flow at the recommended 0.3 m/s
- VSS is not readily available BOD_5 substrate
- Ammonia in the RAS is Negligible compared to the influent. $A_R = 0$

Common Conversions:

1 MGD = 3785.41 m^3/d
1 ft = 0.3048 m
1 mg/L = 1 g/m^3
1 d = 86400 s

AVERAGE FLOW

Known and Starting Values

$* Q_0 - \frac{(16.3 - 12.8)\ MGD}{2\ plants} + 4\ \frac{MGD}{Plant} \rightarrow 5.75\ MGD$

$\frac{5.75\ MGD}{plant} \cdot \frac{Plant}{2\ tanks} \cdot \frac{3{,}785.41\ m^3/d}{MGD} \rightarrow Q_0 = 10{,}883\ m^3/d$

$* X_0 - 300\ g/m^3$ TSS $* A_0 - 72\ g/m^3\ NH_4-N$ $*** N_0 = 0\ g/m^3\ NO_3-N$

$* S_0 - 300\ g/m^3\ BOD_5$

$Q_{int} - Q_{int} = V_{int} A$ $\quad *depth_{normal} = 11.5\ ft\ \frac{0.3048\ m}{ft} \rightarrow d_n = 3.51m$

$A = 3.51m \times 10.97m$ $\quad *W_{Pass} = 35\ ft\ \frac{0.3048\ m}{ft} \rightarrow W_p = 10.97m$

$A = 38.46\ m^2$

$** V_{int} = 0.3 m/s$

$Q_{int} = \frac{0.3m}{s} \cdot \frac{38.46\ m^2}{1} \cdot \frac{86400s}{d} \rightarrow 996\ 883\ m^3/d$

$** X_{int} = 3000\ g/m^3$ (Low end of typical values for oxidation ditches)

$** X'/X = .75$ $\quad ** Q_r/Q_0 = 0.75$

$**** A_2 = 0\ g/m^3\ NH_4-N$ $\quad **** S_2 = 0\ g/m^3\ BOD_5$ $\quad *** A_R = 0\ g/m^3$

$Q_r - \quad Q_r = \frac{Q_r}{Q_0} Q_0 \quad Q_r = (.75)10\ 883\ m^3/d \quad Q_r = 8162.3\ m^3/d$

Microbial Kinetic Coefficents

Process	BOD_5 Oxidation	Nitrification	Denitrification
Purpose	BOD Removal	$NH_4 \rightarrow NO_3$	$NO_3 \rightarrow N_2$
μ_m	$6\ d^{-1}$	$0.75\ d^{-1}$	$3.12\ d^{-1}$
K_s	$20\ g\ BOD/m^3$	$0.74\ g\ NH_4-N/m^3$	$1.\ g\ BOD/m^3$
Y	0.4 gVSS/gBOD	0.12 gVSS / gNH_4-N	0.26 gVSS / gBOD
k_d	$0.12\ d^{-1}$	$0.08\ d^{-1}$	$0.05\ d^{-1}$
Source	**	**	***
Oxygen	aerobic	aerobic	anaerobic

Values are for 20°C

Nitrification values include conversion of NH_4 to NO_2 and NO_2 to NO_3

Determine BOD Required for Denitrification

Total BOD Loading : $TBL = Q_0 S_0$, $TBL = \frac{10{,}883\ m^3}{d} \frac{300\ g}{m^3}$

$TBL = 3{,}264{,}900\ g\,BOD/d$

Total Ammonia Loading : $TAL = Q_0 A_0$, $TAL = \frac{10{,}883\ m^3}{d} \frac{72\ g}{m^3}$

$TAL = 783{,}576\ g\ NH_4\text{-}N/d$

Total Nitrate Loading : $TNL = TAL$ with complete Nitrification and no influent Nitrate

$TNL = 783{,}576\ g\ NO_3\text{-}N/d$

g BOD / g NO_3-N : (equation 22-34 **)

$\frac{g\ BOD}{g\ NO_3\text{-}N} = \frac{2.86}{1-1.42\,Y} \rightarrow \frac{2.86}{1-1.42\ 0.26\ \frac{g\,VSS}{g\,BOD}}$

$\frac{g\ BOD}{g\ NO_3\text{-}N} = \frac{4.53\ g\,BOD}{g\ NO_3\text{-}N}$

BOD required for denitrification: $TNL \times \frac{g\ BOD}{g\ NO_3\text{-}N}$

$\frac{783{,}576\ g\ NO_3\text{-}N}{d} \frac{4.53\ g\ BOD}{g\ NO_3\text{-}N} \rightarrow 3{,}552{,}674\ g\ BOD/d$

Effluent NO_3-N Concentration :

$N_1 = (\text{BOD required for Denit} - TBL) \frac{1\ g\ NO_3\text{-}N}{4.53\ g\ BOD} \Big/ Q_0$

$$\frac{(3{,}552{,}674\ g\,BOD\text{-}d - 3{,}264{,}900\ g\ BOD/d)\ \frac{1\ g\ NO_3\text{-}N}{4.53\ g\ BOD}}{10\,883\ m^3/d}$$

$N_1 = 5.83\ g/m^3 \rightarrow 5.83\ mg\ NO_3\text{-}N/L$

Denitrification must go first so that there is enough BOD to denitrify down to 5.83 mg NO_3-N/L.

Ammonia Concentration in Pass 1 (A1)

$$A_1 = \frac{A_D Q_D + A_R Q_R + A_2 Q_{Int}}{Q_D + Q_R + Q_{Int}} \qquad A_R = 0 \quad A_2 = 0$$

$$A_1 = \frac{A_D Q_D}{Q_D + Q_R + Q_{Int}} \qquad A_1 = \frac{72\ g/m^3 \ \ 10{,}883\ \frac{m^3}{d}}{(10\,883 + 8162.3 + 996\,883.3)\,m^3/d}$$

$A_1 = 0.771\ g\ NH_4\text{-}N/m^3$

Total Nitrogen in Effluent

$TN = N + A \qquad TN = 5.83\ mg\ NO_3\text{-}N/L + 0.77\ mg\ NH_4\text{-}N/L$

$TN = 6.60\ mg\ N/L$

permit level $< 8\ mg\ N/L$

TN okay

Hydraulic Retension Times

$\theta_{Tank} = \frac{V}{Q_D + Q_R} \qquad V_{tank} = \text{Area}_{surface} \times d_{normal} \qquad * A_{surface} = 21{,}764\ ft^2$

$V_{tank} = 21\,764\ ft^2 \times 11.5\ ft \rightarrow 250{,}286\ ft^3$

$\theta_{Tank} = \frac{7087\ m^3}{(10{,}883 + 8162)\frac{m^3}{d}} \qquad V_{tank} = 250{,}286\ ft^3 \ \frac{(0.3048\ m)^3}{ft^3} \rightarrow 7087.3\ m^3$

$V_{pass} = \frac{V_{Tank}}{2} \rightarrow V_{pass} = \frac{7087.3\ m^3}{2} \rightarrow V_{pass} = 3544\ m^3$

$\theta_{tank} = 0.372\ d$

$\theta_{pass} = \frac{V_{pass}}{Q_{int}} \qquad \theta_{pass} = \frac{3544\ m^3}{996\,883\ m^3/d} \rightarrow \theta_{pass} = 0.00356\ d$

MLVSS

$X' = \frac{X'}{X} X \qquad X' = 0.75 \ \ 3000\ g\ TSS/m^3 \rightarrow X' = 2250\ g\ VSS/m^3$

Fraction of MLVSS Nitrifying

$$f_n = \frac{0.16\,(NH_4 \text{ removed})}{0.6\,(BOD \text{ removed}) + 0.16\,(NH_4 \text{ removed})} \qquad \text{(Equation 22-29 **)}$$

$$f_n = \frac{0.16\ (72\ g/m^3)}{0.6\ (300\ g/m^3) + 0.16\ (72\ g/m^3)} \qquad f_n = 0.06$$

$X'_n = 2250 \cdot 0.06 \rightarrow X'_n = 135.3\ g/m^3\ VSS_{nit}$

Specific Substrate Utilization Rate, Ammonia

$U = \frac{S_0 - S}{\theta X}$ (Equation 23-18 **)

$U_A = \frac{A_1 - A_2}{\theta_{pass} X_n}$ $\quad U_A = \frac{(0.771\ g/m^3 - 0\ g/m^3)\ NH_4\text{-}N}{0.00356 d * 135.3\ g/m^3\ VSS}$

$U_A = 1.60 \frac{g\ NH_4\text{-}N}{g\ VSS - d}$

Solids Retention Time Required for Nitrification

$\frac{1}{\theta_{X\ min}} = YU - k_d$ (Equation 23-19 **)

$\theta_{X\ min} = \frac{1}{YU - k_d} \rightarrow \theta_{X\ min} = \frac{1}{0.12 \frac{g\ VSS}{g\ NH_4\text{-}N} 1.60 \frac{g\ NH_4\text{-}N}{g\ VSS\text{-}d} - 0.08}$

$\theta_{X\ min} = 8.93\ d$

Assumed safety factor: 2.5 **

$\theta_X = \theta_{X\ min} \cdot SF$ $\quad \theta_X = 8.93d \cdot 2.5 \rightarrow \theta_X = 22.3\ d$

Return and Waste Activated Sludge

$\theta_X = \frac{V_{tank} X_{int}}{Q_w X_r + (Q_D - Q_w) X_e}$ (Equation 23-13 **)

