2019 AZ Water Student Design Competition

CCWRP REHABILITATION PROJECT: FINAL REPORT

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Northern Arizona University

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LETTER OF TRANSMITTAL

Water Environmental Federation Student Design Competition Team

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Northern Arizona University

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AZ Water Association Judging Panel
2019 Regional Competition

Dear AZ Water Association Judging Panel,

The Northern Arizona University Water student design team is pleased to present the final plan for the Cave Creek Water Reclamation Plant rehabilitation project as part of the Water Environment Federation student design competition. This plan includes an assessment of the existing conditions, a projected growth analysis, proposed improvements, unit process expansion phasing, and necessary supporting documentation. Phase 2 of the rehabilitation project is expected to reach capacity in 2037 and will cost approximately $173 million. Phase 3 of the project will reach capacity in 2050 and will cost approximately $138 million.

Abstract

The Cave Creek Water Reclamation Plant (CCWRP) was built to treat wastewater north of the Central Arizona Project canal. It was in operation from 2002 to 2009, when it was shut down due to slowed population and development growth in the sewershed. During operation, the plant had a maximum capacity of 8 million gallons per day (MGD) and produced A+ quality water. Due to subsequent growth in the sewer collection area, the City of Phoenix will reopen the plant in 2025. The purpose of this project is to increase the capacity of the facility to handle future flow and loading, as well as propose improvements to the process to maximize treatment efficiency.

An evaluation of historic wastewater data and population projections were used to develop a two-phase expansion of design flow capacity. Phase 2 capacity is 20 MGD and the final expansion (Phase 3) capacity is 33 MGD. The enclosed report includes an assessment of existing site conditions, a projected growth analysis, a proposed effluent usage, plant upgrade options, design criteria, selection of proposed improvements, economic analysis, and future recommendations.

The final expansion design will include:

- 5 12” slurry pumps
- 4 bar screens
- 3 Vortex Grit Chambers
- 4 Primary Sedimentation Basins
- 4 Aeration Basins
- 7 Secondary Sedimentation Basins
- 3 Tertiary Filters
- 9 Banks for Ultra Violet Disinfection
- 19 Reverse Osmosis Systems

The total cost of the proposed design will be approximately $173 million for Phase 2 and $138 million for Phase 3.

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List of Abbreviations

ADEQ – Arizona Department of Environmental Quality

BOD – Biological Oxygen Demand

CAP - Central Arizona Project

CASS – Central Arizona Salinity Study

CCWRP – Cave Creek Water Reclamation Plant

CFS – Cubic Feet per Second

COD – Chemical Oxygen Demand

CY – Cubic Yards

EA - Each

EPA – Environmental Protection Agency

ft^2 – Square Feet

FPS – Feet per Second (ft/s)

GPCPD – Gallons per Capita per Day

GPD– Gallons per Day

HGL – Hydraulic Grade Line

in – Inches

IMLR – Intermediate Mixed Liquor Pump

MGD – Million Gallons per Day

O&M – Operation and Maintenance

NPDES- National Polluant Discharge Elimination System

PSB- Primary Sedimentation Basin

TSS – Total Suspended Solids

TDS – Total Dissolved Solids

W – Watts

WWTP – Wastewater Treatment Plant

1.0 Project Introduction

1.1 Project Description

The Cave Creek Water Reclamation Plant (CCWRP) was initially built to support the development north of the Central Arizona Project (CAP) canal [1]. The plant produced Class A+ reclaimed water for irrigation and recharge within the service area. The CCWRP has been inactive since November of 2009 due to a decrease in projected population and development north of Loop 101. Figure 1 displays the location and site map of the CCWRP.

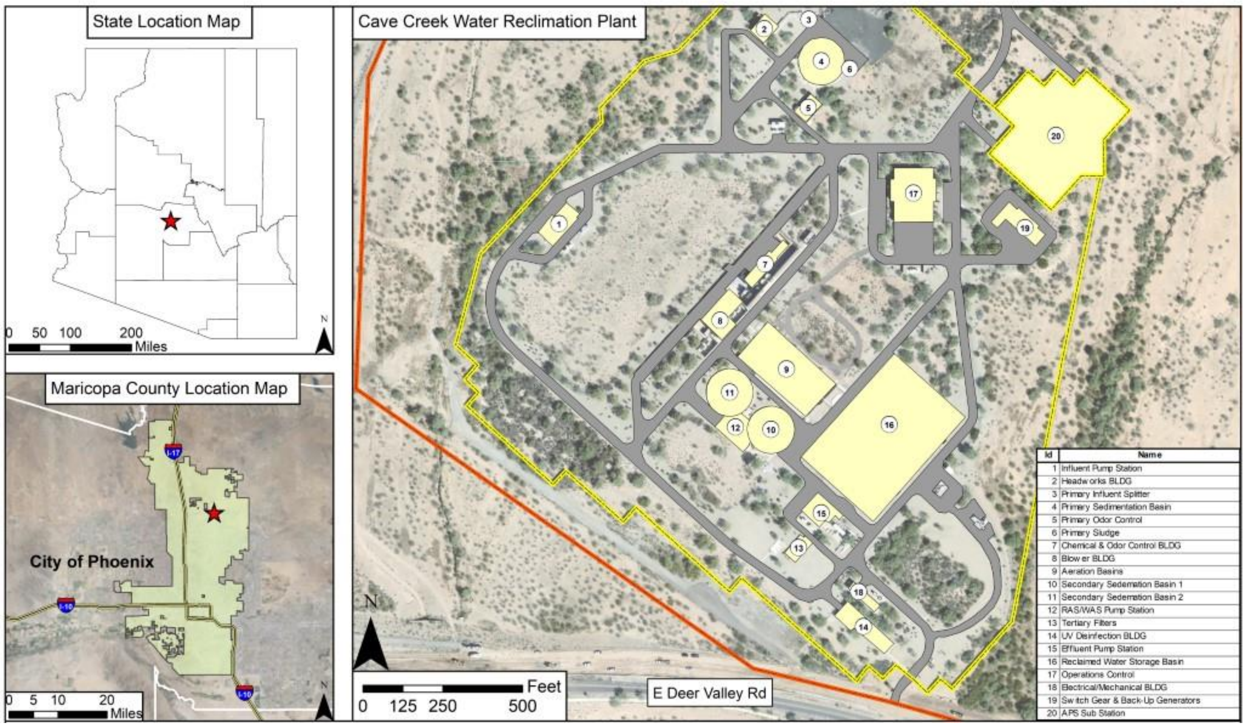

Figure 1: Site Map of Cave Creek and Existing CCWRP

The City of Phoenix plans to reopen the CCWRP by 2025. This will require plant expansions based off future flow predictions, reevaluation of plant operations, and recommended effluent use. Plant expansions will occur in 3 phases:

- Phase 1: Initial 2003 construction with a maximum capacity of 8 million gallons per day (MGD), this flow will be surpassed by the 2025 reopening
- Phase 2: Maximum capacity of 20 MGD, anticipated peak flow rate to be reached in 2037
- Phase 3: Maximum capacity of 33 MGD, anticipated peak flow rate to be reached in 2050

The evaluation of existing conditions yielded six areas of improvement; pump station, headworks, grit removal, ultraviolet (UV) disinfection, desalination, and energy source. The scope of work for the CCWRP Rehabilitation Project included the following:

- Reviewing current water quality regulations for effluent use options
- Analysis of historic wastewater quality data for the collection system
- Projections of population growth in the sewer collection area
- Conservative estimate of future flow and loading based off population projections
- Evaluation of selected treatment technologies to address areas needing improvement
- Proposed cost of expansions and improvements for Phase 2 and 3
- Evaluation of effluent use options

1.2 Team Member Roles

Katherine Dougherty:

Ms. Dougherty coordinated headworks and preliminary treatment design, including sizing pumps, headworks, and grit removal improvements and expansions. She worked with Mr. Lezhniuk to complete the hydraulic analysis and hydraulic grade line profile of the facility. She was the primary contact to vendors regarding product selection and cost estimates.

Hadley Habeck:

Ms. Habeck analyzed and improved treatment after grit removal, including the biological and chemical treatment processes. She calculated the predicted flow and loading for the primary clarifier, aeration basin, secondary clarifier, and tertiary treatment and sized the expansions of each accordingly. She worked with Mr. Lezhniuk to determine how the effluent can be implemented. She analyzed how the proposed solutions would improve the treatment efficiency to ensure the water quality requirements were met for the chosen effluent method, including analyzing the addition of a desalination system.

Artem Lezhniuk:

Mr. Lezhniuk analyzed the existing conditions of the CCWRP along with Mr. Stacy. This included identifying existing issues, conducting hydraulic analysis, and population analysis and projections. He worked with Ms. Habeck for the final selection of the desalination system. Mr. Lezhniuk and Ms. Habeck identified potential effluent applications and selected one based off criteria. He was also the primary AutoCAD drafter.

Hunter Stacy:

Mr. Stacy worked on the analysis of the existing and historical flow conditions with Mr. Lezhniuk and Ms. Habeck. He was tasked with replacing the UV disinfection system, feasibility analysis of implementing solar panels, and the cost analysis of the project. He generated the site map, collection area map, and expansion phasing map for the project.

2.0 Existing Condition

2.1 Historical Wastewater Flow and Loading

The CCWRP design capacities for flow, loading, and average daily flow from 2008 are presented in Table 1 [2]. The loadings include the Chemical Oxygen Demand (COD) and the Total Suspended Solids (TSS) that are produced [2]. Sewer meter readings from August 13th, 2017 to October 3rd, 2017 for Cave Creek Road were used to determine the minimum, maximum, and average daily flows. The sewer data was graphed to justify the City of Phoenix peaking factor of 1.85 [3]. See Appendix B for 2017 Sewer Meter Data) [1].

Additionally, the Total Dissolved Solids (TDS) data was obtained from the Central Arizona Salinity Study (CASS) in which the CCWRP was used as a case study in 2006 [4]. The TDS data is a measurement of salinity, which is crucial aspect for effluent application. Table 1 below outlines the flow and loading data of 2008 of the treatment plant in its last year of operation [2].

Table 1: CCWRP 2008 Existing Flow and Loading Conditions

CCWRP 2008 Exiting Design Conditions: Flowrate and Loading							
Population	Conditions	Q		COD		TSS	
		MGD	GPCPD[a]	lbs/day	lb/gal	lbs/day	lb/gal
40,000	First Phase Design Capacity	8	200	32,000	0.004	13,300	0.0017
	Daily Average	3.51	87.75	13,000	0.0037	6,000	0.0017

a = Gallons per Capita per Day

Figure 2 below provides a process flow diagram of the existing CCWRP treatment processes as well as the influent flow and loading capacity.

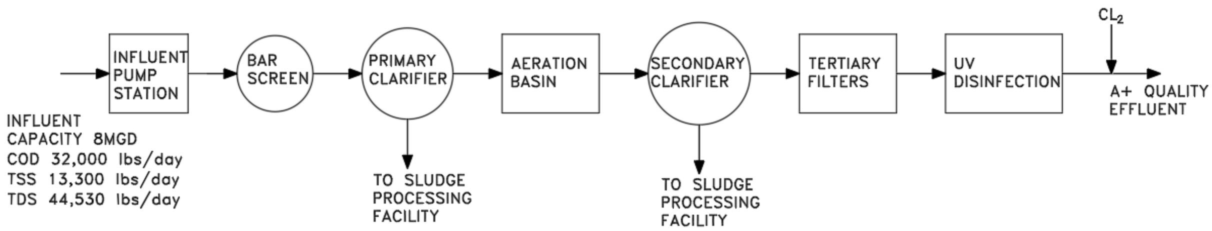

Figure 2: CCWRP Flow Diagram Displaying Existing Flow Capacities and Loading [2]

2.2 Existing Conditions

The treatment plant closed in 2009 and will require repair and improvements to be able to function by the proposed reopening in 2025. Issues while in operation included [2]:

- Large amounts of grit in the system causing increased wear on the pumps
- Lack of a grit removal system causing overloading in the primary sedimentation basin
- Insufficient disinfection due to operation and maintenance issues the UV system
- High salinity due to source water and residential water softener backwash [4]
- Lack of duplicate systems causing the plant to shut down for maintenance

Grit and sludge removed during the treatment process returned to the sewer system and was sent to the 91st Avenue Wastewater Treatment Plant for processing. The CCWRP maintains a low profile, most of its facilities are located underground to minimize noise and odor pollution. This presented a challenge with implementation of new processes due to constructability difficulties and limited space. A site map of the CCWRP can be seen in Figure 1.

2.3 Hydraulic Analysis

A hydraulic profile was created based on as-builts to analyze the implementation of the proposed improvements (Appendix A). The analysis required determining headlosses in the pipe network and unit processes. This was done to ensure that required head and water surface elevations met within all unit processes. Currently the influent enters the wet well of the influent pump station, from where it is being pumped to the headworks building. The wastewater is pumped to a vertical distance of 38 ft and horizontal distance of 800 ft. Once the influent reaches the headworks building it enters into the gravity-forced system as it travels through the reclamation plant to the effluent pump station. The total change in water surface elevation across the gravity-forced system is 28 ft.

2.4 Projected Population Analysis

The tributary area of the CCWRP is located north of Central Arizona Project (CAP) canal in Phoenix, Arizona. Online population data paired with a GIS collection system map was utilized to estimate the current population being served by the CCWRP. The 2019 population was estimated to be 75,334 people [5]. It was used as a baseline to estimate the 2025 through 2050 populations.

Census data indicates that the communities that are served by the CCWRP are growing at a rate of approximately 4% per year [5]. The 4% growth will likely decrease with time, but not by a large margin since the Phoenix metropolitan area tends to expand north of the CAP Canal[5]. Equation 1 and Equation 2 in Appendix C were used to estimate the population that will be served by the CCWRP upon opening of the facility in 2025 and by 2050 [6]. The most conservative estimate was used for the projected flow and loading calculations.