$X_e = 10\ g\ TSS/m^3$ *

Balancing X into the tank assuming constant MLSS

$X_{in} = X_{out}$ $\quad X_{in} = X_D Q_D + X_r Q_r$

$X_{out} = X_{int}(Q_D + Q_r)$

$X_D Q_D + X_r Q_r = X_{int}(Q_D + Q_r)$

$X_r = \frac{X_{int}(Q_D + Q_r) - X_D Q_D}{Q_r}$

$X_r = \frac{3000 \frac{g}{m^3} (10,883 + 8162.3) \frac{m^3}{d} - 300 \frac{g}{m^3} 10,883 \frac{m^3}{d}}{8162.3 \frac{m^3}{d}}$

$X_r = 6600\ g/m^3$

Plugging in X_r into equation 23-13 ** and using Excel solver:

$Q_w = 128.5\ m^3/d$

Oxygen Requirement

Assuming soluble BOD is used for denitrification but VSS could still be consumed aerobically:

$$Y_{obs} = \frac{Y}{1 + k_d \theta_x}$$ (Equation 23-37 **)

$$Y_{obs} = \frac{0.4 \frac{g}{g}}{1 + 0.12 d^{-1} \cdot 22.3 d} \rightarrow Y_{obs} = 0.109 \frac{g}{g}$$

$$P_x = Y_{obs} Q (S_0 - S) \frac{kg}{1000 g}$$ (Equation 23-38 **)

Assuming influent VSS is being consumed completely:

$$P_x = 0.109 \frac{g}{g} \, 10{,}883 \frac{m^3}{d} \left(300 \frac{g}{m^3} \times .75 - 0\right) \frac{1 kg}{1000 g}$$

$$P_x = 266.5 \text{ kg/d}$$

$$M_{O_2} = Q_D (S_0 - S) \frac{kg}{1000 g} - 1.42 P_x + 4.33 Q_D A_D$$ (Equation 23-44 **)

$$M_{O_2} = 10{,}883 \frac{m^3}{d} \, 225 \frac{g}{m^3} \frac{kg}{1000 g} - 1.42 \cdot 266.5 \frac{kg}{d} + 4.33 \cdot 10{,}883 \frac{m^3}{d} \, 72 \frac{g}{m^3} \frac{kg}{1000 g}$$

$$M_{O_2} = \frac{5463 \text{ kg } O_2}{d}$$

Oxygen Supply

$\frac{8.2 \text{ kg } O_2}{h \cdot m}$ for 9 m rotors ** (Brush aerator)

Supply per rotor

$$\frac{8.2 \text{ kg } O_2}{h \cdot m} \; \frac{24 hr}{d} \; \frac{9 m}{rotor} \rightarrow \frac{1771.2 \text{ kg } O_2}{d - rotor}$$

Required # of Rotors

$$\frac{5463 \text{ kg } O_2}{d} \; \frac{d - rotor}{1771.2 \text{ kg } O_2} \rightarrow 3.08 \text{ Rotors}$$

Round up → 4 rotors

Total Oxygen supply

$$\frac{1771.2 \text{ kg } O_2}{d - rotor} \; \frac{4 \text{ rotors}}{} \rightarrow \frac{7085 \text{ kg } O_2}{d}$$

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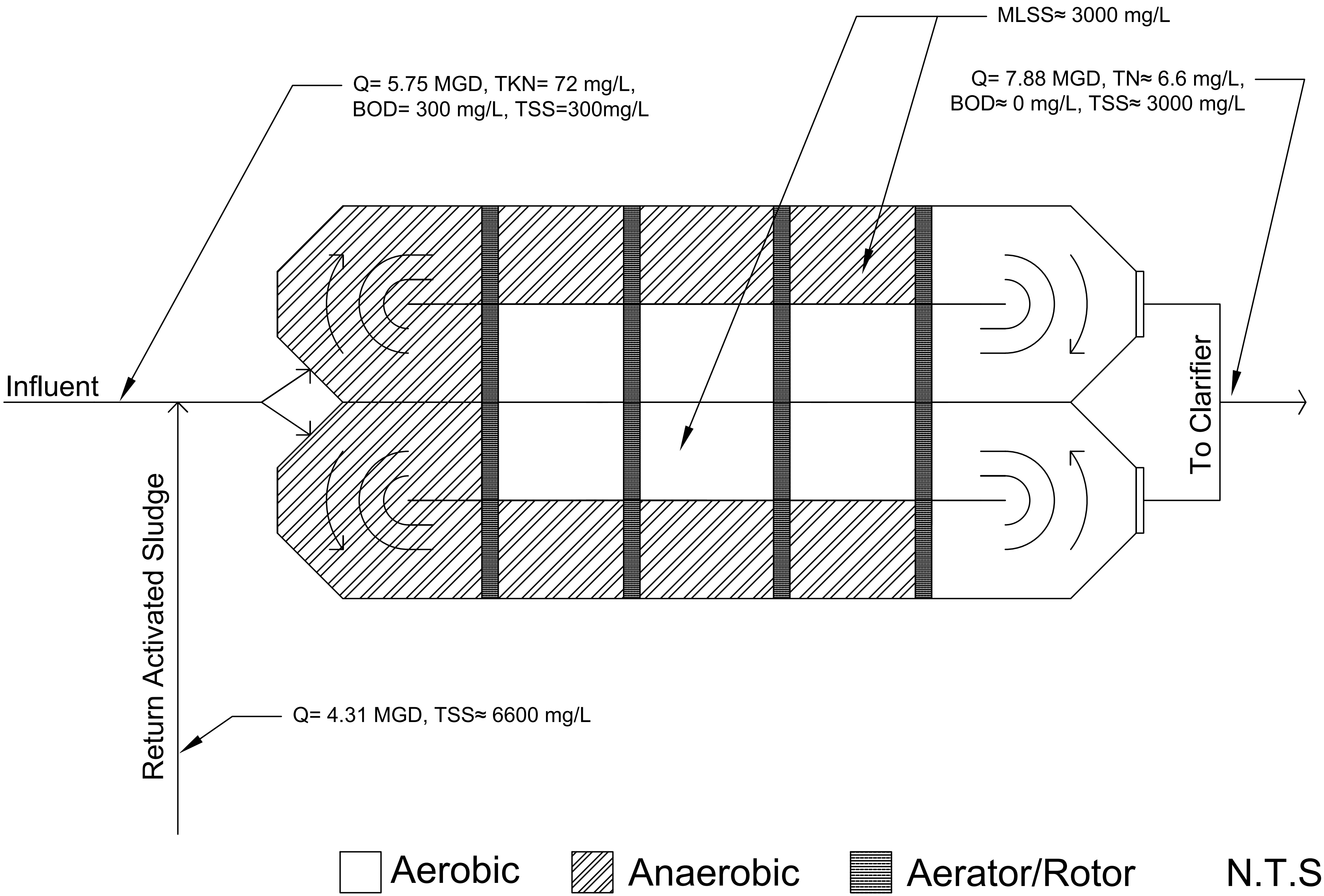

Company: WALNUT CANYON WASTEWATER

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Recommended Operational Parameters for Plants IV and V

PAGE 1 of 1

Appendix K: Parameters and Intermediate Values for Secondary Clarifier

Table J-1: Parameters and Intermediate Values for Secondary Clarifiers

Parameters and Intermediate Values	
$MLSS_{in}$ (g/m^3)	3000.00
A (ft^2)	12272.00
Side Water Depth (ft)	18.00
Q (MGD)	5.75
Q_R (MGD)	4.31
Q_{total} (MGD)	10.06
Peak Q (MGD)	14.38
Peak Q_R (MGD)	10.78
Peak Q_{total} (MGD)	25.16
v_0 (m/h)	1.39
SOF (kg/m2*h)	4.18
Peak v_0 (m/h)	3.48
Peak SOF (kg/m2*h)	10.44
HRT (hr)	3.94

Appendix L: Acceptable Parameters for Secondary Clarifiers

TABLE 25-3

Preferred secondary clarifier overflow rates for activated sludge processes

Flow rate	Average flow overflow rates, m/h	Peak flow overflow rate, m/h	Comments
Conventional activated sludge	0.68–1.2	1.7–2.7	
Extended aeration	0.33–0.68	1.0–1.3	
Oxidation ditch	0.51–0.68	< 1.7	
Biological nutrient removal (BNR)	0.68–1.2	1.7–2.7	
Biological phosphorus removal			
Total P = 2 mg/L	1.0–1.3		
Total P = 1 mg/L	0.67–1.0		Occasional chemical addition
Total P = 0.2–0.5 mg/L	0.50–0.83		Continuous chemical addition

Adapted from Metcalf & Eddy, 2003, and Lakeside Equipment Corporation.

Figure K-12-1: Preferred Secondary Clarifier Overflow Rates [15]

TABLE 25-5

Ranges of loading rate for activated sludge process secondary clarifiers

Flow rate	Average solids loading rates, $kg/m^2 \cdot h$	Peak solids loading rates, $kg/m^2 \cdot h$
Conventional activated sludge	4–6	8
Extended aeration	1.0–5	7
Oxidation ditch		< 12
Biological nutrient removal	5–8	9

Adapted from Metcalf & Eddy, 2003, and Lakeside Equipment Corporation.