Population analysis performed by the team was compared to the CCWRP population estimate. Results are recorded in Table 2. Population analysis performed by the team was selected over given data because analysis method and the date of provided projections is unknown. It is suspected that this analysis was done before the economic recession of 2008, which significantly reduced the population growth across the nation; as such, the CCWRP population estimate could potentially be an overestimate.

Table 2: Population Estimates with Log and Percent Growth Measurements

Year	Provided Population Estimate	Computed Population Estimate (Log Growth)	Computed Population Estimate (Percent Growth)
2019	N/A	75,300	75,300
2020	79,336	79,235	79,173
2025	116,664	97,026	96,179
2030	153,992	119,212	116,416
2040	223,939	175,130	162,530
2050	297,083	262,351	224,948

2.5 Projected Flow and Loading Analysis

The original design capacity of the plant was 8 MGD. It was determined that flows will exceed this capacity by 2025 when the plant will reopen. The future improvements to the plant were divided into 3 phases. Phase 1 is the original 8 MGD capacity of the plant. Phase 2 capacity is 20 MGD, expected to be reached in 2037. The final phase of the plant, Phase 3, is 33 MGD, expected to be reached in 2050. These are summarized in Table 3 below. Implementation of Phase 2 improvements will be complete by the plant reopening in 2025.

The projected influent loads were determined from the historic CCWRP data and an EPA study conducted on the plant. The predicted pounds of COD and TSS produced per day per person (lb/day/per) were calculated using loading capacity and population predictions [2]. The loading per person was multiplied by the population to estimate daily loading of COD and TSS. The COD was converted to BOD using an average ratio of 0.55 BOD to COD [7]. The influent TDS values were obtained from the CASS salinity predictions for the increased water softeners in the serviced area [4]. A complete table of flow and loading predictions as well as TDS predictions explained can be found in Appendix D.

Table 3: CCWRP Predicted Flow and Loads of Each Phase

CCWRP Flow and Loading Phase Summary						
	Flow (MGD)	Year of Implementation	COD (lb/day)	BOD (lb/day)	TSS (lb/day)	TDS [4] (mg/L)
Phase 1	8	Existing	32,000	17,600	13,300	1,520
Phase 2	20	2025-2037	67,480	37,114	28,046	1,696
Phase 3	33	2037-2050	113,036	62,170	46,980	1,775

2.6 Proposed Hydraulics

The reconstruction and expansion of the preliminary treatment consisted of pump and bar screen replacements, channel reconstruction, and implementing vortex grit chambers. The improved preliminary systems were designed to be in parallel to ensure constant headloss through each unit. The headloss across the improved preliminary treatment was calculated to be 2.1 feet (Appendix K). The HGL profile was adjusted to accommodate the headloss across the preliminary treatment to ensure constant flow to the downstream units. This requires the channel operating floor in the headworks building to be increased by 1.5 feet (Appendix K).

For the ease of construction and expansion, the height of the headworks channel will be increased by 2 feet of reinforced concrete to the operation floor. The final design of the channel will have a total depth of 9 feet and will accommodate for 2 feet of freeboard. The Phase 2 expansion requires both existing channels to be heighted by 2 feet. Phase 3 requires a third channel and bar screen.
The capacity of each improved infrastructure was modeled in Excel to determine unit expansion. The expansion is based on the design parameters of the predicted peak flows and loadings capacities for 2025 through 2050. The modeled units include the pump station, bar screens, grit chamber, primary sedimentation basin, aeration basin, secondary sedimentation basin, tertiary treatment, desalination, and UV disinfection (Appendix F).

2.7 Effluent Standards

The standards and regulations for reclaimed water are dependent upon the designated use. Federal standards for reclaimed water use are set by the EPA, but more strict state standards are set by the Arizona Department of Environmental Quality (ADEQ). Appendix G provides the effluent requirements for different reclaimed water applications as per Title 18 Chapter 11 of the Arizona Administrative Code [8]. The level of treatment required is dependent upon the application type. Nutrients and salts are of particular importance and removal practices may be required to depending on concentration in the water and end use application [9]. Table 50 in Appendix G summarizes the treatment levels required for different end uses as well as the cost relationship of increased treatment.

3.0 Proposed Design Solutions

3.1 Effluent Usage

Problem: Produce A+ quality effluent that can be used to diversify City of Phoenix water source portfolio and to make the treatment process as profitable as possible.

Requirements: Effluent quality must be compliant with the Arizona Administrative Code and the EPA standards based on selected application, as described in Appendix G.

Criteria: Optimal effluent usage was determined using the decision matrix presented in Table 5. Three general effluent application options were assessed: irrigation, groundwater recharge, and direct potable use. The application types were evaluated on five criterions, which differ from the criteria used for the process technology. Table 4 displays the criteria, individual weights, and justification for given weights for the selection of effluent usage.

Table 4: Design Criteria for Effluent Usage

Criteria for Effluent Usage		
Criteria	Weight	Reasoning
Environmental Impact	30%	Higher weight due to direct impact on receiving environment. A higher score indicates a minimized negative impact or positive impact on receiving land or water body
Social Impact	30%	Higher weight due to high community impact. Analyzed according to impact on the consumer and according to public perception
Feasibility / Constructability	15%	Analyzed based on national, state, and city regulations for each application and treatment requirements to meet regulations. Also considered infrastructure and energy required.
Maintenance and Operation	10%	Low weight due to minimal requirements for each option. Encompasses regularly scheduled maintenance, repair of damaged parts, and staffing.
Economic Analysis	15%	Low weight due to minimal variation in each option. Encompasses infrastructure cost, installation costs, and operation maintenance

Effluent Reuse Options and Decision Matrix:

The Direct Potable Reuse: This option could be beneficial for general public by potentially reducing water costs and diversifying the water supplies. However, it was determined as not feasible due to high effluent standard requirements and poor public perception of recycling wastewater for drinking water.

Groundwater Recharge: It could be a viable option because the plant has preexisting wells. The social impact of groundwater recharge is less than that of irrigation because the consumer is not directly benefiting from the effluent use. Groundwater recharge has a higher standard for coliform, which lowers its environmental impact, but raises the cost. In addition, wells require back flushing which lowers maintenance and operation costs.

Irrigation: Because the plant is designed to produce A+ Standard water, the effluent can be used for any type of irrigation [8]. The irrigation would have a positive environmental impact on the community by reducing the potable water used for irrigation and crops. Based off the results shown in Table 5, the effluent will be used for irrigation purposes.

Table 5: Effluent Usage Decision Matrix

Effluent Usage Weighted Decision Matrix							
Criteria	Weight	Irrigation		Groundwater Recharge		Direct Potable Reuse	
		Raw Score	Weighted Score	Raw Score	Weighted Score	Raw Score	Weighted Score
Environmental Impact	30%	4	1.2	4	1.2	2	0.6
Feasibility / Constructability	10%	4	0.4	5	0.5	1	0.1
Maintenance and Operation	15%	3	0.45	3	0.45	1	0.15
Social Impact	30%	5	1.5	3	0.9	1	0.3
Economic Impact	15%	4	0.6	2	0.3	4	0.6
Total Score Out of 5	100%		4.15		3.35		1.75

Solution: The treatment processes will be optimized for irrigation purposes, specifically concerning salinity and disinfection. Based off the historic water quality data for the CCWRP, the most restrictive characteristics limiting reuse of the effluent water was the coliform counts and the TDS. The plant was unable to attain the permitted disinfection levels required for coliform removal because of maintenance issues with the disinfection system [4]. The coliform standard for irrigation use is lower than that of groundwater recharge but will still require changes to the disinfection system to be met [8].

High salinity hinders plant growth because it restricts the plant's ability to extract water from the soil, therefore rendering the water poor for irrigation purposes [4]. Golf courses that used the effluent for irrigation when the plant was in operation reported brown grass due to high salinity [10] .

The two treatment improvements required for the chosen reuse option are UV disinfection to lower coliform counts and desalination to lower TDS. The other plant upgrades were determined as necessary to improve the general efficiency of the plant.

3.2 Plant Upgrade Alternatives

The plant upgrades consist of improving the pump station, bar screens, primary and secondary sedimentation basins, tertiary filters, UV disinfection, and effluent usage. Additionally, the design will incorporate new units such as adding a grit removal system, desalination system, and solar panels.

3.3 Design Criteria

Treatment alternatives were determined by using a decision matrix with six criterions to optimize the design and treatment efficiencies. The criterions include process efficiency improvements, feasibility and constructability, maintenance and operation (O&M), staffing levels, economic impact, environmental and social impacts.

The six criterions were assigned an individual weighted percentage that totaled to 100%. Raw scores were also assigned to each alternative with respect to the criteria ranging from 1 to 5. The weighted percentages were multiplied with the raw scores, then totaled to determine which alternative would meet the standards of the facility. The ratings for each criterion can be found in Table 6.

Table 6: Rating Table for Plant Upgrade Alternatives Decision Matrix

Rating Table	
Rating Score	Criteria
1	Design does not meet criteria
2	Design partially meets criteria
3	Design meets basic criteria
4	Design partially exceeds criteria
5	Design exceeds criteria

3.3.1 Influent Pump Station

Problem: The existing pump station pumps experienced erosion due to grit and became clogged from horse hair and rags [2].

Requirement: The pump station must overcome 70 ft of head. The pump station is required to constantly deliver various influent flow rates into the headworks building with adequate efficiency and minimal maintenance.

Criteria: Three pump options were evaluated on six criterions. Table 7 displays the criteria, individual weights, and justification for given weights for the influent pump station.

Table 7: Influent Pump Station Design Criteria for Pump Selection

Criteria for Influent Pump Station		
Criteria	Weight	Reasoning
Process Efficiency	30%	Project emphasis of optimizing treatment efficiency. This encompasses the treatment quality, energy consumption, headlosses, and continuous operation.
Feasibility / Constructability	10%	Removal of existing, installation, construction, functional HGL, and fits within boundaries.
Maintenance and Operation	25%	High weight because of the existing issues related to operations and maintenance. Includes scheduled maintenance, operational costs, and repair for damaged parts.
Staffing Levels	5%	Low weight due to the existing need for monitoring staff and minimal increase in staffing needs.
Economic Impact	25%	High weight due to the capital costs and construction cost.
Environmental and Social Impacts	5%	Low weight due to low pollution, societal concerns, and demand on resources

Influent Pump Station Options and Decision Matrix:

Existing Centrifugal Pumps: The existing system has experienced grit erosion and clogging due to rags and hair, causing an impact of the plant's efficiency [2]. Additionally, by 2025, the pump station will have been inactive for almost two decades causing an increase in maintenance and operation, staffing levels, and overall economic demand. The existing pump station has a low impact the environment and society since the pumps are in an enclosed building, with noise reducing baffles to reduce noise pollution.

Grinders and Centrifugal Pumps: The existing pumps would be replaced with new, properly sized centrifugal pumps. Grinders would be implemented before these pumps to reduce the size of particles entering the pumps [11]. This would reduce the wear of the pumps and allow for better process efficiency. The use of grinders and centrifugal pumps would require more construction, an increase in capital cost, and an increase in operation, maintenance and staffing. Grinders also create noise pollution, causing a negative impact on the environment and the surrounding community.

Heavy Duty Slurry Pump: The existing pumps would be replaced with four 12-inch heavy duty slurry pumps. The pressurized inflow pipe for each pump would need to be resized to 14 inches. The slurry pumps have a highly abrasive design to prevent erosion and clogging [12]. This would increase the process efficiency, lower the operation and maintenance, staffing levels, and overall economic demand for each pump. The slurry pumps would have a low impact on the environment and the surrounding community since the pumps will be placed in the existing enclosed building with noise reducing baffles.

Table 8: Influent Pump Station Decision Matrix

Influent Pump Station Weighted Decision Matrix							
Criteria	Weight	Existing Centrifugal Pumps		Grinders & Centrifugal Pumps		Heavy Duty Slurry Pump	
		Raw Score	Weighted Score	Raw Score	Weighted Score	Raw Score	Weighted Score
Process Efficiency	30%	1	0.3	3	0.9	4	1.2
Feasibility/Constructability	10%	5	0.5	2	0.2	4	0.4
Maintenance and Operation	25%	2	0.5	2	0.5	4	1.0
Staffing Levels	5%	2	0.1	2	0.1	3	0.15
Economic Impact	25%	2	0.5	3	0.75	4	1.0
Environmental and Social Impacts	5%	4	0.2	3	0.15	4	0.2
Total Score Out of 5	100%		2.1		2.6		3.95

Solution: The final design for Phase 2 will replace the existing pumps with four 12-inch Slurry Pumps to support the peak design flow rate of 20 MGD and the Phase 3 flow rate of 33 MGD. Refer to Appendix E for pump specifications. This pump was chosen based on its non-clogging, highly abrasive design, and ability to handle solids up to 11 in in diameter. The pumps have a range of speeds that can be adjusted for the pump to run more efficiently and provide 70 ft of head needed for the system. Appendix F displays the calculations for pump selection and system curve.

3.3.2 Headworks

Problem: The existing headworks has 2 bar screens that experienced higher than anticipated wear due to grit erosion and clogging from rags and horse hair. The screenings discharge to the sewer experienced sedimentation issues [2].

Requirements: Remove rags, hair, and other particles larger than the grit removal is sized for and maintain a channel velocity of 1.5 ft/s to 3 ft/s to prevent sediment deposition and damage to the system [13].

Criteria: Table 9 displays the criteria for the headworks, individual weights, and justification for the given weights for the final bar screen selection.

Table 9: Design Criteria for Bar Screen Selection

Criteria for Headworks		
Criteria	Weight	Reasoning
Process Efficiency	30%	Project emphasis of optimizing treatment efficiency. This encompasses the treatment quality, energy consumption, headloss, and continuous operation.
Feasibility / Constructability	30%	High weight due to required channel construction to maintain a proper HGL. Evaluates removal of existing, installation, construction, and fits within boundaries.
Maintenance and Operation	10%	Scheduled maintenance, operational costs, and repair for damaged parts.
Staffing Levels	5%	Low weight due to the existing need for monitoring staff and minimal increase in staffing needs.
Economic Impact	20%	High weight due to the capital costs and construction cost.
Environmental and Social Impacts	5%	Low weight due to low pollution, societal concerns, and demand on resources.