Figure K-12-2: Preferred Secondary Clarifier Solids Overflow Rates [15]

Appendix M: Anaerobic Digestion Assumptions

Parameter	Unit	Value: Range	Value: Typical
Solids yield, Y			
Fermentation	g VSS/g COD	0.06–0.12	0.10
Methanogenesis	g VSS/g COD	0.02–0.06	0.04
Overall combined	g VSS/g COD	0.05–0.10	0.08
Decay coefficient, k_d			
Fermentation	g/g · d	0.02–0.06	0.04
Methanogenesis	g/g · d	0.01–0.04	0.02
Overall combined	g/g · d	0.02–0.04	0.03
Maximum specific growth rate, μ_m			
35°C	g/g · d	0.30–0.38	0.35
30°C	g/g · d	0.22–0.28	0.25
25°C	g/g · d	0.18–0.24	0.20
Half-velocity constant, K_s			
35°C	mg/L	60–200	160
30°C	mg/L	300–500	360
25°C	mg/L	800–1100	900
Solids retention time (SRT)			
35°C	d	10–20[a]	15
30°C	d	15–30[a]	N/A
24°C	d	20–40[a]	N/A
Methane			
Production at 35°C	m^3/kg COD	0.4	0.4
Density at 35°C	kg/m^3	0.6346	0.6346
Content of gas	%	60–70	65
Energy content	kJ/g	50.1	50.1

[a]Safety factors range from 2.5 to 5 times minimum SRT.

Adapted from Metcalf & Eddy, 2003.

Figure L-1: Supporting Details for Anaerobic Digestion Assumptions [15]

Appendix N: Schematic of Anerobic Digesters

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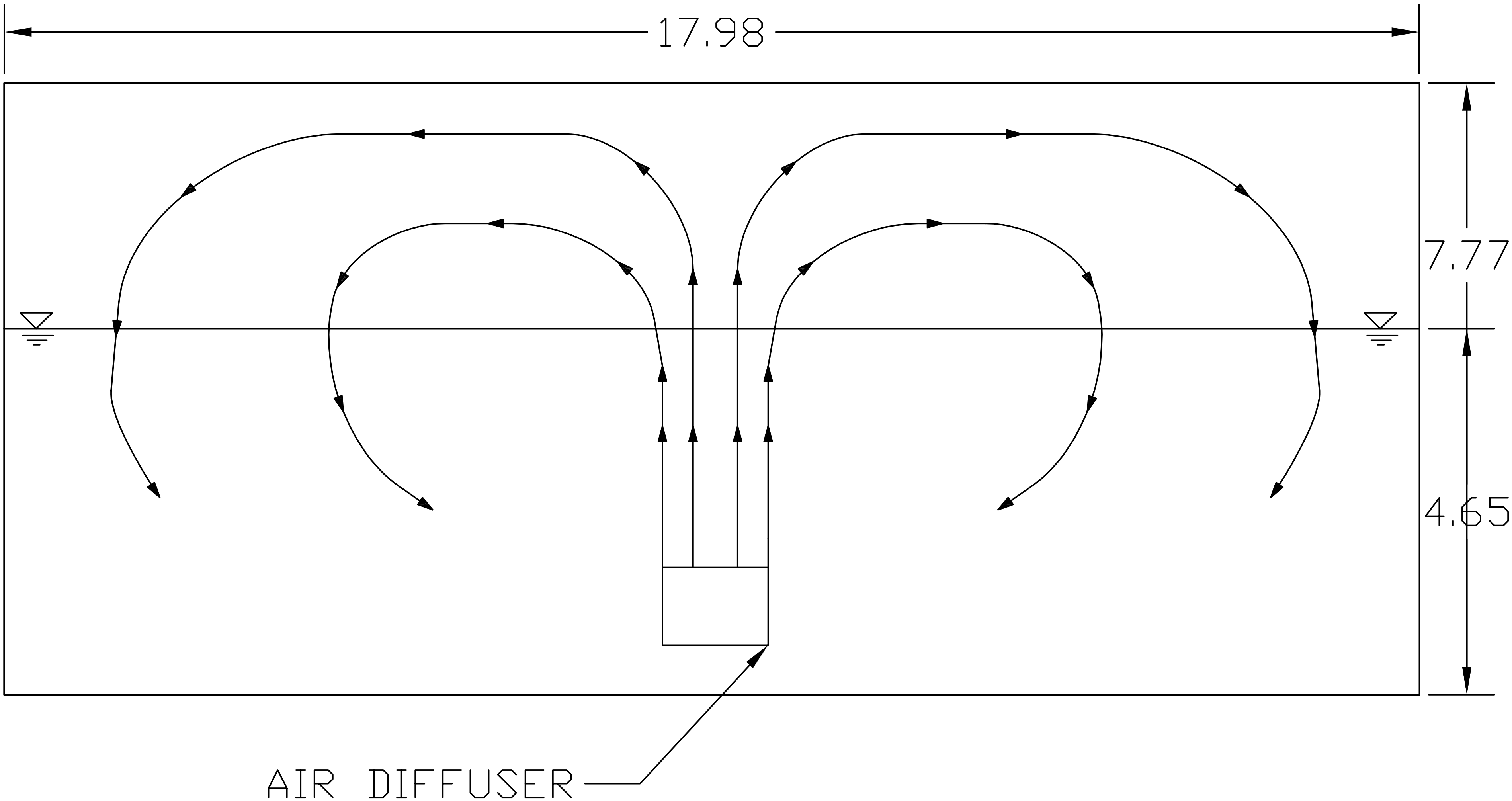

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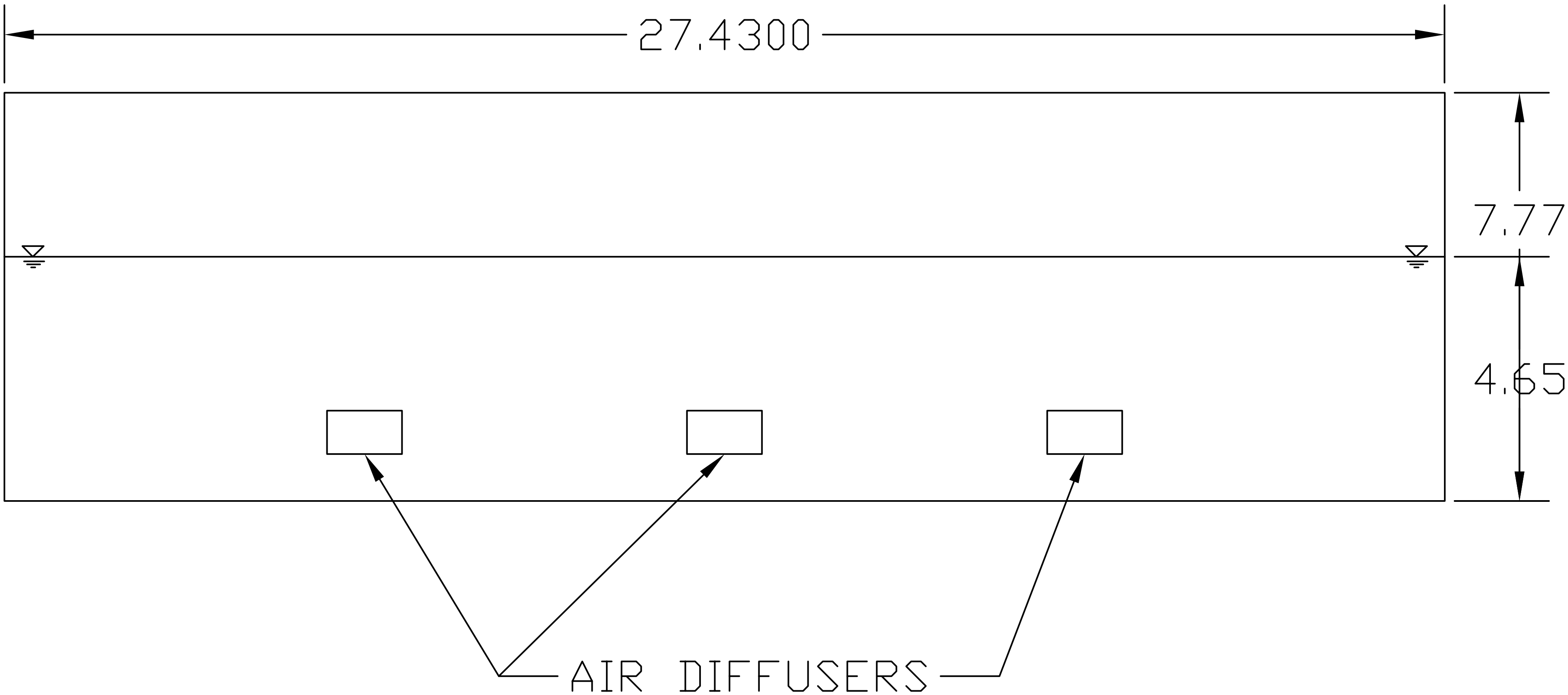
#1	TW	3/20/2025	FRONT VIEW

SCHEMATIC OF ANAEROBIC DIGESTER



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Drawn By: TW

Checked By: AL

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NO.	BY	DATE	DESCRIPTION
#1	TW	3/20/2025	SIDE VIEW

SCHEMATIC OF ANAEROBIC DIGESTER

Appendix O: Heat Exchanger Design Assumptions

Structural composition	U, $W/m^2 \cdot K$
Concrete walls above ground	
300 mm thick, not insulated	4.7–5.1
300 mm thick with air space and brick facing	1.8–2.4
300 mm thick with insulation	0.6–0.8
Concrete walls below ground	
Surrounded by dry soil	0.57–0.68
Surrounded by moist soil	1.1–1.4
Concrete floor	
300 mm thick in contact with dry soil	1.7
300 mm thick in contact with moist soil	2.85
Floating cover	
35 mm wood deck, built-up roofing, no insulation	1.8–2.0
25 mm insulating board installed under roofing	0.9–1.0
Fixed concrete cover	
100 mm thick, built-up roofing, not insulated	4.0–5.0
100 mm thick, built-up roofing, 25 mm of insulation	1.2–1.6
225 mm thick, not insulated	3.0–3.6
Steel cover	
6 mm thick	4.0–5.4

Source: Metcalf & Eddy, 2003.

Appendix P: Hydraulics Calculations

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Summary of Head Loss:

Preliminary:	0.67 ft
trough loss between preliminary and ox ditch:	0.54 ft
Secondary:	ft
head loss from pump station to advanced treatment:	0.32 ft
Advanced:	15.28 ft
Disinfection:	ft

Preliminary:

$$H_L = \left(\frac{1}{2g}\right)\left(\frac{Q}{CA}\right)^{1/2}$$

Q:	10.09667	ft^3/s	16.3 MGD to ft^3/s (divided by 3 for 3 screens in parallel)
g:	32.2	ft^2/s	
C:	0.6		
A:	12.25	ft^2	35% open area, 35ft^2 total area

HL: 0.222944 ft *for one band/fine screen

HEAD LOSS FOR 3 SCREENS IN PARALLEL: 0.668832 ft

Trough Loss:

Preliminary to Plant 3:			solo trought to Plant 3:			
L:	213	ft	L:	66	ft	(length from google maps)
Q:	63.05	(ft^3)/s	Q:	18.57	(ft^3)/s	peak 40.75MGD flow, then just the capacity of plant 3)
n:	0.013		n:	0.013		(concrete, trowel finish)
b:	8	ft	b:	8	ft	
y:	2	ft	y:	2	ft	
R:	1.333	ft	R:	1.333	ft	
A:	16	ft^2	A:	16	ft^2	
HL:	0.2093	ft	HL:	0.0191	ft	

Plant 3 to Plant 4			solo trought to Plant 3:			
L:	309	ft	L:	45	ft	(length from bluebeam takeoff)
Q:	44.48	(ft^3)/s	Q:	22.24	(ft^3)/s	(remaining flow, then split flow with plant 4 and 5)
n:	0.013		n:	0.013		(concrete, trowel finish)
b:	8	ft	b:	8	ft	
y:	2	ft	y:	2	ft	
R:	1.333	ft	R:	1.333	ft	
A:	16	ft^2	A:	16	ft^2	
HL:	0.2142	ft	HL:	0.0156	ft	

Plant 4 to Plant 5:			
L:	226	ft	(length from bluebeam takeoff)
Q:	22.24	(ft^3)/s	(split flow from plant 4 and 5)
n:	0.013		(concrete, trowel finish)
b:	8	ft	
y:	2	ft	
R:	1.333	ft	
A:	16	ft^2	
HL:	0.0783	ft	

Manning's Equation

$$S = \left(\frac{Q * n}{1.49 * A * R^{\frac{2}{3}}} \right)^2$$

$$R_h = \frac{A}{WP} = \frac{b \cdot y}{b + 2y}$$

S is the slope of the energy grade line and $S = h_f/L$ where h_f is energy (head) loss and L is the length

Head Loss from pump station to advanced treatment:

k:	1.318					
Q:	44.48	(ft^3)/s	(peak flow of 28.75MGD for plants 4 and 5)			
pipe diameter:	2.5	ft				
A:	4.90625	ft^2	new, unlined DIP			
C:	130			current velocity:	9.065987	ft/s
Rh:	0.625	ft		max velocity:	10	(from EPA)
L:	1080	ft				
head loss:	0.31900	ft				

*assume full pipe flow

Hazen-Williams Equation:

$$S = \frac{h_f}{L} = \left[\frac{Q}{A\,K\,C\,R_H^{0.63}}\right]^{1/0.54}$$

FOR FULL PIPE

$$A = \frac{\pi D^2}{4} \qquad R_H = \frac{D}{4}$$

Sources: Cruise et al., (2007); Velon and Johnson (1993); Wurbs and James (2002).

Disinfection:	(12MG capacity for one basin)				
bottom elev of basin:	1114.5	ft			
avg water level:	1119.5	ft			
high water level:	1120.5	ft	basin 1:	12	million gallons
****this gives 8ft of freeboard (from high water level)			basin 2:	6.82	million gallons
basin inlet pipe dia:	48	in	total capacity:	18.82	million gallons
basin outlet pipe dia:	48	in			

*data obtained from AZWA SDC documents

RAS Pipes Analysis:

RAS Pipes Analysis:

4.31 MGD RAS average flow:	6.67 cfs	10.775 MGD RAS peak flow:	16.63 cfs
16" pipe		16" pipe	
area of pipe:	1.40 ft^2	area of pipe:	1.40 ft^2
actual velocity (average flow):	4.78 ft/s	actual velocity (peak flow):	11.92 ft/s

*exceeds maximum threshold of 10 ft/s per EPA

Pipe: Secondary to Disk Filters:

(11.5 MGD) Average Flow Rate, Q:	17.79 cfs	(28.75 MGD) Peak Flow Rate, Q:	44.48 cfs
Diameter of Pipe:	30 in	Diameter of Pipe:	30 in
area of pipe:	4.91 ft^2	area of pipe:	4.91 ft^2
max velocity 10ft/s per EPA		*max velocity 10ft/s per EPA*	
actual velocity (average flow):	3.63 ft/s	(peak flow):	9.07 ft/s

Pipe: Disk Filters to Basins:

Average Flow Rate, Q:	25.22 cfs	Peak Flow Rate, Q:	63.05 cfs
Diameter of Pipe:	48 in	Diameter of Pipe:	48 in
area of pipe:	12.56 ft^2	area of pipe:	12.56 ft^2
max velocity 10ft/s per EPA		*max velocity 10ft/s per EPA*	
actual velocity (average flow):	2.01 ft/s	actual velocity (peak flow):	5.02 ft/s

Plant 4 and 5 Avg. RAS Flow : 8.62 MGD

Plant 4 and 5 RAS Pumps:
5 Pumps @ 1400 gpm each

8.62 MGD = 5986.11 gpm

5986.11 gpm ÷ 5 pumps = 1,197.22 gpm/pump

1,197.22 gpm < 1400 gpm capacity

Plant 4 and 5 RAS pumps ok ✓

RAS Pumps for peak flow:

8.62 MGD × 2.5 peaking factor = 21.55 MGD

21.55 MGD = 14,965.28 gpm

14,965.28 gpm ÷ 5 gpm = 2993.06 gpm/pump

2993.06 gpm > 1400 gpm capacity

RAS pumps NOT OK for peak flow

Appendix Q: Existing Site Layout

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SPA 1 Existing Site Layout

Created by: Andrew Shipley, Walnut Canyon Wastewater
Date:3/17/25

0 125 250 500 750 1,000 US Feet

Legend

Site Boundary
Infrastructure

Spatial Reference
Name: NAD 1983 StatePlane Arizona Central FIPS 0202 Feet
PCS: NAD 1983 StatePlane Arizona Central FIPS 0202 Feet
GCS: GCS North American 1983
Datum: North American 1983
Projection: Transverse Mercator

Central Meridian: -111.9167
Latitude of Origin: 31.0000
Longitude of Origin: 0.0000
False Easting: 699,998.6000
False Northing: 0.0000
Central Parallel: 0.0000
Map Units: Foot US
Authority: Esri

Appendix R: Proposed Site Layout

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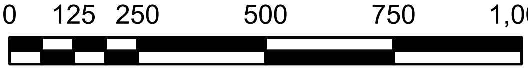
SPA 1 Proposed Site Layout

Created by: Andrew Shipley, Walnut Canyon Wastewater
Draft: Final
Date:3/17/25

Spatial Reference
Name: NAD 1983 StatePlane Arizona Central FIPS 0202 Feet
PCS: NAD 1983 StatePlane Arizona Central FIPS 0202 Feet
GCS: GCS North American 1983
Datum: North American 1983
Projection: Transverse Mercator

Central Meridian: -111.9167
Latitude of Origin: 31.0000
Longitude of Origin: 0.0000
False Easting: 699,998.6000
False Northing: 0.0000
Central Parallel: 0.0000
Map Units: Foot US
Authority: Esri



Legend

- Site Boundary
- Clarifier
- Decommissioned
- Headworks With New Band Screens
- New Conventional Operation Style
- Retrofitted Anaerobic Digesters
- Unchanged