Headworks Options and Decision Matrix:

<u>Existing System:</u> The existing system experienced grit erosion and clogging from rags and horse hair causing an impact in the plants efficiency. The existing bar screens have been idle 10 plus years causing a demand for increased maintenance and staffing levels. This system would not be able to handle the predicted increase in flow for 2025. The existing system has a low impact on the environment since the bar screens are in an enclosed building, where noise and odor pollution are controlled.

<u>Fine Screen - Step Screen:</u> The existing system would be replaced with step screens to support the predicted flows for Phase 2 and Phase 3. The determination of the step screen was dependent on the type of pump that was selected. Since the heavy-duty slurry pump was selected, the fine screen would experience clogging and grit erosion, reducing the efficiency of the step screen and increasing the need for maintenance, staffing, and the overall economic demand of the system [14]. The system produces 8-in of headloss, which mitigates the need to raise increase the depth of the channel. One additional channel is required to be constructed to support the predicted flow for Phase 3.

<u>EscaMax Perforated Bar Screen:</u> The existing system would be replaced with the EscaMax Perforated bar screen. The perforated plates allow for a greater separation efficiency to process grit and larger materials [15]. The EscaMax is easily able to be retrofitted into the two existing channels and can handle the predicted flow rates for both phases, while producing a headloss of 27 in. Implementing this system has a negative economic impact since it requires the channel elevation to be raised by 2 ft. One additional channel is required to be constructed to support the predicted flow for Phase 3.

Table 10: Decision Matrix for Bar Screen

Headworks Weighted Decision Matrix							
Criteria	Weight	Existing		Fine Screen: Step Screen		EscaMax Perforated Bar Screen	
		Raw Score	Weighted Score	Raw Score	Weighted Score	Raw Score	Weighted Score
Process Efficiency	30%	2	0.6	3	0.9	4	1.2
Feasibility/Constructability	30%	5	1.5	3	0.9	2	0.6
Maintenance and Operation	10%	1	0.1	3	0.3	5	0.5
Staffing Levels	5%	2	0.1	2	0.1	4	0.2
Economic Impact	20%	2	0.4	2	0.4	2	0.4
Environmental and Social Impacts	5%	3	0.15	4	0.2	4	0.2
Total Score Out of 5	100%		2.85		2.8		3.1

Solution: The final design for Phase 2 will incorporate two EscaMax bar screens to support the peak design flow rate of 20 MGD. Phase 3 will require one additional screen to support the peak flow rate of 33 MGD and allow for redundancy. This screen was selected based on its high process efficiency and low maintenance, easy retrofit into the existing channels and will provide a functioning HGL after the channel elevation is raised 2 ft. This design has a significant economic impact due to the modification of the existing channels, adding an additional channel for redundancy, and initial cost of equipment.

3.3.3 Grit Removal

Problem: There is no existing grit removal system at CCWRP. The grit is being accumulated in the primary sedimentation basin, causing overloading on the sludge scrapper mechanism. This resulted in frequent primary sedimentation basin shutdowns for maintenance and a reduction in process efficiency.

Requirements: Grit removal system must catch 95% of the particles of up to 0.0117 in (50 mesh) diameter from the wastewater, have minimal headloss across the unit, and be easily implemented into the existing infrastructure.

Criteria: Table 11 displays the criteria, individual weights, and justification for the final design selection for the grit chamber.

Table 11: Design Criteria for Grit Removal Selection

Criteria for Grit Removal		
Criteria	Weight	Reasoning
Process Efficiency	30%	Evaluates the quality of the treatment, energy consumption, removal efficiency and continuous operation
Feasibility / Constructability	25%	High weight due to required construction for new chamber equipment. Evaluates installation and construction requirements
Maintenance and Operation	15%	Scheduled maintenance, operational costs, and repair of damages
Staffing Levels	5%	Low weight due to the existing need for monitoring staff and minimal increase in staffing needs.
Economic Impact	15%	High weight due to the capital costs and construction cost.
Environmental and Social Impacts	10%	Encompasses pollution, odor control, societal concerns, and demand on resources.

Grit Removal Options and Decision Matrix:

Existing System: Currently, there is no existing grit removal system. The grit is primarily removed in the primary sedimentation basin. Due to the heavy grit loads, the plant was not able to remain in constant operation due to maintenance on the primary sedimentation basin [2]. This method reduces the process efficiency to remove grit and the ability to maintain in constant operation. This increases the need for maintenance and staffing costs. Removing the grit in the primary sedimentation basin has a low impact on the environment since it is in an enclosed system.

Aerated Grit Chamber: The aerated grit chamber provides high grit removal efficiencies over a variety of flow rates and can be used for pre-aeration and flocculation [16]. However, this system has a high energy demand and can require more maintenance on the aeration system, which increases the overall cost of the system. Additionally, this system has a negative impact on the environment and surrounding community due to potential releases of odor and volatile organic compounds from the chamber.

Vortex Grit Chamber: The vortex grit chamber provides a greater process efficiency due to the high grit removal efficiency and low energy demand [17]. The high removal efficiencies would reduce the need for maintenance on the primary sedimentation basin, allowing for the plant to stay in constant operation. This system has a compact, space saving design, which allows for easier constructability to the existing plant. The vortex grit chamber has a higher unit cost for construction cost. The vortex grit chamber has a low impact on the environment since it is an enclosed system.

Table 12: Grit Removal Decision Matrix

Grit Removal Weighted Decision Matrix							
Criteria	Weight	Existing (Nonexistent)		Aerated Grit Chamber		Vortex Grit Chamber	
		Raw Score	Weighted Score	Raw Score	Weighted Score	Raw Score	Weighted Score
Process Efficiency	30%	1	0.3	3	0.9	5	1.5
Feasibility/Constructability	25%	5	1.25	3	0.75	3	0.75
Maintenance and Operation	15%	2	0.3	3	0.45	4	0.6
Staffing Levels	5%	2	0.1	2	0.1	3	0.15
Economic Impact	15%	2	0.3	3	0.45	2	0.3
Environmental and Social Impacts	10%	5	0.5	2	0.2	5	0.5
Total Score Out of 5	100%		2.75		2.85		3.8

Solution: The final design for Phase 2 will incorporate two Huber Vortex Grit Chamber that can hold the peak design flow rate of 20 MGD. Phase 3 will require adding one additional vortex grit chamber to support the peak design flow rate of 33 MGD and allow for redundancy. The vortex grit chamber was selected for the final design for its low energy consumption, minimal headloss of 1.7 in, high separation efficiencies, low maintenance, and low odor releases. The initial cost of the vortex grit chamber is higher due to the cost per unit and installation costs. However, the costs for maintenance and operation for downstream equipment will decrease after the grit is removed.

3.3.4 Desalination

Problem: The CCWRP treated effluent has high levels of salinity due to TDS in source water and residential and commercial water softener use in the sewershed. New residential developments are more likely to use water softeners, thus growth in the area will increase TDS. High levels of salinity limit effluent reuse possibilities and there is currently no treatment method in place to reduce TDS levels.

Requirements: Reduce the TDS levels to below 1,000 mg/L - the concentration at which irrigation problems due to salinity emerge.

Criteria: Three options were evaluated on six criterions. Table 13 displays the criteria, individual weights, and justification for given weights for the desalination system selection.

Table 13: Criteria for Desalination System

Criteria for Desalination System		
Criteria	Weight	Reasoning
Process Efficiency	10%	Minimal effect on other processes, score determined by energy usage
Feasibility / Constructability	20%	System size requirements, treatment of concentrated salinity, construction of building to house treatment system
Maintenance and Operation	15%	Membrane backwashing, repair required, pressure requirements
Staffing Levels	15%	Monitoring required for system and evaporation pond
Economic Impact	20%	High weight encompasses benefits associated with reducing salinity
Environmental and Social Impacts	20%	High weight due to negative impacts on receiving grounds if salinity is not removed

Desalination Options and Decision Matrix:

<u>Existing System:</u> There is currently no system in place to remove TDS from the CCWRP effluent. Lack of salinity removal results in lower process efficiency because it lowers the quality of the effluent. No action towards salinity removal would most feasible because it would require no maintenance, operation, or staffing. High levels of salinity limit effluent application options, which limits revenue opportunities for the plant, therefore raising the lifecycle cost of the entire operation. High salinity can negatively impact the environment by causing browning of grass when used for irrigation [4].

<u>Reverse Osmosis (R.O.) SANRO HS2:</u> The SANRO reverse osmosis option is individual membrane that would be combined in parallel to treat a percentage of the effluent to lower salinity. It increases process efficiency by improving the quality of the effluent but is not feasible because it requires pumps to provide adequate pressure, construction of building to house the system, and an evaporation pond for membrane concentrate [18]. The system would require regular maintenance including backwashing of membranes and salt removal and disposal from the evaporation ponds, which requires staffing. It would lower lifecycle costs by allowing for irrigation use, which produces revenue for the plant. Irrigation use would also have a positive impact on the receiving environment and community.

<u>R.O.- PureAqua TW-900K-18780:</u> The PureAqua is a complete system containing 126 R.O. membranes and a pump to provide operating pressure. Similar to the SANRO system, it would improve process efficiency. It would be more feasible because it has pumps included in the system and is more compact and therefore a lower space requirement [19]. The uniformity of the system would allow for easier maintenance and replacement of parts, also improving lifecycle costs. The staffing and social and environmental impacts are the same as the SANRO system.

Table 14: Desalination System Decision Matrix

Desalination System Weighted Decision Matrix							
Criteria	Weight	Existing (Nonexistent)		R.O. – SANRO HS2		R.O. – TW-900K-18780	
		Raw Score	Weighted Score	Raw Score	Weighted Score	Raw Score	Weighted Score
Process Efficiency	10%	1	0.1	4	0.4	4	0.4
Feasibility/Constructability	20%	5	1.0	2	0.4	4	0.8
Maintenance and Operation	15%	5	0.75	1	0.15	2	0.3
Staffing Levels	15%	5	0.75	2	0.3	2	0.3
Economic Impact	20%	1	0.2	3	0.6	4	0.8
Environmental and Social Impacts	20%	1	0.2	5	1.0	5	1.0
Total Score Out of 5	100%		3.0		2.85		3.6

Solution: The selected design is the Pure Aqua TW-900K-18780 Reverse Osmosis system [19]. Each system can handle 900,000 gpd. Phase 2 requires 10 systems and Phase 3 requires 18 systems. An insulated warehouse about 50 ft by 100 ft and 30 ft tall will be constructed to house the system and allows adequate space for maintenance. An evaporation pond will be constructed to treat membrane concentrate.

3.3.5 UV Disinfection

Problem: The existing UV disinfection system has maintenance issues and does not disinfect the effluent to the required standards. The bulbs in the existing system leak and break, lowering the disinfection efficiency and adding cost to the maintenance and operation of the system [2].

Requirement: The UV system must disinfect the effluent, consume minimal energy and require minimal maintenance.

Criteria: Table 15 displays the criteria that the UV system will be judged on, weights for the criteria, and justification for given weights in the UV disinfection decision matrix.

Table 15: Design Criteria for UV Disinfection

Criteria for UV Disinfection Selection		
Criteria	Weight	Reasoning
Process Efficiency	25%	Project emphasis on efficiency of system disinfection and energy usage
Feasibility / Constructability	10%	Accounts for the removal of the existing system, transportation and instillation of the new system, and initial capital cost of the system
Maintenance and Operation	25%	Accounts for maintenance, UV bulb replacement, cleaning of system, and operational costs and demands.
Staffing Levels	10%	Low weight due to the existing need for monitoring staff and minimal increase in staffing needs
Economic Impact	20%	High weight due to cost of energy, maintenance cost, UV bulb cost, and high capital cost of UV disinfection systems
Environmental and Social Impacts	10%	Low weight due to low pollution, societal concerns, and demand on resources

UV Disinfection Options and Decision Matrix:

Existing System: The existing system is inefficient due to the high electrical demand and breakage of bulbs. The bulb deficiencies cause an increase in operation and maintenance, requires more staff and increases the economic demands of the system. The system is not able to fully disinfect the water impacting the environment by increasing resource demand on other sources [2].

Trojan UV Signa: This system implements energy efficient bulbs, sensors to adjust power and intensity of bulbs, automated weirs to maintain the highest efficiency possible [20]. This system is designed to be directly installed in place of the existing system. The automated aspects of the system reduce staffing needs, the economic impacts and the environmental impacts. The UV has a positive impact with society because it does not involve chemicals.

Trojan UV 4000+: This system would be a direct replacement of the existing system with modern improvements. The system uses 3200 Watt (W) high intensity bulbs that draw large amounts of power and increase the demand on the environment [21]. The system would need to be shut down to replace the low life bulbs decreasing efficiency, increasing operation and maintenance values, and increasing staffing needs. This system has casings that fixed the issues of water leaking and bulbs breaking.

Table 16 shows the decision matrix for the three UV disinfection options based off the criteria, requirements, system specifications and the problems that the CCWRP is facing.

Table 16: Decision Matrix for UV Disinfection

UV Disinfection Weighted Decision Matrix							
Criteria	Weight	Existing		Trojan UV Signa		Trojan UV 4000+	
		Raw Score	Weighted Score	Raw Score	Weighted Score	Raw Score	Weighted Score
Process Efficiency	25%	2	0.50	4	1.00	4	1.00
Feasibility/Constructability	10%	5	0.50	5	0.50	3	0.30
Maintenance and Operation	25%	1	0.25	4	1.00	3	0.75
Staffing Levels	10%	1	0.10	3	0.30	3	0.30
Economic Impact	20%	1	0.20	3	0.60	2	0.40
Environmental and Social Impacts	10%	2	0.20	3	0.30	3	0.30
Total Score Out of 5	100%		1.75		3.70		3.05

Solution: The Phase 2 design will include the Trojan UV Signa system. This system will include two flow channels, six duty banks per channel, 161,000 W UV lamps per bank, a control system with sensors, automated sluice gate for water level adjustment, and one redundant bank per channel. Phase 3 will include the addition of two duty banks per channel. This design was chosen because of low energy demand, low maintenance needs, ease of maintenance, and ease of installation.

3.3.6 Solar Power

Problem: The UV disinfection system and Proposed R.O. system both require large amounts of energy leading to a high operation cost.

Requirement: Provide enough energy to offset the costs of the systems in the plant.

Criteria: Three options were evaluated on six criterions. Table 17 displays the criteria, individual weights, and justification for given weights for the solar panel selection.

Table 17: Design Criteria for Solar Panel Selection

Criteria for Solar Power Feasibility Selection		
Criteria	Weight	Reasoning
Process Efficiency	15%	Project emphasis on efficiency of system disinfection and energy usage
Feasibility / Constructability	25%	High weight because of transportation, instillation, and initial capital cost of installing solar power
Maintenance and Operation	20%	High weight because of addition of new system to maintain and reduction in operational needs from exterior sources
Staffing Levels	5%	Low weight due to the existing need for monitoring staff and minimal increase to staffing needs
Economic Impact	15%	Median weight due to the cost of energy, maintenance cost, and high capital cost of solar power systems
Environmental and Social Impacts	20%	High weight because of demand on other resources, public opinion, and pollution considerations

Solar Power Options and Decision Matrix:

Existing System: Currently there is no solar or alternative power to help offset the electrical usage of the plant causing a negative impact on the economic considerations, process efficiency and environmental impacts. This has no impact on the staffing levels and does not increase the operation and maintenance.

Addition of Solar Power: This plan would call for the addition of 3400, 400 W solar panels that would reduce the electrical demand, long term economic impact and environmental impacts [22]. The panels would be easily constructed with a ground mounted system making operation and maintenance feasible. The panels have a high efficiency of 19% improving the operation and social impacts.

Table 18 shows the decision matrix comparing the alternatives above with the requirements, criteria, and needs of the CCWRP.

Table 18: Decision Matrix for Solar Panel Selection

Solar Power Feasibility Weighted Decision Matrix					
Criteria	Weight	Existing No Solar Power		Addition of Solar Power	
		Raw Score	Weighted Score	Raw Score	Weighted Score
Process Efficiency	15%	1	0.15	4	0.60
Feasibility / Constructability	25%	5	1.25	5	1.25
Maintenance and Operation	20%	5	1.00	2	0.40
Staffing Levels	5%	5	0.25	3	0.15
Economic Impact	15%	1	0.15	3	0.45
Environmental and Social Impacts	20%	1	0.20	4	0.80
Total Score Out of 5	100%		3.00		3.65

Solution: The Phase 2 design will include the addition of 3,400 solar panels to the field directly north of the APS substation. The solar power system is designed to be 400 W panels with an efficiency of 19.3%, with an area of 22 square feet (ft^2) which includes hookups and mounting systems [22]. This system would require approximately 85,000 ft^2 to install the system. Phase 3 will include an addition of 1,000 solar panels to the site bringing the total area needed for the panels to 110,000 ft^2. The implementation of solar panels was chosen because of the reduction in energy usage from the grid resulting in significant cost reductions over the life of the system.

4.0 Selection of Design Improvements

4.1 Implementation of Construction and Phasing

The plant was originally constructed with a maximum capacity of 8 MGD. Based off population projections, the influent flow will surpass this capacity by the plant reopening in 2025. Considering the original design as Phase 1, plant expansions will occur in Phase 2 and 3. Phase 2 has a capacity of 20 MGD and construction for Phase 2 will begin in 2023 for the opening in 2025. Phase 2 will reach flow capacity in 2037. Phase 3 has a capacity of 33 MGD and construction for phase 3 will begin 2035 so the expansion can come online in 2037. Phase 3 will reach flow capacity in 2050. The construction phasing for each unit process was calculated using the flow, loading, and the removal efficiency. The expansions also accounted for redundant systems to allow for maintenance. The calculations for each unit expansion are provided in Appendix F. Table 19 summarizes the existing infrastructure for each unit process and the required additions for each phase.

Table 19: Unit Expansion Phasing for CCWRP

Unit Expansion and Phasing			
Unit Process	Phase 1 (Existing)	Phase 2 (Reopening in 2025-2037)	Phase 3 (2037-2050)
Pump Station	4 Centrifugal Pumps	3 12-inch Slurry Pumps: 2 for Flow, 1 for Redundancy	5 12-inch Slurry Pumps: 4 for Flow, 1 for Redundancy
Screening	2 Mechanical Bar Screens	Replace 2 Bar Screens: 1 for Flow, 1 for Redundancy	3 Bar Screens Total: 2 for Flow, 1 for Redundancy
Grit Removal	Nonexistent	2 Vortex Grit Chambers: 1 for Flow, 1 for Redundancy	3 Vortex Grit Chambers: 2 for Flow, 1 for Redundancy
Primary Sedimentation Basin	1 Basin	3 Basins Total: 2 for Flow, 1 for Redundancy	4 Basins Total: 3 for Flow, 1 for Redundancy
Aeration Basin	1 Basin	3 Basins Total: 2 for Flow, 1 for Redundancy	4 Basins Total: 3 for Flow, 1 for Redundancy
Secondary Sedimentation Basin	2 Basins, 1 per Channel	6 Basins Total: 4 for Flow, 2 for Redundancy	7 Basins Total: 6 for Flow, 1 for Redundancy
Tertiary Filter	1 Filter	2 Filters in Parallel	3 Filters in Parallel
UV Disinfection	1 UV System	7 Banks: 6 for Flow, 1 for Redundancy	9 Banks, 8 for Flow, 1 for Redundancy
Desalination	Nonexistent	11 Systems: 10 for Flow, 1 for Redundancy	19 Systems: 18 for Flow, 1 for Redundancy
Solar Panels	Nonexistent	3400 Panels	4400 Panels

4.2 Proposed Staffing Levels

The plant staffing needs were determined using the EPA manual for estimating wastewater treatment facilities staffing requirements [23]. The staffing criterion was modified to account for improved technologies and automation of unit processes. The operation staffing levels were adjusted from 5% to 15% and the associated maintenance requirements were adjusted from 5% to 10%. It was estimated that the plant will require 16 employees working an average of 1,500 hours annually. Table 52 in Appendix I shows the calculated hours and employee requirements for the CCWRP.

5.0 Proposed Cost

5.1 Design Cost

The design cost is the cost of engineering services to design the rehabilitation plan for the plant. The design cost was calculated by multiplying typical rates for engineers on a design team by the hours spent by that position on the design for the rehabilitation of the CCWRP. Table 20 shows the summary of the engineering hours, the billable rate for the position and the cost of services for each position, then totalized to determine the cost of engineering design services. The full breakdown of engineering design firm hours is in Table 56 of Appendix J.

Table 20: Cost of Engineering Design Services for Rehabilitating CCWRP

Cost of Engineering Design Services			
Title	Hourly Rate	Hours Spent on Project	Cost of Services
Senior Engineer	$ 160	172	$ 27,520
Project Engineer	$ 110	211	$ 23,210
EIT	$ 65	221	$ 14,365
Intern	$ 25	162	$ 4,050
Total			$ 69,145

5.2 Unit Process Cost

The cost of each improvement was calculated based on average values for materials and services plus the estimates from manufacturers for the units. The average construction values were determined from Felix Construction services and RSmeans [24]. The prices for each unit were obtained from the manufacturers. Estimations for excavation and concrete were calculated based on square footage and volume needs of the units. Length of pipe and electrical connections were estimated to be 30% and 35% of the total unit cost respectively. The future worth of each unit was calculated with a recommended discount rate of 3% from the US Federal Reserve [25]. A summary of the costs per improvement area is displayed in Table 21 for the present worth and the 2025 future worth and the full unit cost line item sheet is in Table 55 in Appendix J.

Table 21: CCWRP Unit Process Improvement Capital Cost

CCWRP Unit Process Improvement Capital Cost Summary		
Unit	Present Value	2025 Future Value
Influent Pump Station	$ 599,150	$ 787,060
Headworks Building	$ 807,363	$ 964,033
Grit Removal	$ 876,975	$ 1,047,154
Primary Clarifiers	$ 896,775	$ 1,070,796
Aeration Basins	$ 54,450,000	$ 65,016,148
Secondary Clarifiers	$ 1,907,400	$ 2,277,535
Tertiary Filters	$ 3,526,875	$ 4,211,273
UV Disinfection	$ 2,003,100	$ 2,391,806
Desalination	$ 7,590,000	$ 9,062,857
Solar Power	$ 6,920,000	$ 8,262,842

Construction contractor and cost estimates were calculated based on percentages of the unit process cost. The general conditions was 9% of capital cost to cover general contractor supervision, overhead was 5%, and estimated profit for the contractor was 10%, privilege tax was 5.395% and a bond was 1.2% for insurance. Labor costs for the General Contactor were calculated for an 8-person crew and the subcontractor cost was calculated to be 30% of the general contractor cost. The summary of construction costs can be seen in Table 22, the full calculations for the contractor calculations are in Table 57 of Appendix J.

Table 22: Summary of CCWRP Rehabilitation and Expansion Cost

Summary of CCWRP Estimated Construction Cost		
Design Costs	$	69,145
Capital Costs	$	95,091,504
General Contractor Costs	$	34,521,246
Sub-Contractor Costs	$	10,356,374
Contingency	$	32,403,187
Total Cost	$	172,441,456

5.3 Operation and Maintenance Cost

The operations and maintenance costs for each unit were estimated to be 7.5% of the initial 2025 capital cost with an estimated increase for each year after the 1st year service [26]. This cost accounts for the required maintenance for each unit and the increase in maintenance needs for future years due to the age of the equipment. The estimated operations and maintenance cost for each unit can be seen in Table 23 below.

Table 23: CCWRP Yearly Operations and Maintenance Cost

CCWRP Yearly Operations and Maintenance Cost					
Unit	Present Value	2025 Future Value	Initial Maintenance	Yearly Additional Maintenance	Yearly Maintenance
Influent Pump Station	$ 351,000	$ 419,112	$ 31,433	$ 2,619	$ 34,053
Headworks Building	$ 477,250	$ 569,861	$ 42,740	$ 3,562	$ 46,301
Grit Removal	$ 524,000	$ 625,683	$ 46,926	$ 3,911	$ 50,837
Primary Clarifiers	$ 506,000	$ 604,190	$ 45,314	$ 3,776	$ 49,090
Aeration Basins	$ 30,000,000	$ 35,821,569	$ 895,539	$ 74,628	$ 970,167
Secondary Clarifiers	$ 1,056,000	$ 1,260,919	$ 94,569	$ 7,881	$ 102,450
Tertiary Filters	$ 1,996,000	$ 2,383,328	$ 178,750	$ 14,896	$ 193,645
UV Disinfection	$ 1,204,000	$ 1,437,639	$ 107,823	$ 8,985	$ 116,808
Desalination	$ 4,400,000	$ 5,253,830	$ 394,037	$ 32,836	$ 426,874
Solar Power	$ 5,135,000	$ 6,131,459	$ 306,573	$ 25,548	$ 332,121

5.4 Projected Savings

The projected yearly electrical savings for Phase 2 and Phase 3 is $410,000 and $578,000, as a result of the implementation of the solar array on site. The break-even point for the solar power system is 14 years. This includes the cost for both Phase 2 and 3 as well as the estimated operations and maintenance costs for the solar array. After 2050 the solar power system will reduce the plants operating costs of approximately $250,000 a year. This was calculated without accounting for the tax refunds received from generating solar power on site. Table 58 in Appendix J shows the estimated electrical savings.

6.0 Recommendations

Additional innovative improvements are recommended to be added to the plant by Phase 2 and Phase 3, due to the increase in flow. The CCWRP currently does not treat any grit or sludge onsite. Once the grit is removed, it is pumped back to the 91st Avenue Wastewater Treatment Plant (WWTP), as is the sludge. Since the sewer is gravity fed and assumed to not be self-cleaning, the solids buildup will continue to thicken and accumulate causing a greater stress on the 91st Avenue WWTP's equipment and process efficiency. Treating the grit and sludge onsite at the CCWRP would reduce solids accumulation in the sewer and increase process efficiency, and operation and maintenance at the 91st Avenue WWTP. The properly treated solids will be disposed of in a landfill.

All the evacuation tailings generated during the construction Phases 2 and 3 can be stockpiled on site for future use. The City of Phoenix will save money on soil removal and it will also be available for the future projects in the area, both municipal and private.

According to proposed design, once the CCWRP exceeds its capacity all excess raw sewage will be redirected to the 91st Avenue WWTP for processing. This could be an issue as the waste flows produced increases. Proposing an onsite constructed equalization basin will offset the amount of flow that is redirected to the sewer network and the 91st Ave WWTP. This will also work as a safety feature in case the 91st Avenue WWTP reaches full capacity or experiences technical difficulties.

The proposed RO filtration system generates high salinity byproducts. There is no current infrastructure to treat the membrane concentrate because there is currently no salinity removal system. An evaporation pond could be constructed that would be cleaned periodically to remove salt build up. Another option is to construct a saltwater wetlands with halophilic plants, which would create an aesthetically pleasing buffer-zone between the CCWRP and the surrounding community. This will completely remove the excess TDS and will have a positive environmental impact.

Direct potable reuse could be a viable option in the future as the water demand increases, particularly in the southwest. This innovation would be a source of revenue for the CCWRP and is a feasible option due to the proximity of the Union Hills Water Treatment Plant. The largest obstacle in direct potable reuse is convincing the community of its benefits and changing the public perception.