WALNUT CANYON WASTEWATER

Appendix S: Proposed Process Flow Diagram

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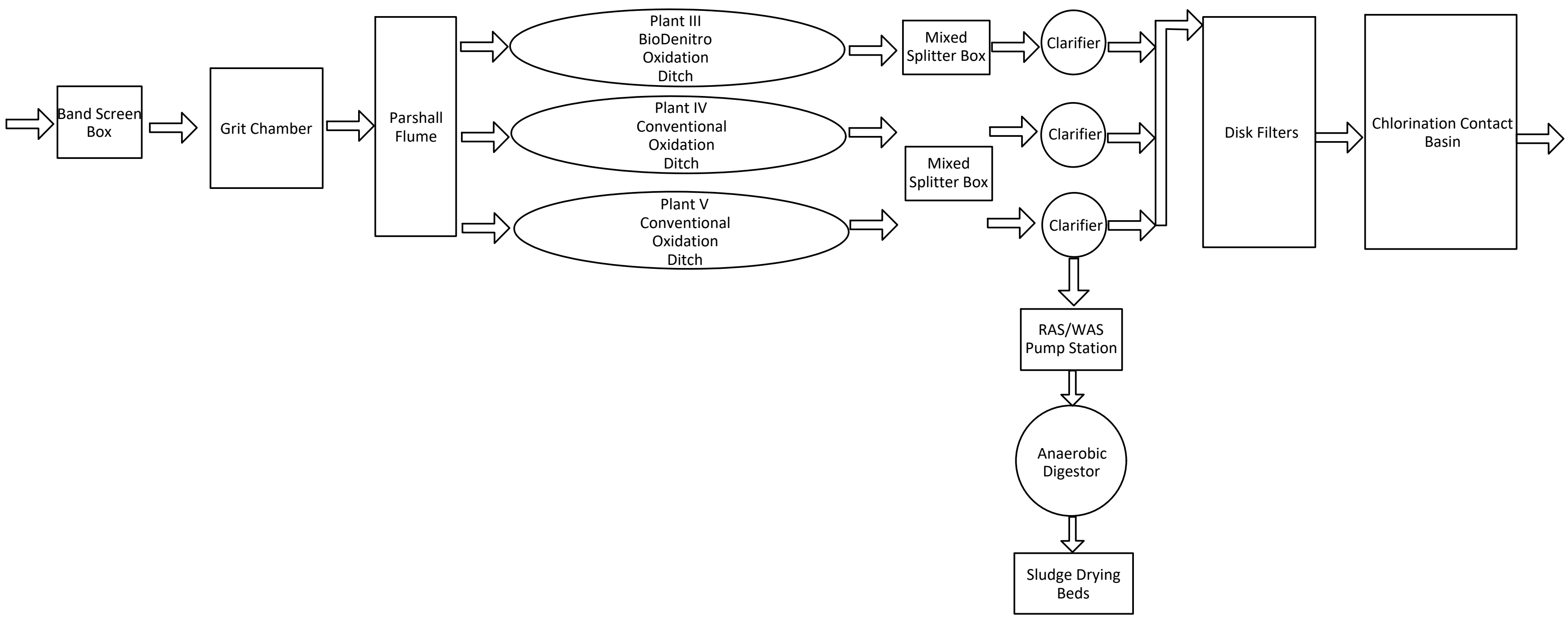

Company: Walnut Canyon Wastewater

Drawn By: OL

Checked By: AL

REVISIONS

NO.	BY	DATE	DESCRIPTION

PROPOSED FLOW DIAGRAM

PAGE 1 of 1

Appendix T: Proposed Hydraulic Profile

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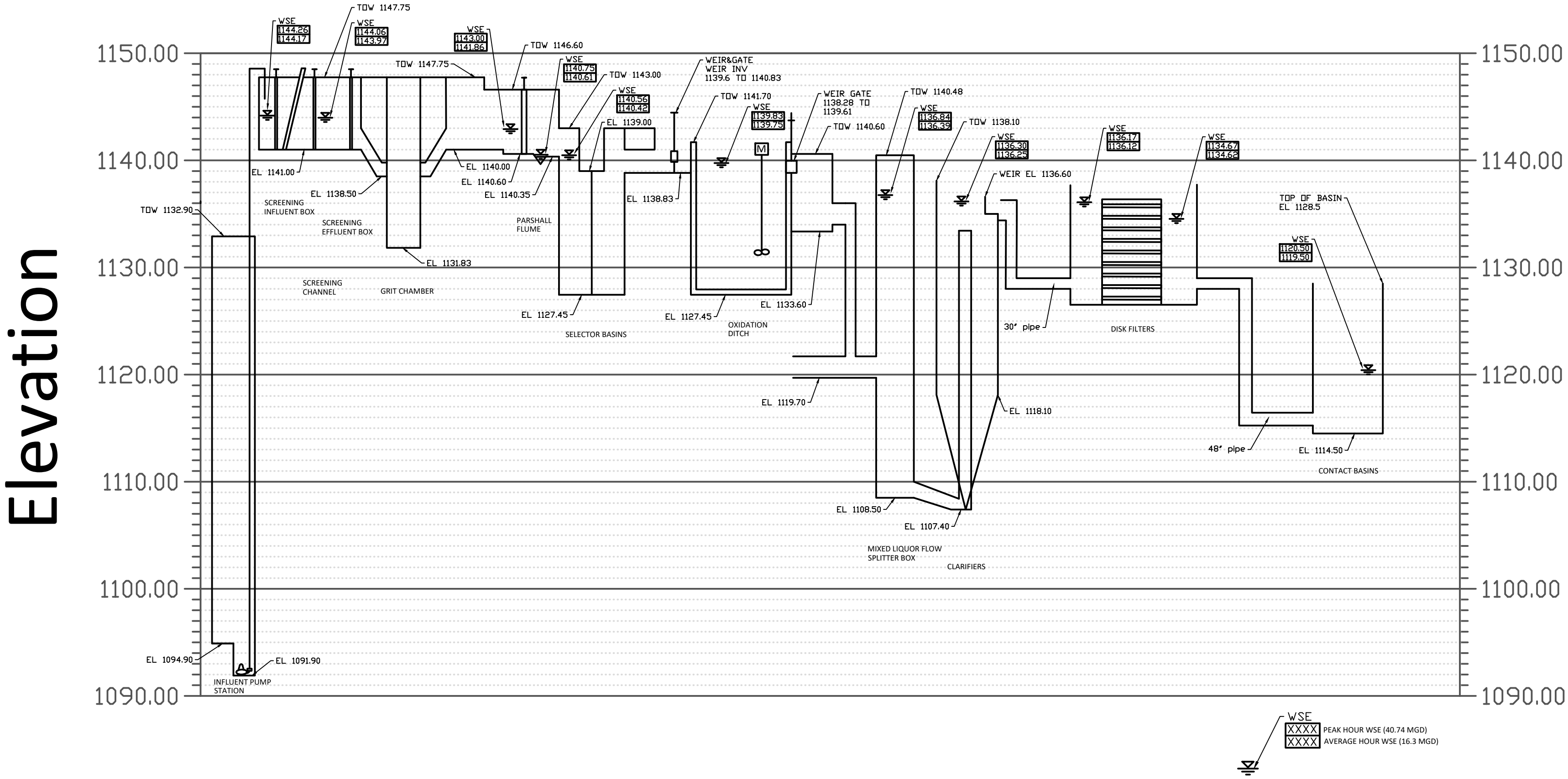

	REVISIONS			
Company: WALNUT CANYON WASTEWATER	NO.	BY	DATE	DESCRIPTION
Drawn By: AL				
Checked By: OL				

PROPOSED HYDRAULIC PROFILE

PAGE 1 of 1

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Engineers' Opinion of Probable Construction Cost (EOPCC)

Project: Expansion of SPA 1 Water Reclamation Facility for the City of Surprise

Description	Quantity	Unit	Unit Price	Amount
Preliminary Treatment				
Bandscreen, 3mm perforations	3	EA	$ 150,000	$ 450,000
Subtotal				$ 450,000
Secondary Treatment				
Convert to conventional treatment style	1	LS	$ 5,000	$ 5,000
Subtotal				$ 5,000
Solids Handling				
Converting to aerobic digestors along with adding improvments of scraped surfuce heat exchanger, gas collection system, and air scrubber	1	LS	$ 1,000,000	$ 1,000,000
Subtotal				$ 1,000,000

Grand Total: $ 1,455,000

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Estimate of Annual Operations and Maintenance Costs

Project: Expansion of SPA 1 Water Reclamation Facility for the City of Surprise

Description	Quantity	Unit		Unit Price		Amount
Preliminary Treatment						
Annual Energy Cost	22,521	kW-hr	$	0.15	$	3,378
Annual Inspection and Maintenance Cost	208	hr	$	25	$	5,200
Subtotal					$	8,578
Secondary Treatment						
Annual Energy Cost	7,864,336	kW-hr	$	0.15	$	1,179,650
Brush Aerators	32	year	$	60,000	$	1,920,000
Subtotal					$	3,099,650
Advanced Treatment						
Annual Operation Cost	1	LS	$	31,632	$	31,632
Replacement of Filter (every 5 years)	0.2	5-years	$	33,471	$	6,694
Subtotal					$	38,326
Disinfection						
Annual Operation Cost	1	LS	$	63,280	$	63,280
Subtotal					$	63,280
Solids Handling						
Cost of Operating Digester, Scraped Surface	1	LS	$	80,000	$	80,000
Subtotal					$	80,000
Hydraulics						
Annual Energy Cost of RAS/WAS Pumps	1,911,000	kW-hr	$	0.15	$	286,650
Annual Energy Cost of Influent Pumps	1	LS	$	300,000	$	300,000
Subtotal					$	586,650
Additional						
Operator 1	3	operators	$	52,000	$	156,000
Operator 2	3	operators	$	62,400	$	187,200
Operator 3	2	operators	$	72,800	$	145,600
Subtotal Subtotal					$	343,200
		Grand Total:			**$**	**4,219,685**