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7.0 Appendices

Appendix A: Existing Site Conditions

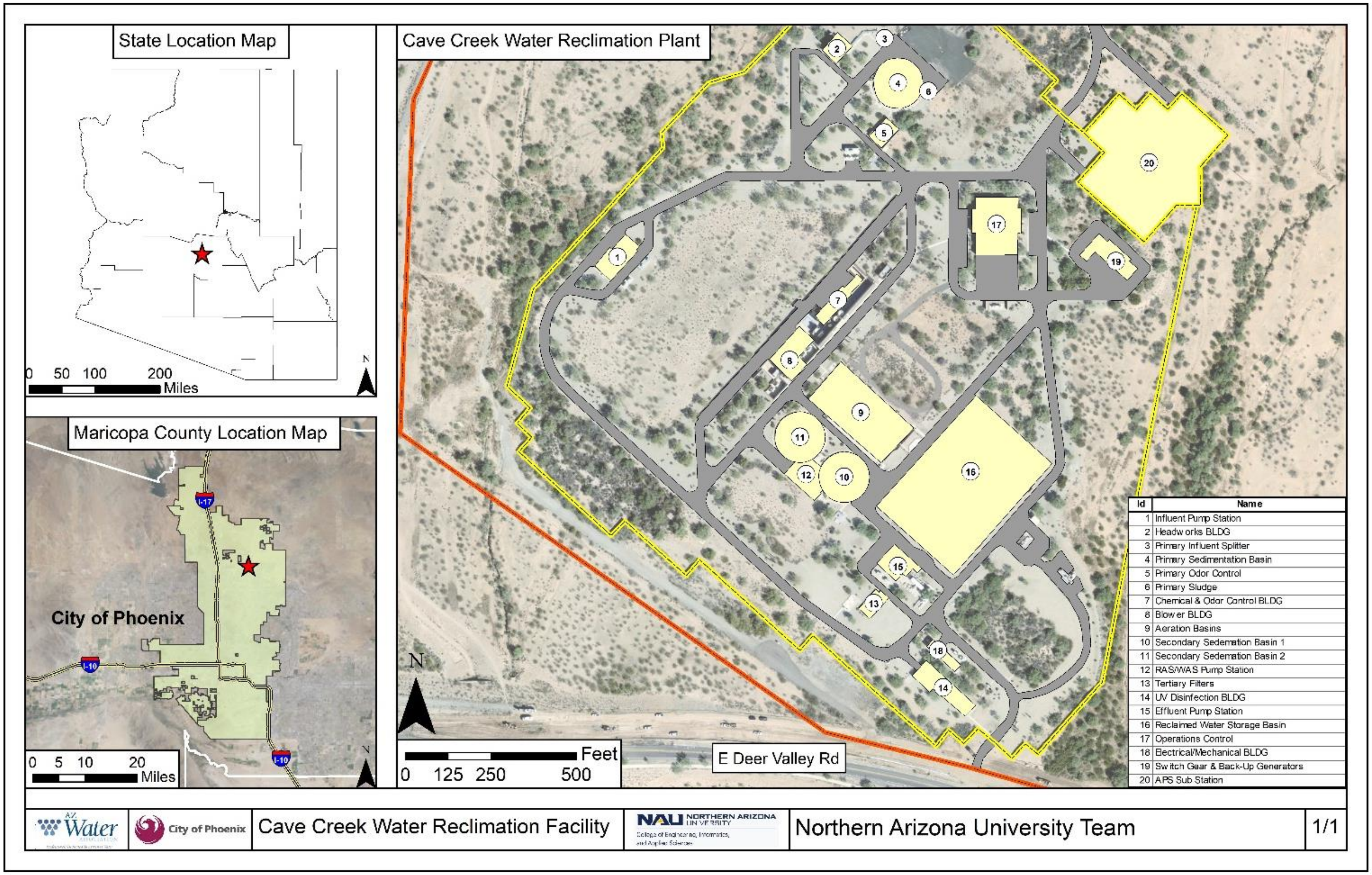


Figure 2: CCWRP Site Map

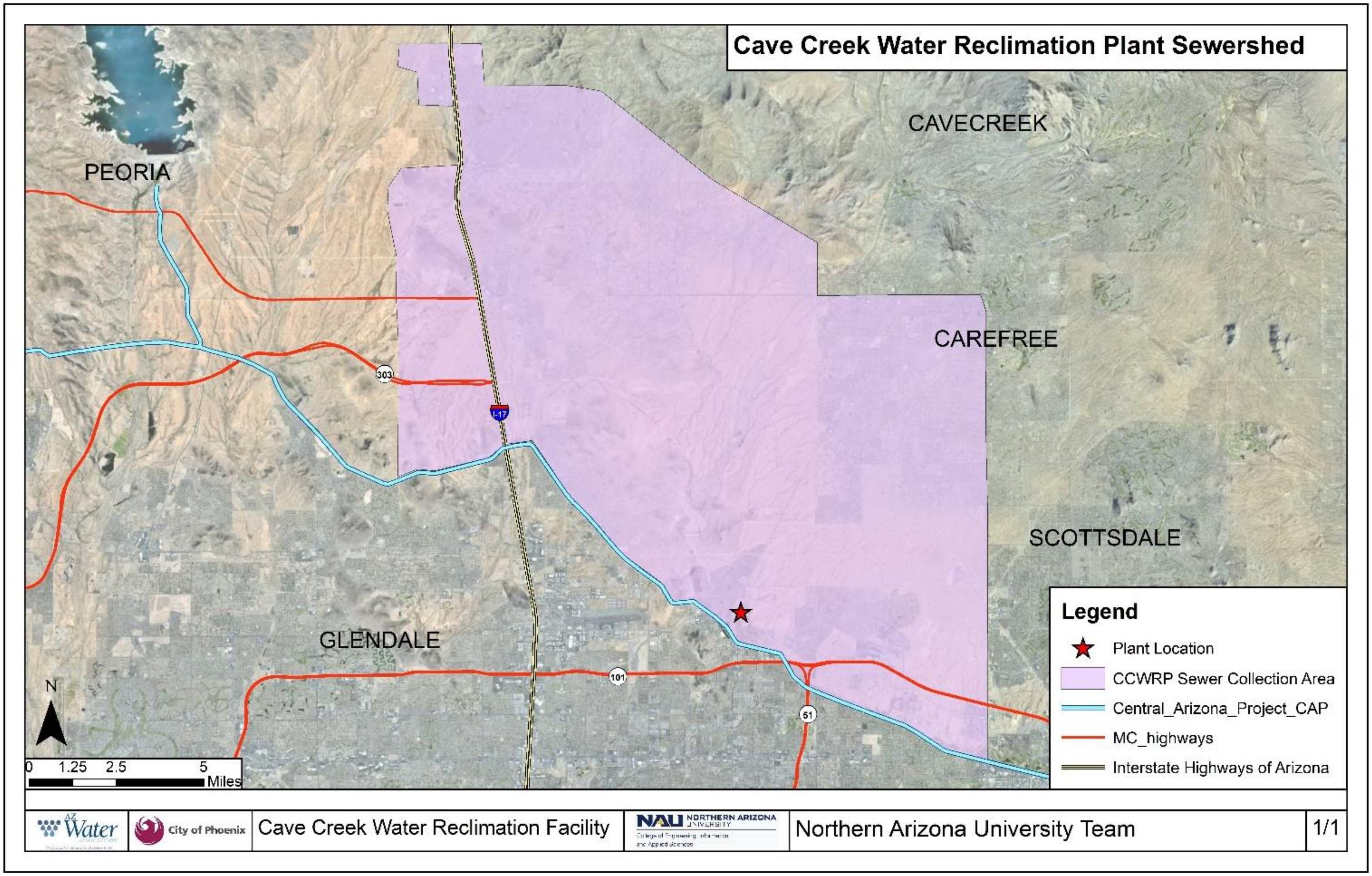


Figure 3: CCWRP Sewershed

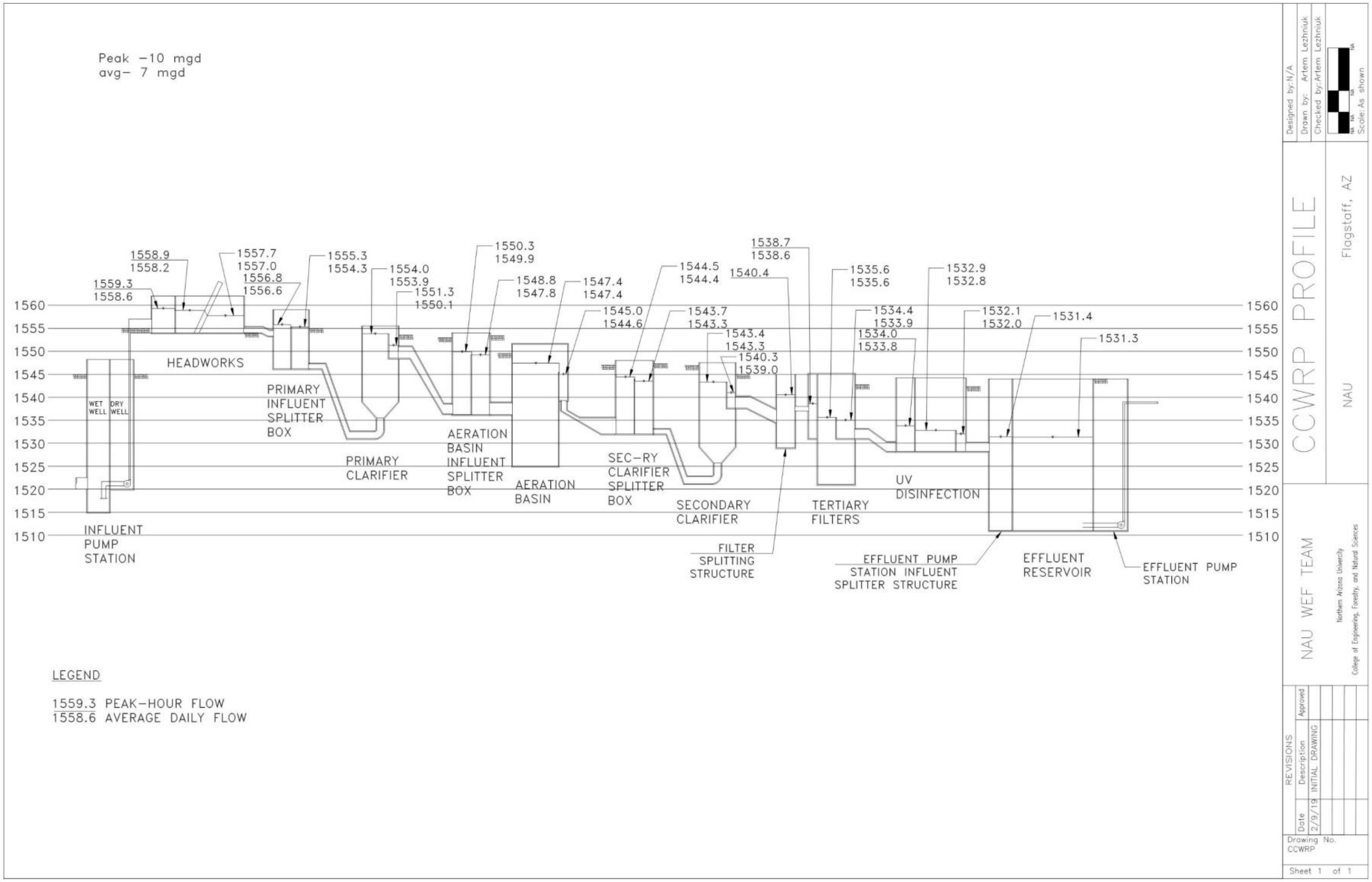


Figure 4: Existing CCWRP Hydraulic Profile

Appendix B: 2017 Flow Data Analysis

Sewer flow data from 2017 was analyzed to determine the historic maximum, average and minimum flows. These flows were then checked with the design standards in the City of phoenix Water and Wastewater Design Manual [3]. The flows matched the peaking factor of 1.85 in the design manual and the design flow equation values.

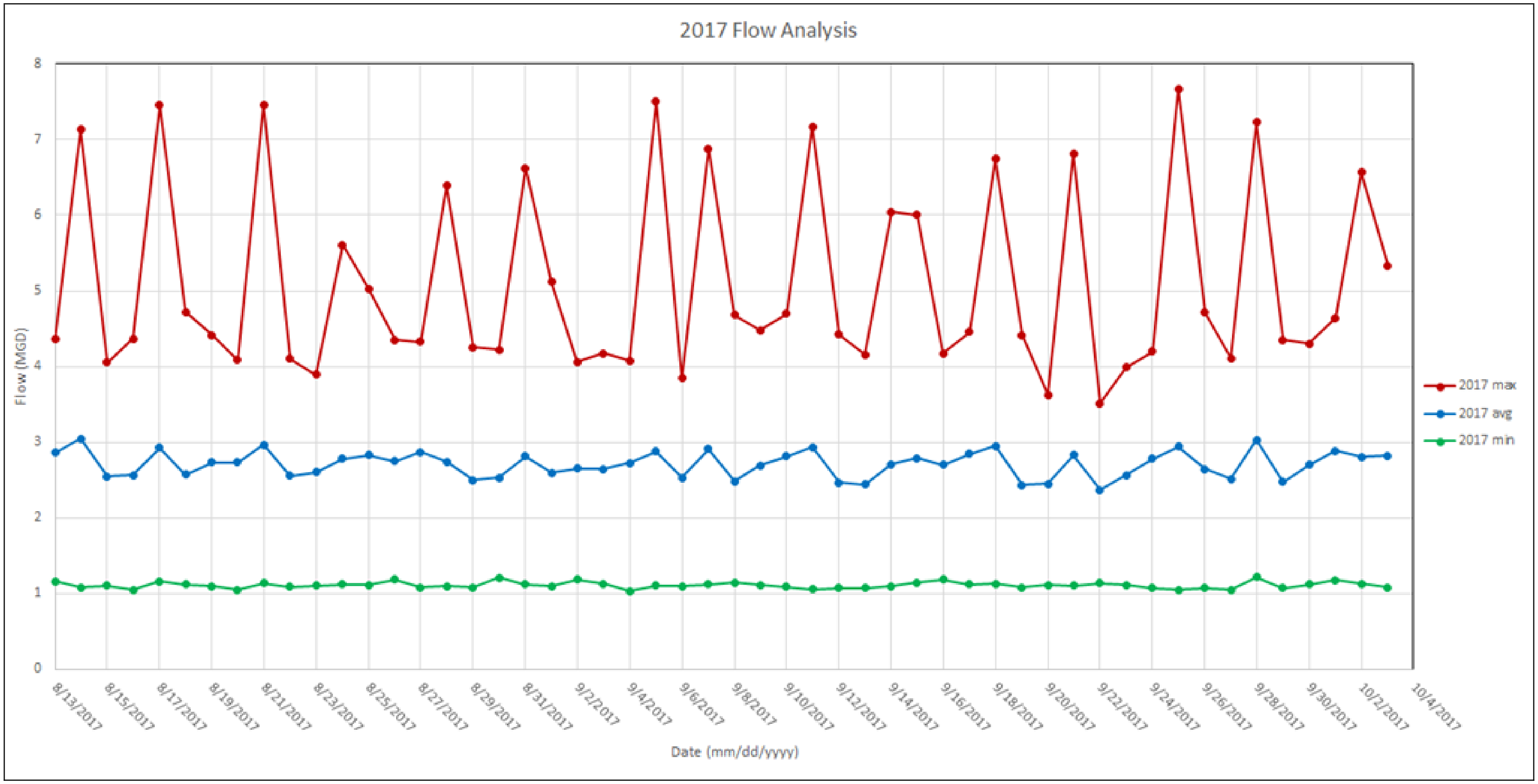


Figure 5: Flow Analysis from 2017

Appendix C: Population Projections

Population Projections: Log Growth Model

Equation 1: Population Growth Rate (Log Growth) [6]

$$P = P_i e^{(r \times t)}$$

Where:

P = Projected Population

P_i = Initial Population

r = Growth Rate

t = Time Interval (years)

Projected population has been modified further by introducing a factor of safety. The factor of safety has a greater value every year to account for any unexpected growth. Projected population of the tributary area to the CCWRP, anticipated growth rates and safety factors are presented in Table 24.

Table 24: Population Projections Measurements using Log Growth Model

Year	Population growth, %	Population	Safety Factor, %	Modified Population
2019	4	75,300	0	N/A
2020	3.9	78,373	1.1	79,235
2021	3.9	81,490	1.2	82,468
2022	3.9	84,731	1.4	85,917
2023	3.85	88,101	1.5	89,422
2024	3.82	91,559	1.7	93,115
2025	3.79	95,124	2	97,026
2026	3.76	98,798	2.2	100,972
2027	3.73	102,584	2.5	105,148
2028	3.7	106,482	3	109,677
2029	3.67	110,496	3.5	114,363
2030	3.64	114,627	4	119,212
2031	3.61	118,876	5	124,820
2032	3.58	123,246	6	130,640
2033	3.55	127,738	5	134,356
2034	3.52	132,354	6	139,671
2035	3.49	137,096	6	145,151
2036	3.46	141,965	6	150,799
2037	3.43	146,963	7	156,618
2038	3.4	152,091	7	162,611
2039	3.37	157,351	7	168780
2040	3.34	162,744	8	175,130
2041	3.31	168,272	8	181,662
2042	3.28	173,935	9	189,589
2043	3.25	179,734	10	197,708
2044	3.22	185,672	11	205,167
2045	3.19	191,748	11	212,840
2046	3.16	197,963	12	221,719
2047	3.13	204,319	13	230,880
2048	3.1	210,815	14	240,329
2049	3.07	217,452	15	250,070
2050	3.04	224,232	17	262,351

Population Projections: Percent Growth Model

Equation 2: Population Growth Rate (Percent Growth) [6]

$$P = P_i((1+r)^n)$$

Where:

P = Projected Population

P_i = Initial Population

r = Growth Rate

n= number of periods

Table 25: Population Projection Measurements using Percent Growth Model

Year	Population growth, %	Population	Safety Factor, %	Modified Population
2019	4	75,300	0	N/A
2020	3.9	78,312	1.1	79,173
2021	3.9	81,288	1.2	82,263
2022	3.9	84,458	1.4	85,641
2023	3.85	87,752	1.5	89,068
2024	3.82	90,955	1.7	92,501
2025	3.79	94,293	2	96,179
2026	3.76	97,698	2.2	99,847
2027	3.73	101,166	2.5	103,695
2028	3.7	104,697	3	107,838
2029	3.67	108,289	3.5	112,079
2030	3.64	111,938	4	116,416
2031	3.61	115,644	5	121,426
2032	3.58	119,403	6	126,568
2033	3.55	123,213	5	129,597
2034	3.52	127,071	6	134,096
2035	3.49	130,973	6	138,669
2036	3.46	134,917	6	143,312
2037	3.43	138,899	7	148,024
2038	3.4	142,915	7	152,800
2039	3.37	146,962	7	157,637
2040	3.34	151,036	8	162,530
2041	3.31	155,132	8	167,477
2042	3.28	159,246	9	173,578
2043	3.25	163,375	10	179,712
2044	3.22	167,512	11	185,101
2045	3.19	171,655	11	190,536
2046	3.16	175,797	12	196,892
2047	3.13	179,934	13	203,325
2048	3.1	184,060	14	209,829
2049	3.07	188,172	15	216,398
2050	3.04	192,263	17	224,948

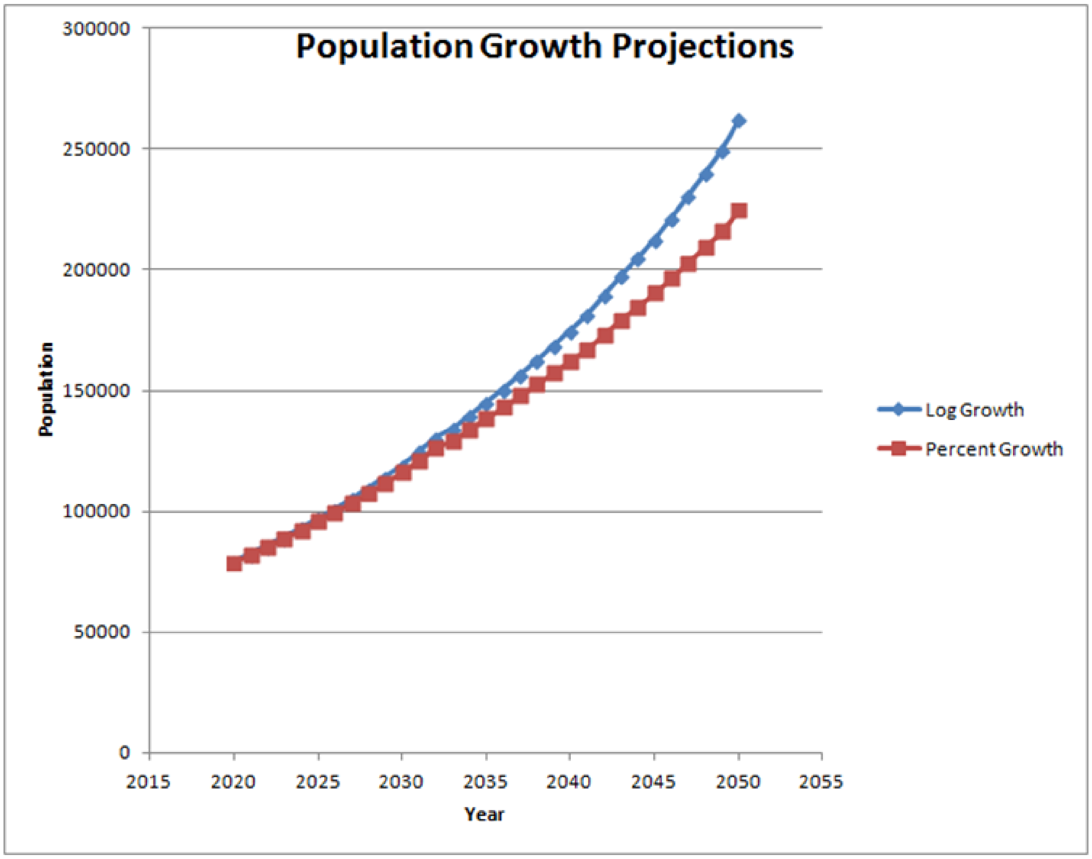


Figure 6: CCWRP Sewer Shed Population Projections

Appendix D: Flow and Load Phasing

Table 26: Complete CCWRP Flow and Loading Phase Estimations

CCWRP Flow and Loading Phase Estimations										
	Year	Population	Q (Gal/cap/day)[a]	Q Total	Q (gal/day) (F.O.S of 1.85)[b]	Q MGD	COD[c] (lb/day)	BOD[d] (lb/day)	TSS[d] (lb/day)	TDS[e] (mg/L)
Phase 1	2008	40,000	87.75	3,510,000	6,493,500	6.49	17,234	9,479	7,163	N/A
	2019	75,300	68.57	5,163,429	9,552,343	9.55	32,443	17,844	13,484	1,587
	2020	79,235	68.57	5,433,257	10,051,526	10.05	34,139	18,776	14,189	1,593
	2021	82,468	68.57	5,654,949	10,461,655	5.65	35,532	19,543	14,768	1,599
	2022	85,917	68.57	5,891,451	10,899,185	10.90	37,018	20,360	15,386	1,605
	2023	89,422	68.57	6,131,794	11,343,819	11.34	38,528	21,190	16,013	1,611
	2024	93,115	68.57	6,385,029	11,812,303	11.81	40,119	22,066	16,675	1,617
Phase 2	2025	97,026	68.57	6,653,211	12,308,441	12.31	41,804	22,992	17,375	1,624
	2026	100,972	68.57	6,923,794	12,809,019	12.81	43,504	23,927	18,082	1,630
	2027	105,148	68.57	7,210,149	13,338,775	13.34	45,304	24,917	18,829	1,636
	2028	109,677	68.57	7,520,709	13,913,311	13.91	47,255	25,990	19,640	1,642
	2029	114,363	68.57	7,842,034	14,507,763	14.51	49,274	27,101	20,480	1,648
	2030	119,212	68.57	8,174,537	15,122,894	15.12	51,363	28,250	21,348	1,654
	2031	124,820	68.57	8,559,086	15,834,309	15.83	53,779	29,579	22,352	1,660
	2032	130,640	68.57	8,958,171	16,572,617	16.57	56,287	30,958	23,394	1,666
	2033	134,356	68.57	9,212,983	17,044,018	17.04	57,888	31,838	24,060	1,672
	2034	139,671	68.57	9,577,440	17,718,264	17.72	60,178	33,098	25,012	1,678
	2035	145,151	68.57	9,953,211	18,413,441	18.41	62,539	34,397	25,993	1,684
	2036	150,799	68.57	10,340,503	19,129,930	19.13	64,973	35,735	27,004	1,690
	2037	156,618	68.57	10,739,520	19,868,112	19.87	67,480	37,114	28,046	1,696

	Year	Population	Q (Gal/cap/day)[a]	Q Total	Q (gal/day) (F.O.S of 1.85)[b]	Q MGD	COD[c] (lb/day)	BOD[d] (lb/day)	TSS[d] (lb/day)	TDS[e] (mg/L)
Phase 3	2038	162,611	68.57	11,150,469	20,628,367	20.63	70,062	38,534	29,119	1,702
	2039	168,780	68.57	11,573,486	21,410,949	21.41	72,720	39,996	30,224	1,708
	2040	175,130	68.57	12,008,914	22,216,491	22.22	75,456	41,501	31,361	1,714
	2041	181,662	68.57	12,456,823	23,045,122	23.05	78,270	43,049	32,531	1,720
	2042	189,589	68.57	13,000,389	24,050,719	24.05	81,686	44,927	33,951	1,727
	2043	197,708	68.57	13,557,120	25,080,672	25.08	85,184	46,851	35,404	1,733
	2044	205,167	68.57	14,068,594	26,026,899	26.03	88,397	48,619	36,740	1,739
	2045	212,840	68.57	14,594,743	27,000,274	27.00	91,703	50,437	38,114	1,745
	2046	221,719	68.57	15,203,589	28,126,639	28.13	95,529	52,541	39,704	1,751
	2047	230,880	68.57	15,831,771	29,288,777	29.29	99,476	54,712	41,345	1,757
	2048	240,329	68.57	16,479,703	30,487,450	30.49	103,547	56,951	43,037	1,763
	2049	250,070	68.57	17,147,657	31,723,166	31.72	107,744	59,259	44,781	1,769
	2050	262,351	68.57	17,989,783	33,281,098	33.28	113,036	62,170	46,980	1,775

[a] The average daily flow per capita is explained in Table 26 , [b] A factor of safety of 1.85 was used as per the City of Phoenix [3] , [c] The average COD and TSS produced daily was determined using historic loading and population data [2]. [d] The BOD was determined using a ratio of BOD:COD of 0.55 [27]. [e] The TDS was estimated using a best fit line of data obtained from CASS, shown in Figure 7 below [4].