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Band Screen Energy Cost

\$0.15 per kW-hr

1.0 Watt per m^3 of wastewater (Huber Technology, 2025)

$$16,300,000 \text{ gal} \times \frac{1 \, m^3}{264.17 \text{ MG}} = 61,702.21 \, m^3$$

$$61,702.21 \, m^3 \times 1 \text{ watt}/m^3 = 61,702.21 \text{ watt per day}$$

$$61,702.21 \text{ watt/day} = 61.7 \text{ kW/day}$$

$$61.7 \text{ kW/day} \times 365 \text{ days} = 22,520.5 \text{ kW/year}$$

$$22,520.5 \text{ kW/year} \times \$0.15/\text{kW} = \boxed{\$3,378.08/\text{year}}$$

Rotor

Plant IV $75 \frac{HP}{rotor} \left(16 rotor\right)\left(\frac{70 min}{cycle}\right)\left(\frac{6.5 cycle}{1 d}\right)\left(\frac{365 d}{1 yr}\right)\left(1 yr\right)\left(\frac{1 hr}{60 min}\right) = 3{,}321{,}500$ HP-hr

$3{,}321{,}500$ HP-hr $\left(\frac{0.7456 \text{ Kwh}}{1 \text{ HP-hr}}\right) = 2{,}476{,}510.4$ Kwh

$2{,}476{,}510.4$ kWh $\left(\frac{\$.15}{\text{Kwh}}\right) = \$371{,}476.56$

Plant V 16 rotors at 75 HP/rotor

$371,476.56

Mixers

Plant IV $9 \frac{HP}{\text{all mixers}} \left(\text{all mixers}\right)\left(\frac{75 min}{cycle}\right)\left(\frac{6.5 cycle}{d}\right)\left(\frac{365 d}{1 yr}\right)\left(1 yr\right)\left(\frac{1 hr}{60 min}\right) = 26{,}690$ HP-hr

$26{,}690$ HP-hr $\left(\frac{0.7456 \text{ Kwh}}{1 \text{ HP-hr}}\right) = 19{,}900.53$ kwh

$19{,}900.53$ kwh $\left(\frac{\$.15}{\text{kwh}}\right) = \$2{,}985.07$

Plant V 4 Mixers at 9HP as well

$2,985.07

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Estimate of Annual Operations and Maintenance Costs

Project: Expansion of SPA 1 Water Reclamation Facility for the City of Surprise

Description	Quantity	Unit	Unit Price	Amount
Preliminary Treatment				
Annual Energy Cost	22,521	kW-hr	$ 0.15	$ 3,378
Annual Inspection and Maintenance Cost	208	hr	$ 25	$ 5,200
Subtotal				$ 8,578
Secondary Treatment				
Annual Energy Cost	7,864,336	HP	$ 0.15	$ 1,179,650
Brush Aerators	16	5-years	$ 60,000	$ 960,000
Subtotal				$ 2,139,650
Advanced Treatment				
Annual Operation Cost	1	LS	$ 31,632	$ 31,632
Replacement of Filter (every 5 years)	0.2	5-years	$ 33,471	$ 6,694
Subtotal				$ 38,326
Disinfection				
Annual Operation Cost	1	LS	$ 63,280	$ 63,280
Subtotal				$ 63,280
Solids Handling				
Cost of Operating Digester, Scraped Surface Heat Excahnger, Air Srubber, and Gas Collection System	1	LS	$ 80,000	$ 80,000
Potential Savings from Biomethane Production	1	LS	$ (157,000)	$ (157,000.00)
Subtotal				$ (77,000)
Hydraulics				
Annual Energy Cost of RAS/WAS Pumps	1,911,000	kW-hr	$ 0.15	$ 286,650
Annual Energy Cost of Influent Pumps	1	LS	$ 300,000	$ 300,000
Subtotal				$ 586,650
Additional				
Operator 1	3	operators	$ 52,000	$ 156,000
Operator 2	3	operators	$ 62,400	$ 187,200
Operator 3	2	operators	$ 72,800	$ 145,600
Subtotal				$ 343,200

Grand Total: $ 3,102,685

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Manual of Permitted Operations

Prepared For: Special Planning Area 1 Water Reclamation Facility, City of Surprise

Prepared By: Walnut Canyon Wastewater

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1. Purpose

The purpose of this Manual of Permitted Operations (MOPO) is to ensure that proposed changes to the Special Planning Area 1 (SPA 1) Water Reclamation Facility (WRF) are constructed safely and without interrupting the WRF's ongoing treatment requirements. This MOPO identifies foreseen construction activities, potentially required construction activities, and potentially required maintenance activities that may need to be completed while construction is ongoing. An order in which the required construction should be completed to ensure there are no interruptions is established. Additionally, a matrix showing activities that can and cannot be completed safely in adverse weather/potentially limiting conditions was developed. Finally, a matrix showing which activities can and cannot be safely completed simultaneously was also developed.

2. Defining Safety

For this MOPO, safety includes both workers' personal safety as well as the safety of the facility's ongoing operations. Any condition that endangered either workers' health, wellbeing, or life, or put the facility at risk of not being able to maintain continuous operations was deemed to be unsafe. Both conditions were determined to be impermissible and are not distinguished in subsequent matrices.

3. Construction Sequencing

The ability of the facility to continuously operate and produce effluent within its permit levels is of paramount importance. To ensure this, it is important to ensure that each system maintains as much redundancy as possible during construction.

The replacement of the facility's fine screens with band screens in the headworks should be done one screen at a time to ensure that at least two screens are functioning at all times. The two screen types are made by the same manufacturer and the new band screen systems should fit into the headworks essentially the same as the fine screens, limiting effects on other systems.

For the oxidation ditches, no real construction work is required. The weirs and brush aerators are controlled digitally, these systems will require some reprogramming. When transitioning the oxidation ditches in Plants 4 and 5 it is recommended to transition them one at a time so that there is a minimum of two plants in full operation. Each plant will need a short period of batching to adjust the microbiome to new conditions (approx. 30 min.).

It is recommended that the new band screens be installed in the facility's headworks before the Plant 4 and 5 oxidation ditches are switched to the new operating style. The current fine screens allow rags through, which can damage brush aerators in the oxidation ditches, requiring replacement. By installing the band screens first, it reduces the risk that an oxidation ditch will be taken offline while a plant is being batched to adjust to the new operating style. Changes to the headworks and oxidation ditches should not be made at the same time. The reduction of screens from 3 to 2 increases the likelihood that overflow channels must be used, which only has a bar screen. This increases the likelihood that rags or other objects that can bypass headworks and damage the oxidation ditches. If the facility also has a plant down for batching, the risk that one of the remaining plants is damaged is impermissible.

Many existing systems require ongoing maintenance, and in general it is recommended that required maintenance take priority over recommended construction. For example, if all existing fine screen systems are due to have their fine screens be replaced, at least two systems should have that maintenance done before the replacement of a system with a new band screen system. In this way, when the third screen is removed, there are two additional screens to handle the incoming influent. Additionally, it is recommended that if any brush aerators need replacement, that this be done before the systems are converted to the new operating style. In this way, when the systems are being batched, the other two systems will be available to handle incoming flow until the batching system is ready to handle new flow.

The aerobic digestors are currently unused, and while connected to other systems, currently do not take flow from or provide flow to other systems. This means that the construction required to convert them to anaerobic digestors can coincide with most other maintenance and construction activities.

Finally, any deliveries of large equipment should be planned and scheduled so as not to interfere with the loading and hauling of dried solids offsite to the landfill.

4. Permissible Coinciding Activities and Conditions

A table was created to show which activities are permissible both during adverse weather conditions, and which activities can be completed at the same time. A box labeled in green represents that no additional safety requirements are expected and the two can coincide. A box labeled in yellow represents that the work can be done, but increased caution should be used or additional equipment and safety procedures are required. Finally, a box labeled red means that the work is unsafe to either the plant or worker and the two should not coincide.

The first table shows which activities are and are not recommended during adverse weather conditions and during high influent flow.

The second table shows which construction and maintenance activities are and are not recommended to happen simultaneously.