Table 27: Determination of Average Flow/Person

Determination of Flow/Person	
Q for Single Family Dwelling (gal/day)	240
Average Residents Per Dwelling	3.5
Q Per Person (gal/day)	68.6

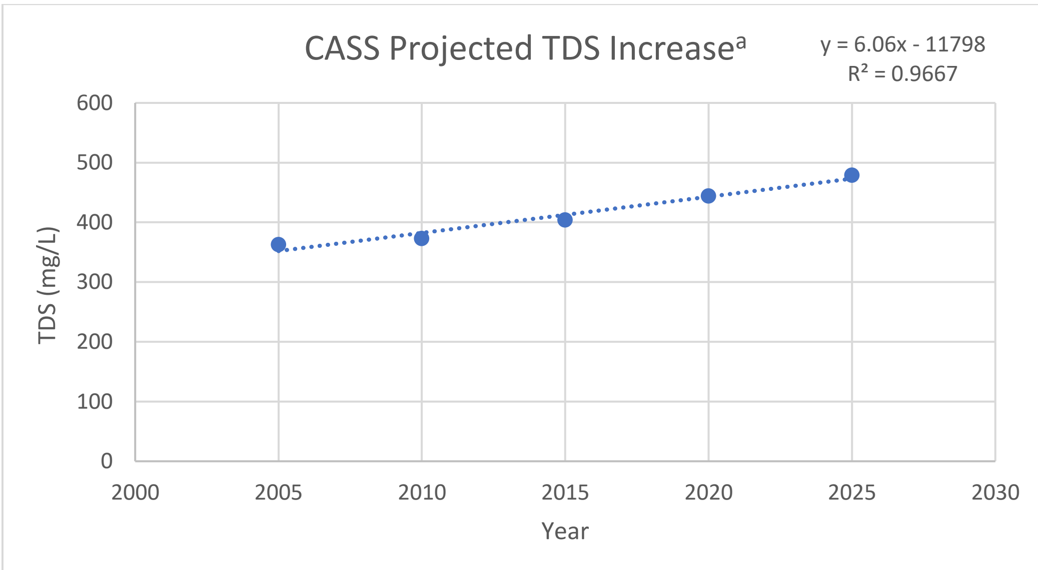


Figure 7-CASS Projected TDS Increase in CCWRP

[a] The TDS increase was added to a concentration of 1150 mg/L, which was the current TDS level at the time of the study [4]

Appendix E: Manufacturer Specifications

Pump Station

EDDY PUMP CORPORATION

TECH SPECS

12-Inch

HEAVY DUTY SLURRY PUMP
12-Inch HD 12000 EDDY Pump Specs. Contact us for details pertaining to your specific job.

OPERATING LEVELS	
MIN FLOW	1800 GPM
MAX FLOW	7000 GPM
HEAD RANGE	40-570 Ft
DISCHARGE SIZE	12 inch
SUCTION SIZE	14 inch
SOLIDS HANDLING	Solids up to 11 inches
MAX SPEED	1800 RPM
PERCENT SOLIDS	Up to 40-70% Solids

Typical Eddy Pumps. Process pumps and dredge pumps can be deployed vertically or horizontally. Contact us for further details. Photos for general guidance.

PARTS	STANDARD MATERIALS
ROTOR	High Chrome 28%, Ductile Iron, Stainless Steel, Duplex Stainless - Various sizes and custom metals available.
VOLUTE CASING	High Chrome 28%, Ductile Iron, Stainless Steel, Duplex Stainless. Custom metals available.
SHAFT	Chromemoly or Stainless Steel
MECHANICAL SEAL	Dual Tungsten or Silicon Carbide Mechanical Seal with Self Contained Seal Flushing System
BEARING HOUSING	Ductile Iron or Stainless Steel

EDDY Pump industrial slurry pumps are non-clog pumps designed for high solids industrial pumping applications. Our patented pump technology outperforms all centrifugal, vortex and positive displacement pumps in a variety of the most difficult pumping applications.

Available in alternative case materials, power options and rotor sizes.

Features and Benefits

- Non-Clog, High Viscosity, High Specific Gravity, High Abrasives, Low pH Pumping Design
- Transport 40-70% Solids
- Ability to pump objects of up to 9-inches in diameter
- 100% American Built

Applications

- Mining
- Wastewater
- Chemical
- Sand & Agg
- Oil and Gas
- Paper & Pulp
- Fly Ash & Coal Ash

Fluid Pumped

- Sludge
- Slurry
- Drilling Mud
- Mine Tailings
- Grit
- Paste

We Pump Solids Not Water

Eddy Pump | El Cajon, CA 92021 USA | EddyPump.com| Phone: (619) 258-7020 | Fax: (619) 258-0305

HD12000 V2.2

Figure 8: EDDY Heavy Duty 12-Inch Slurrt Pump Specification

Pump Data Sheet - Eddy Pump

Company: NAU.EDU
Name: Katherine Dougherty
Date: 02/21/2019

10MGD (6,945 gpm)
0 horizontal, 40 ft. vertical

EDDYPUMP CORPORATION

Pump:

Size:	HDX12000	Dimensions:	Suction:	14 in
Type:	Heavy Duty		Discharge:	12 in
Synch Speed:	900 rpm			
Dia:	17 in			
Curve:	---			

Fluid:

Name:	horse hair		
SG:	1.3	Vapor Pressure:	1 psi a
Density:	81 lb/ft³	Atm Pressure:	14.7 psi a
Viscosity:	2 cP	NPSHa:	15.6 ft
Temperature:	50 °F		

Pump Limits:

Temperature:	---	Sphere Size:	---
Wkg Pressure:	---		

Search Criteria:

Flow:	6945 US gpm	Near Miss:	---
Head:	58.55 ft	Static Head:	0 ft

Motor:

Standard:	NEMA	Size:	200 hp
Enclosure:	TEFC	Speed:	900 rpm
Frame:	449T		
Sizing Criteria:	Max Power on Design Curve		

Pump Selection Warnings:

Selected motor does not meet initial motor sizing criteria.
Catalog does not contain data to verify that NPSHa is sufficient.

--- Duty Point ---

Flow:	6949 US gpm
Head:	58.6 ft
Eff:	52.6%
Power:	252 hp
NPSHr:	---
Speed:	993 rpm

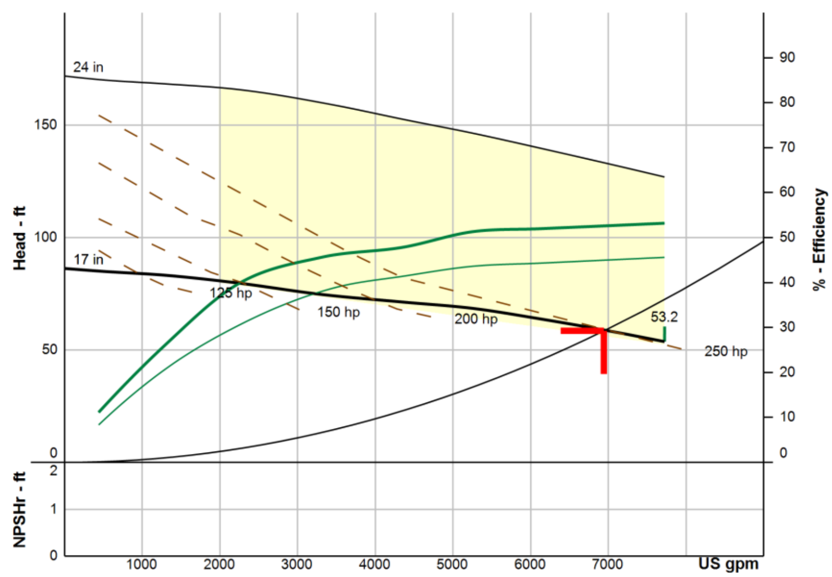
--- Design Curve ---

Shutoff Head:	86.3 ft
Shutoff dP:	48.5 psi
Min Flow:	--- US gpm

BEP: 53.2% @ 7723 US gpm
NOL Power:
256 hp @ 7723 US gpm

--- Max Curve ---

Max Power:
706 hp @ 7723 US gpm



Please contact Eddy Pump to accommodate selections outside of the normal operating range.

Performance Evaluation:

Flow	**Speed**	**Head**	**Efficiency**	**Power**	**NPSHr**
US gpm	rpm	ft	%	hp	
8334	993	---	---	---	---
6945	993	58.6	52.6	251	---
5556	993	66.9	51.5	236	---
4167	993	71.9	47.5	207	---
2778	993	77.2	42.6	163	---

Selected from catalog: Eddy Pump.60, Vers 1.3

Figure 9: Pump Curve for the EDDY Heavy Duty 12-Inch Slurry Pump

Bar Screens

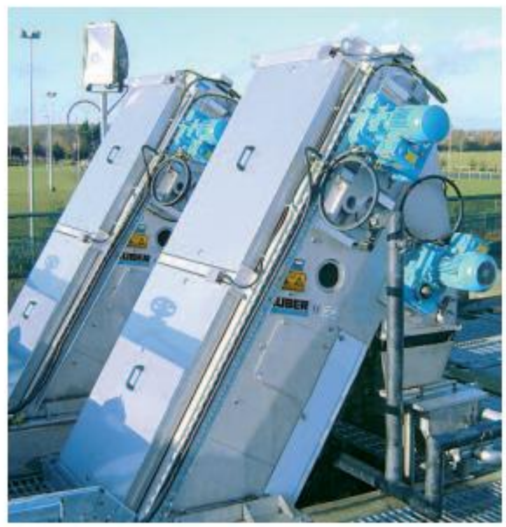
HUBER TECHNOLOGY
WASTE WATER Solutions

Benefits

- Excellent capture rate
- Reliable cleansing with a single rotating brush
- No need for additional drive cleansing elements
- Maintenance-free bearings. Submerged bearings are made out of wear-resistant ceramic
- Compact design and small height above operating floor
- Completely enclosed screen with easily removeable covers
- No odor nuisance
- Easy retrofitting into existing channels, no need for recesses in their walls and bottom
- Self-supporting and liftable stainless steel frame
- No impairment by gravel or grit
- Simple and easily accessible chain tensioning system
- All wetted components are made of stainless steel and passivated in a pickling bath (except chains, drives, bearings).
- Chains and cog wheels are also available in stainless steel

Installation examples

Rear view of an EscaMax® with Wash Press WAP



Fully enclosed for odor control

Dimensional Data

- Discharge height: up to 26'
- Channel width: up to 7'- 2"
- Perforation: 3.5, 6, 8 or 10 mm
- Installation angle: 45° - 70°

Figure 10: Huber Technology EscaMax Perforated Plate Bar Screen Specifications

Vortex Grit Chamber

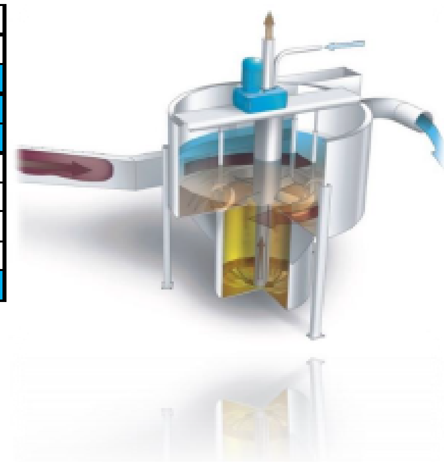
Scope of Supply

HUBER TECHNOLOGY

Vormax Design Information

Vormax Technical Data

Peak Design Flow	20	*MGD*
Maximum Flow Capacity	20	*MGD*
Removal Efficiency @ maximum Flow	95% greater than 50 mesh	
	85% greater than 70 mesh	
	65% greater than 100 mesh	
Headloss @ Max. Flow	1.7	*inches*
Upper Chamber diameter	16	*feet*
Lower Chamber diameter	5	*feet*
Design Orientation	270	*degree*
Pump Mounting	Top	



Vormax Details

Model	***Size 6***
Quantity	1
Material	304L Stainless Steel Construction; picked and passivated in acid bath
Gear Box	Enclosed bull gear in a heavy cast iron casing Gear reducer to include anti-friction bearings with high overhung load properties, and double lip temperature oil seals
Drive Shaft	304L stainless steel turning drive shaft, driven by the bull gear
Drive Motor	1.5 HP, Class 1 Division 1, 480 VAC, 3 phase, 60 Hz
Paddles	Mounted on the drive shaft. The paddles are adjustable in all directions, inter-lockable with counter screws.
Floor Plate	Two piece floor plate for separation of upper and lower chamber
Anchor Bolts	M12 316L, Included
Control Panel(s)	NEMA 4X Stainless Steel Enclosure, Allen Bradley MicroLogix PLC, Allen Bradley PanelView Plus 800 Color Touch OIU, Huber Standard Components, Preprogrammed and Factory Tested
LCS	Included, NEMA7 LCS

Note *Concrete grit basin by others.*

Figure 11: Huber Technology Vortex Grit Chamber Specification

Primary Clarifier

WesTech
Advanced Clarifier Design - C.O.P.™
WesTech Project No. 1960178 Page 1

Checked: ________

Date: ___________

PROJECT INFORMATION

Run Date:	3/26/2019
WesTech No.:	1960178
Project:	Cave Creek
Customer:	-
Engineer:	-
Run By:	OL17
Units:	English
Application:	Municipal
Clarifier Type:	Wastewater Primary
Configuration:	Column supported

INPUT DATA

Avg. Effluent Flow(MGD):	8.0*
Design Effl. Flow(MGD): (Max. Month)	11.0*
Max. Effluent Flow(MGD): (Max. Day)	17.0*
Peak Effluent Flow(MGD):	20.0
No. of Clarifiers:	2
Sludge Removal:	Spiral Rakes
Sludge Withdrawal Ring:	None

* This is an ASSUMED value. This data should be verified before using program output for design.