Adverse Weather and High Influent Flow

Required Construction Activity	High Influent Flow, Greater than 50% Existing Capacity (6.4 MGD)	Night Time Work	Heavy Precipitation	Lightning	Limited Visibility (Dust, Rain, Fog, etc.)	High Winds	High Ambient Temperature (>45 Deg C
Fine Screen Removal							
Band Screen Installation							
Ditch Rotor/Weir Reprograming							
Ditch Batching during Conversion							
Retrofitting of Existing Aerobic Digestors							
Refurbishing Sludge Piping Leading to Aerobic Digestor							
Addition of Gas Collection System for Solids Handling							
Addition of Air Scrubber for Solids Handling							
Addition of Heat Exchanger and Boiler							
Potentially Required Construction Activities							
Excavation							
Working at Heights							
Use of Crane or Other Lifting Apparatus							
Confined Space Entry							
On-Site Vehicle Use, Including for Dried Sludge and Solids Disposal							
Outdoor Concrete Pouring							
Delivery of Large Equipment							
Potential Required Maintenance							
Replacement of Individual Fine screen Parts							
Maintenance on Grit Chamber							
Replacement of Brush Aerators							
Maintenance on RAS/WAS Pumps							
Replacement of Disk Filters							
Maintenance on Chlorine Generation System							
Maintenance on Chlorine Contact Basins							
Maintenance on Centrifuge Sludge Dewaterers							
Maintenance on Solar Drying Facility							
Maintenance on Reclaimed Water Distribution Pumps							

- No Additional Safety Concerns Expected
- Use Extra Caution, Equipment, or Procedures
- Do Not Complete at The Same Time
- N/A or Duplicate

Permissible Coinciding Construction and Maintenance

	Required Construction Activity	Fine Screen Removal	Band Screen Installation	Ditch Rotor/Weir Reprograming	Ditch Batching during Conversion	Retrofitting of Existing Aerobic Aerators	Refurbishing Sludge Piping Leading to Aerobic Digestor	Addition of Gas Collection System for Solids Handling	Addition of Air Scrubber for Solids Handling	Addition of Heat Exchanger and Boiler	**Potential Required Maintenance**	Replacement of Individual Fine screen Parts	Maintenance on Grit Chamber	Replacement of Brush Aerators	Maintenance on RAS/WAS Pumps	Replacement of Disk Filters	Maintenance on Chlorine Generation System	Maintenance on Chlorine Contact Basins	Maintenance on Centrifuge Sludge Dewaterers	Maintenance on Solar Drying Facility	Maintenance on Reclaimed Water Distribution Pumps
Required Construction Activity	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Fine Screen Removal	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Band Screen Installation	Gray	Red	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Ditch Rotor/Weir Reprograming	Gray	Green	Red	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Ditch Batching during Conversion	Gray	Red	Green	Green	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Retrofitting of Existing Aerobic Digestor	Gray	Green	Green	Green	Green	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Refurbishing Sludge Piping Leading to Aerobic Digestor	Gray	Green	Green	Green	Green	Green	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Addition of Gas Collection System for Solids Handling	Gray	Green	Green	Green	Green	Yellow	Yellow	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Addition of Air Scrubber for Solids Handling	Gray	Green	Green	Green	Green	Yellow	Yellow	Yellow	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Addition of Heat Exchanger and Boiler	Gray	Green	Green	Green	Green	Yellow	Yellow	Yellow	Yellow	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Potential Required Maintenance	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Replacement of Individual Fine screen Parts	Gray	Red	Red	Green	Red	Green	Green	Green	Green	Green	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Maintenance on Grit Chamber	Gray	Yellow	Yellow	Green	Red	Green	Green	Green	Green	Green	Gray	Yellow	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Replacement of Brush Aerators	Gray	Red	Red	Green	Red	Green	Green	Green	Green	Green	Gray	Red	Red	Gray	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Maintenance on RAS/WAS Pumps	Gray	Green	Green	Green	Red	Green	Green	Green	Green	Green	Gray	Green	Green	Red	Gray	Gray	Gray	Gray	Gray	Gray	Gray
Replacement of Disk Filters	Gray	Green	Green	Green	Red	Green	Green	Green	Green	Green	Gray	Green	Green	Red	Green	Gray	Gray	Gray	Gray	Gray	Gray
Maintenance on Chlorine Generation System	Gray	Green	Green	Green	Green	Green	Green	Green	Green	Green	Gray	Green	Green	Green	Green	Green	Gray	Gray	Gray	Gray	Gray
Maintenance on Chlorine Contact Basins	Gray	Green	Green	Green	Green	Green	Green	Green	Green	Green	Gray	Green	Green	Green	Green	Green	Green	Gray	Gray	Gray	Gray
Maintenance on Centrifuge Sludge Dewaterers	Gray	Green	Green	Green	Green	Green	Green	Green	Green	Green	Gray	Green	Green	Green	Green	Green	Green	Green	Gray	Gray	Gray
Maintenance on Solar Drying Facility	Gray	Green	Green	Green	Green	Green	Green	Green	Green	Green	Gray	Green	Green	Green	Green	Green	Green	Green	Green	Gray	Gray
Maintenance on Reclaimed Water Distribution Pumps	Gray	Green	Green	Green	Green	Green	Green	Green	Green	Green	Gray	Green	Green	Green	Green	Red	Green	Green	Green	Green	Gray

Green	No Additional Safety Concerns Expected
Yellow	Use Extra Caution, Equipment, or Procedures
Red	Do Not Complete at The Same Time
Gray	N/A or Duplicate

Appendix Z: Proposed Gantt Chart

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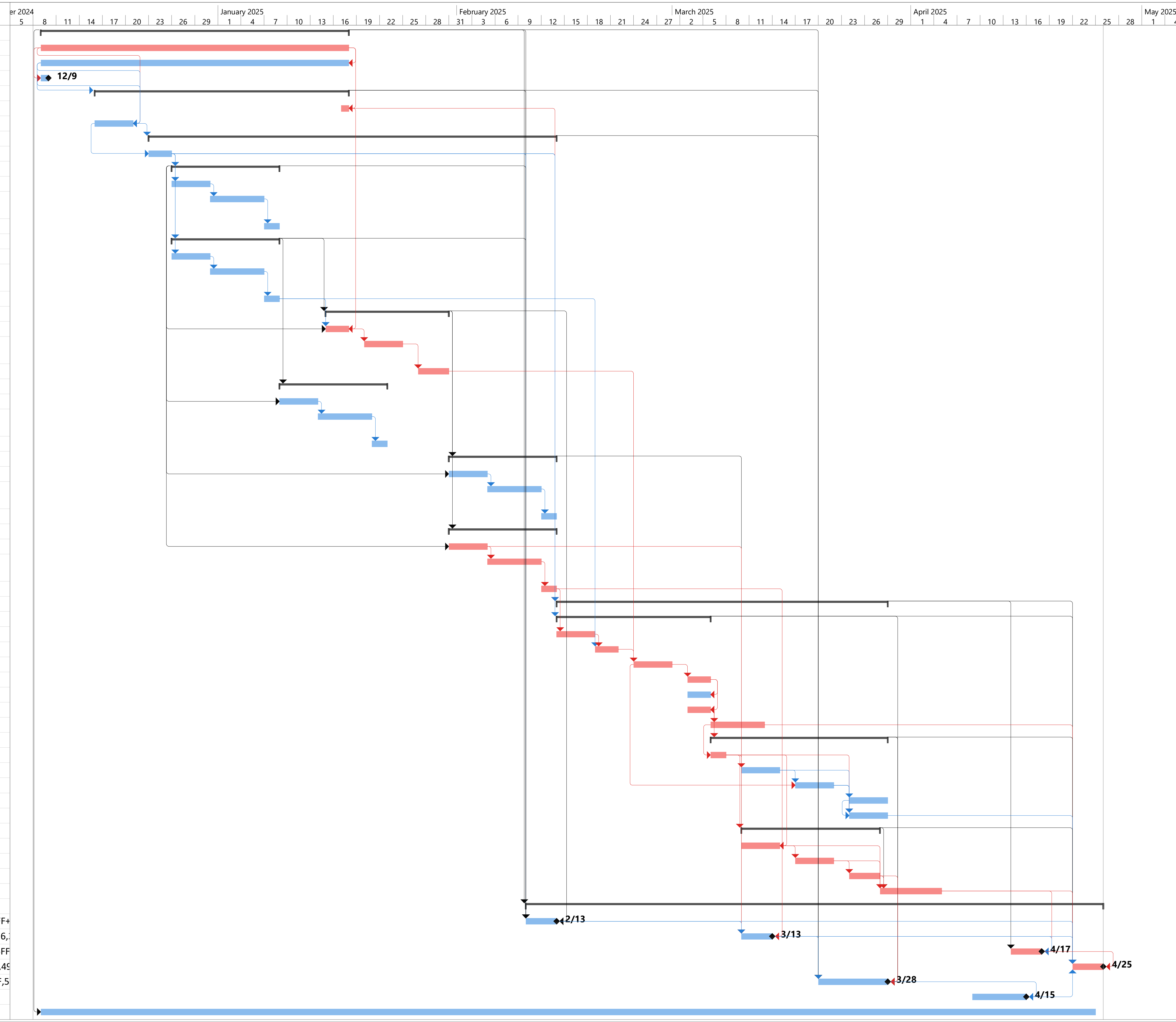
ID	Task Name	Duration	Start	Finish	Predecessors
1	**Task 1: Research Preparation**	**30 days**	**Mon 12/9/24**	**Fri 1/17/25**	
2	Task 1.1: Regulation Research	30 days	Mon 12/9/24	Fri 1/17/25	
3	Task 1.2: Wastewater Treatment Research	30 days	Mon 12/9/24	Fri 1/17/25	2FF
4	Task 1.3: WEF Application	1 day	Mon 12/9/24	Mon 12/9/24	2SS
5	**Task 2: Site Assessment**	**25 days**	**Mon 12/16/2**	**Fri 1/17/25**	**3SS+5 days**
6	Task 2.1: Site Visit	1 day	Fri 1/17/25	Fri 1/17/25	2FF
7	Task 2.2: Data Analysis	5 days	Mon 12/16/2	Fri 12/20/24	4SF,2SF,3SF
8	**Task 3: Treatment Process Selection**	**39 days**	**Mon 12/23/2**	**Thu 2/13/25**	**7**
9	Task 3.1: Determine Plant Requirements	3 days	Mon 12/23/2	Wed 12/25/2	7SS+3 days
10	**Task 3.2: Preliminary Treatment Selection**	**10 days**	**Thu 12/26/2**	**Wed 1/8/25**	**9**
11	Task 3.2.1 Determine Criteria	3 days	Thu 12/26/2	Mon 12/30/2	9
12	Task 3.2.2: Develop Preliminary Treatment Alternatives	5 days	Tue 12/31/24	Mon 1/6/25	11
13	Task: 3.2.3: Select Best Alternative	2 days	Tue 1/7/25	Wed 1/8/25	12
14	**Task 3.3: Primary Treatment Selection**	**10 days**	**Thu 12/26/2**	**Wed 1/8/25**	**9**
15	Task 3.3.1 Determine Criteria	3 days	Thu 12/26/2	Mon 12/30/2	9
16	Task 3.3.2: Develop Preliminary Treatment Alternatives	5 days	Tue 12/31/24	Mon 1/6/25	15
17	Task: 3.3.3: Select Best Alternative	2 days	Tue 1/7/25	Wed 1/8/25	16
18	**Task 3.4: Secondary Treatment Selection**	**12 days**	**Wed 1/15/25**	**Thu 1/30/25**	**14**
19	Tasks 3.4.1 Determine Criteria	3 days	Wed 1/15/25	Fri 1/17/25	10SS,17,6FF
20	Task 3.4.2: Develop Secondary Treatment Alternatives	5 days	Mon 1/20/25	Fri 1/24/25	19
21	Task: 3.4.3: Select Best Alternative	4 days	Mon 1/27/25	Thu 1/30/25	20
22	**Task 3.5: Advanced Treatment Selection**	**10 days**	**Thu 1/9/25**	**Wed 1/22/2**	**14**
23	Tasks 3.5.1 Determine Criteria	3 days	Thu 1/9/25	Mon 1/13/25	10SS
24	Task 3.5.2: Develop Advanced Treatment Alternatives	5 days	Tue 1/14/25	Mon 1/20/25	23
25	Task: 3.5.3: Select Best Alternative	2 days	Tue 1/21/25	Wed 1/22/25	24
26	**Task 3.6: Disinfection Technology Selection**	**10 days**	**Fri 1/31/25**	**Thu 2/13/25**	**18**
27	Tasks 3.6.1 Determine Criteria	3 days	Fri 1/31/25	Tue 2/4/25	10SS
28	Task 3.6.2: Develop Disinfection Alternatives	5 days	Wed 2/5/25	Tue 2/11/25	27
29	Task: 3.6.3: Select Best Alternative	2 days	Wed 2/12/25	Thu 2/13/25	28
30	**Task 3.7: Solids Management Selection**	**10 days**	**Fri 1/31/25**	**Thu 2/13/25**	**18**
31	Tasks 3.7.1 Determine Criteria	3 days	Fri 1/31/25	Tue 2/4/25	10SS
32	Task 3.7.2: Develop Advanced Treatment Alternatives	5 days	Wed 2/5/25	Tue 2/11/25	31
33	Task: 3.7.3: Select Best Alternative	2 days	Wed 2/12/25	Thu 2/13/25	32
34	**Task 4: Final Design**	**31 days**	**Fri 2/14/25**	**Fri 3/28/25**	**6,9**
35	**Task 4.1: Final Treatment Process Design**	**14 days**	**Fri 2/14/25**	**Wed 3/5/25**	**9**
36	Task 4.1.1: Preliminary Treatment Design	3 days	Fri 2/14/25	Tue 2/18/25	33
37	Task 4.1.2: Primary Treatment Design	3 days	Wed 2/19/25	Fri 2/21/25	17,36
38	Task 4.1.3: Secondary Treatment Design	5 days	Mon 2/24/25	Fri 2/28/25	21,37
39	Task 4.1.4: Advanced Treatment Design	3 days	Mon 3/3/25	Wed 3/5/25	38
40	Task 4.1.5: Disinfection Design	3 days	Mon 3/3/25	Wed 3/5/25	39FF
41	Task 4.1.6: Solids Management Design	3 days	Mon 3/3/25	Wed 3/5/25	39FF
42	Task 4.2: Site Layout	5 days	Thu 3/6/25	Wed 3/12/25	41
43	**Task 4.3: Hydraulic Analysis**	**17 days**	**Thu 3/6/25**	**Fri 3/28/25**	**41**
44	Task 4.3.1 Existing Piping Analysis	2 days	Thu 3/6/25	Fri 3/7/25	42SS
45	Task 4.3.2: New Piping Design	5 days	Mon 3/10/25	Fri 3/14/25	44
46	Task 4.3.2 Pump Selection	5 days	Mon 3/17/25	Fri 3/21/25	38SS+2 days,45
47	Task 4.3.4: Develop New Hydraulic Profile	5 days	Mon 3/24/25	Fri 3/28/25	46
48	Task 4.4: Construction Phasing	5 days	Mon 3/24/25	Fri 3/28/25	44,45,46,47SS
49	**Task 4.5: Life Cycle Cost Analysis**	**14 days**	**Mon 3/10/2**	**Thu 3/27/25**	**44**
50	Task 4.5.1: Construction Cost	5 days	Mon 3/10/25	Fri 3/14/25	44FF
51	Task 4.5.2: Maintenance and Operation Cos	5 days	Mon 3/17/25	Fri 3/21/25	50
52	Task 4.5.3: Calculate Life Cycle Cost	4 days	Mon 3/24/25	Thu 3/27/25	51
53	Task 5: Project Impacts Analysis	6 days	Fri 3/28/25	Fri 4/4/25	52,49,50,51
54	**Task 6: Project Deliverables**	**55 days**	**Mon 2/10/2**	**Fri 4/25/25**	**1**
55	Task 6.1: 30% Deliverables	4 days	Mon 2/10/25	Thu 2/13/25	14,1,5,9,10,18FF+
56	Task 6.2: 60% Deliverables	4 days	Mon 3/10/25	Thu 3/13/25	33FF+20 days,26,
57	Task 6.3: 90% Deliverables	4 days	Mon 4/14/25	Thu 4/17/25	53FF+9 days,56FF
58	Task 6.4: Final Deliverable	4 days	Tue 4/22/25	Fri 4/25/25	53,34,35,42,43,4
59	Task 6.5: Competition Final Report	7 days	Thu 3/20/25	Fri 3/28/25	1,5,8,35FF,43FF,5
60	Task 6.6: Competition Final Presentation	5 days	Wed 4/9/25	Tue 4/15/25	59FF+12 days
61	Task 7: Project Management	99 days	Mon 12/9/24	Thu 4/24/25	1SS



Appendix AA: Actual Gantt Chart

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ID	Task Name
1	**Task 1: Research Preparation**
2	Task 1.1: Regulation Research
3	Task 1.2: Wastewater Treatment Research
4	Task 1.3: WEF Application
5	**Task 2: Site Assessment**
6	Task 2.1: Site Visit
7	Task 2.2: Data Analysis
8	**Task 3: Treatment Process Selection**
9	Task 3.1: Determine Plant Requirements
10	**Task 3.2: Preliminary Treatment Selection**
11	Task 3.2.1 Determine Criteria
12	Task 3.2.2: Develop Preliminary Treatment Alternatives
13	Task: 3.2.3: Select Best Alternative
14	**Task 3.3: Primary Treatment Selection**
15	Task 3.3.1 Determine Criteria
16	Task 3.3.2: Develop Preliminary Treatment Alternatives
17	Task: 3.3.3: Select Best Alternative
18	**Task 3.4: Secondary Treatment Selection**
19	Tasks 3.4.1 Determine Criteria
20	Task 3.4.2: Develop Secondary Treatment Alternatives
21	Task: 3.4.3: Select Best Alternative
22	**Task 3.5: Advanced Treatment Selection**
23	Tasks 3.5.1 Determine Criteria
24	Task 3.5.2: Develop Advanced Treatment Alternatives
25	Task: 3.5.3: Select Best Alternative
26	**Task 3.6: Disinfection Technology Selection**
27	Tasks 3.6.1 Determine Criteria
28	Task 3.6.2: Develop Disinfection Alternatives
29	Task: 3.6.3: Select Best Alternative
30	**Task 3.7: Solids Management Selection**
31	Tasks 3.7.1 Determine Criteria
32	Task 3.7.2: Develop Advanced Treatment Alternatives
33	Task: 3.7.3: Select Best Alternative
34	**Task 4: Final Design**
35	**Task 4.1: Final Treatment Process Design**
36	Task 4.1.1: Preliminary Treatment Design
37	Task 4.1.2: Primary Treatment Design
38	Task 4.1.3: Secondary Treatment Design
39	Task 4.1.4: Advanced Treatment Design
40	Task 4.1.5: Disinfection Design
41	Task 4.1.6: Solids Management Design
42	Task 4.2: Site Layout
43	**Task 4.3: Hydraulic Analysis**
44	Task 4.3.1 Existing Piping Analysis
45	Task 4.3.2: New Piping Design
46	Task 4.3.2 Pump Selection
47	Task 4.3.4: Develop New Hydraulic Profile
48	Task 4.4: Construction Phasing
49	**Task 4.5: Life Cycle Cost Analysis**
50	Task 4.5.1: Construction Cost
51	Task 4.5.2: Maintenance and Operation Cos
52	Task 4.5.3: Calculate Life Cycle Cost
53	Task 5: Project Impacts Analysis
54	**Task 6: Project Deliverables**
55	Task 6.1: 30% Deliverables
56	Task 6.2: 60% Deliverables
57	Task 6.3: 90% Deliverables
58	Task 6.4: Final Deliverable
59	Task 6.5: Competition Final Report
60	Task 6.6: Competition Final Presentation
61	Task 7: Project Management