PROGRAM OUTPUT

Item	Value	Comments
BASIN GEOMETRY		
Tank Diameter (ft):	110.0	
Side Water Depth (ft):	16.0	
Floor Slope (in/ft):	1.0	
Flat Diameter (ft):	3.67	
Overflow rate @ average flow (gal/ft2.day):	420.91	
Overflow rate @ design flow (gal/ft2.day):	578.75	
Overflow rate @ max. flow (gal/ft2.day):	894.42	
Overflow rate @ peak flow (gal/ft2.day):	1052.26	
CENTER COLUMN		
Column Outside Diameter (in):	30.0	
Number of ports:	4	
Port Width (in):	11.5	
Total Port Height (in):	24.0	(includes 3" freeboard)

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Figure 12: Primary Clarifier Design Specifications

Secondary Clarifier

Item A – (5) 110' Diameter Clarifier Mechanisms Model COPC1G

General Scope of Supply

Item	Unit	Value/Description
Number of Mechanisms	Each	5
Application	-	Activated Sludge Secondary
Tank Diameter	ft	110
Tank Side Wall Depth	ft	17.5
Tank Side Water Depth	ft	15.5
Tank Bottom Slope	-	1:12
Average Flow Rate	MGD	17.8
Design Flow Rate	MGD	20*
Peak Flow Rate	MGD	32
Influent MLSS Concentration	mg/L	3000*

Detailed Scope of Supply

Item	Unit	Qty	Size/Description	Material
Walkway Bridge	each	1	Pony Truss Type	Steel
Walkway Handrail	-	-	Truss serves as railing	Steel
Walkway Flooring	-	-	1-1/4" Grating	Aluminum
Platform Handrail	-	-	2 Rail Component	Aluminum
Platform Flooring	-	-	1/4" Checker Plate	Aluminum
Center Column Diameter	in	1	42	Steel
Dual-Gate EDI Diameter	ft	1	10	Steel
Dual-Gate EDI Total Height	ft	-	3.5	
Feedwell Diameter	ft	1	24	Steel
Feedwell Total Height	ft	-	6	
Feedwell Supports	-	-	Supported from the Cage	Steel
Full Radius Rake Arms	-	2	Box truss w/ spiral scrapers	Steel
Sludge Withdrawal Ring	-	1	20% of tank dia. w/ multiple ports	Steel
Squeegees	-	-	Bolted to scraper blades	304 SS
Scum Skimmer	each	2	Std. hinged skimmer blade	304 SS
Scum Box	each	1	5' Standard scum box	Steel
Scum Flushing Valve	each	1	Skimmer actuated	Polymer/SS
Anchor Bolts & Fasteners	-	-	-	304 SS

WesTech

Proposal No. 1960178

Figure 13: Secondary Clarifier Design Specifications

Tertiary Treatment

Item A – SuperSand™ Filtration System, Model Number SS8S41

WesTech is pleased to offer a SuperSand filtrating system consisting of 4 filters in 16 basins. Each filter basin will include internals, piping, valves, air control panels, grating, and media. Concrete basins to be supplied by others.

General Scope of Supply

Description	Unit	Dimension/Capacity
Application	-	TSS Reduction
Number of Basins	each	16
Filters per Basin	each	4
Design Flow	MGD	8.0
Peak Flow	MGD	14.8
Filter Area per Filter	ft^2	50
Design Loading Rate	gpm/ft^2	1.7
Peak Loading Rate	gpm/ft^2	3.2

Detailed Scope of Supply per Filter

Item	Unit/Size	Quantity	Description	Material
Silica Sand	in	80	1.2-2.0 mm and UC ≤ 1.65	-
Washbox	each	1	Sand washer	HDPE
Air-Lift Pump	each	1	-	HDPE
Distribution Radial and Cone	each	1	-	304SS
Anchor Bolts	-	-	-	304SS
Bottom Cone	each	1	-	FRP

Detailed Scope of Supply per Basin

Item	Unit/Size	Quantity	Description	Material
Influent Valve	each	1	Bray Manual Butterfly with operator	Ductile Iron
Washbox and Grating Supports	each	1	C-Channel with Brackets	304 SS
Air Control Panel	each	1	NEMA 4X	304SS
Level Switch	each	1	Chicago Sensor	Stainless Steel
Influent Piping	each	1	-	HDPE
Waste Piping	each	1	4" header and lateral to each module	PVC
Drain Piping	each	1	4" header and lateral to each module	PVC

Figure 14: Tertiary Treatment Design Specifications

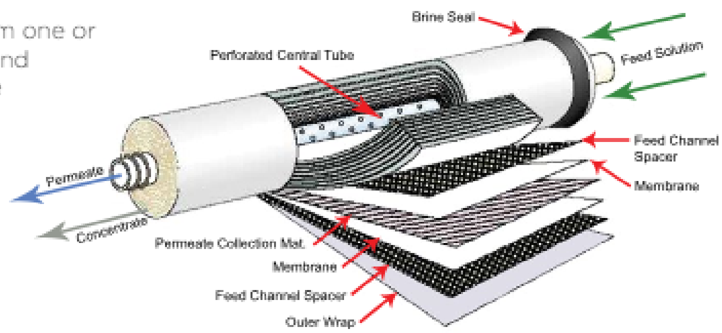
Reverse Osmosis Membrane

Industrial Brackish RO Systems

Capacity: 200,000 to 900,000 GPD

RO-500 SERIES

The spiral membrane is constructed from one or more membrane envelopes wound around a perforated central tube. The permeate passes through the membrane into the envelope and spirals inward to the central tube for collection. The layers of the membrane envelope are detailed in the diagram to the right.



Operation Specifications

- Max. feed water temperature: 42°C
- Feed water pressure: 20 to 80 psi
- Operating pressure: 150 to 250 psi
- Hydrogen Sulfide must be removed
- Turbidity should be removed
- Max. iron content: 0.05 ppm
- Feed water TDS: 0 to (1,000 or 3,000 or 5,000 ppm)
- Equipment upgrade for TDS over 5,000 ppm
- Hardness over 1 GPG requires antiscalant dosing
- pH tolerance range: 3-11
- Max. Silica Tolerance: 60 ppm @ 60% recovery
- Operate at higher TDS by lowering recovery

Model #	Permeate Flow Rate		Quantity of 8" Membranes	Motor Rating at 1,000 ppm		Approx. Weight (lbs)	Dimensions L"xW"xH"
	GPD	M³/D		60 Hz (hp)	50 Hz (kw)		
TW-200K-4780	200,000	758	28	30	22	4,700	350x72x80
TW-225K-5680	225,000	852	30	30	22	4,850	300x72x80
TW-270K-6680	270,000	1,023	36	40	30	5,050	300x72x80
TW-320K-7680	320,000	1,212	42	40	30	5,200	300x72x90
TW-360K-8680	360,000	1,364	48	50	37	5,750	300x72x90
TW-410K-9680	410,000	1,553	54	60	37	6,250	300x72x90
TW-450K-10680	450,000	1,705	60	60	45	7,500	300x72x90
TW-500K-11680	500,000	1,894	66	60	45	8,500	300x72x80
TW-550K-11780	550,000	2,083	77	75	45	8,750	350x72x90
TW-600K-13780	600,000	2,273	91	75	55	9,250	350x84x92
TW-700K-14780	700,000	2,652	98	100	75	9,650	350x84x94
TW-800K-16780	800,000	3,030	112	2X60	2x37	10,200	350x84x96
TW-900K-18780	900,000	3,409	126	2X60	2x37	10,650	350x84x98

Note: If the feed water TDS exceeds 1,000 ppm, the system model number changes to BW-XXXK-XXXX, and a suffix is added to the end of the model number: "-3" is added if the TDS is 3,000 ppm or less, and "-5" is added if the TDS is 5,000 ppm or less.

Example: Required system to produce 320,000 GPD with a feed water TDS of 5,000 ppm, the corresponding model number is: "BW-320K-7680-5".

Pure Aqua also supplies: Custom Engineered Solutions, Multimedia Pretreatment, Activated Carbon Pretreatment, Water Conditioning, Chemical Dosing Systems, Ultraviolet (UV) Sterilizers and Ozonation Systems.

Figure 15: Reverse Osmosis Manufacturer Specifications

UV Disinfection Specification

Table 28: Manufacturer Specification Sheet for the Trojan Signa UV Disinfection System

System Specifications	
System Characteristics	**TrojanUVSigna**
Lamp Type	TrojanUV Solo Lamp (amalgam)
Lamp Driver	Electronic, high-efficiency (99% power factor)
Input Power Per Lamp	1000 Watts
Lamp Control	30 - 100% variable lamp power (1% increments)
Lamp Configuration	Staggered, inclined array (two-row, four-row or six-row)
Module/Bank Frame	Type 6P (IP67)
Ballast Enclosure	Type 4X (IP66)
Cleaning System	Automatic ActiClean chemical/mechanical
UV Intensity Sensor	1 per bank – with automatic chemical cleaning
Bank Lifting Device	1 per bank - Automatic Raising Mechanism (ARM)
Level Control Device	Fixed weir or motorized weir gate
Water Level Sensor	High and low water level sensors available (one per channel)
Installation Location	Indoors or outdoors
System Control Center	Standard color HMI, 16 digital I/O, 4 analog I/O, SCADA compatible PLC options available

Appendix F: Unit Expansions

Pump Station

The elevation at the pump station is 1522 feet. The influent is pumped to elevation of 1,560 feet, where it is moved through an 800-foot pressurized pipeline to the headworks building. The pipe material and diameter were assumed to be PVC and 14-inches. The system curve was developed for the facilities varied flow rates for Phase 2 and Phase 3. The pump efficiency and pump curve for the 12-inch pump was plotted against the system curve to determine the pump operational point (Figure 16). The total dynamic head was computed by adding the calculated friction losses, minor losses, change in elevation, the headloss through the bar screens, channels, and grit chamber. The friction and minor headlosses were computed by using Equation 3 through Equation 6. Table 29 displays the system curve calculations.

Table 30 displays the flows rates, head, and efficiencies for the 12-inch pump. The final design for Phase 2 will incorporate two 12-inch slurry pumps, plus one additional pump to allow for redundancy. Phase 3 will require adding two additional pumps, to support the predicted flow rate of 33 MGD and for system redundancy.

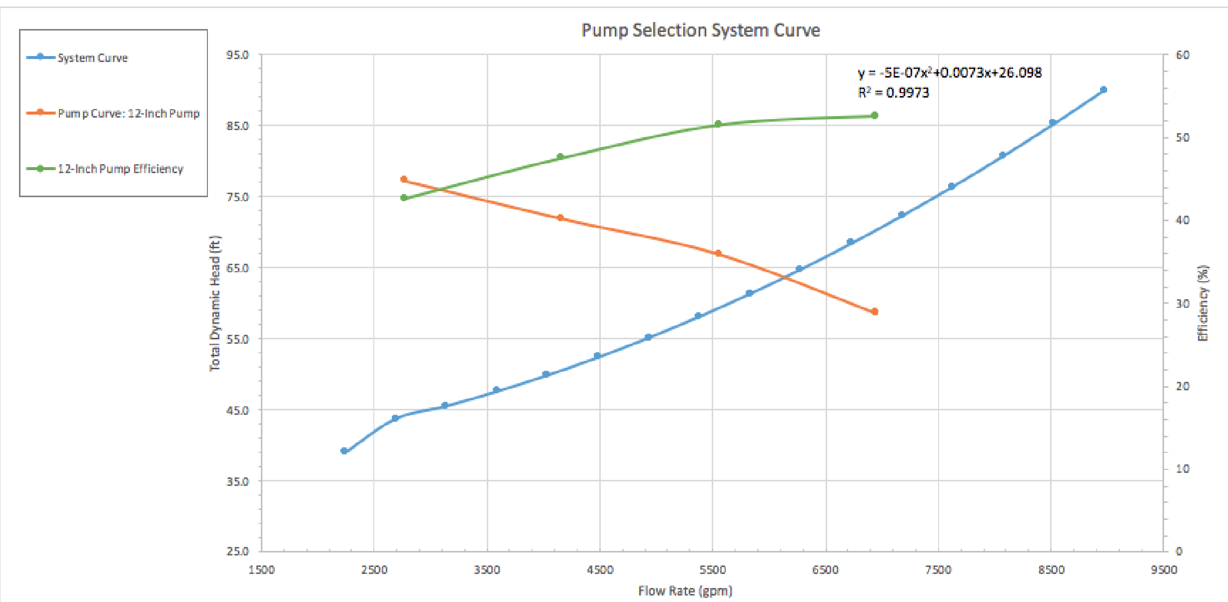


Figure 16: System Curve, 12-Inch Pump Curve, and 12-Inch Efficiency Curve

Equation 3: Swamme Jain [28]

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

Where:

f = Darcy Weisbach Friction Factor

D = Diameter (ft)

k_s = Pipe Roughness

R_e = Reynold's Number

Equation 4: Friction Losses [28]

$$h_{L_f} = f\frac{L}{D}\frac{V^2}{2g}$$

Where:

h_{Lf} = Friction Loss (ft)

V = Velocity (ft/s)

g = Gravitational Constant (ft/s^2)

Equation 5: Minor Losses [28]

$$h_{L_m} = K\frac{V^2}{2g}$$

Where:

h_{Lm} = Minor Headloss (ft)

K = Minor Loss Coefficient

Equation 6: Total Dynamic Head [28]

$$TDH = h_{L_f} + h_{L_m} + \Delta z + h_{L_{bar\,screens+channel+grit\,chamber}}$$

Where:

TDH = Total Dynamic Head (ft)

Δz = Change in Elevation (ft)

Table 29: Converted Flow Measurements and Total Dynamic Head for System Curve

Q (cfs)	Q (gpm)	Q (MGD)	TDH (ft)
5	2244	3	39.0
6	2693	4	43.7
7	3142	5	45.5
8	3591	5	47.6
9	4039	6	49.9
10	4488	6	52.4
11	4937	7	55.1
12	5386	8	58.1
13	5835	8	61.3
14	6284	9	64.8
15	6732	10	68.4
16	7181	10	72.3
17	7630	11	76.4
18	8079	12	80.7
19	8528	12	85.2
20	8977	13	90.0

Table 30: 12-Inch Pump Curve and Efficiency Curve

Pump and Efficiency Curve (12-inch Heavy Duty Slurry Pump)		
Q (gpm)	Head (ft)	Efficiency (%)
8334	--	--
6949	58.6	52.6
6945	58.6	52.6
5556	66.9	51.5
4167	71.9	47.5
2778	77.2	42.6

Headworks

The existing channels leading to the bar screens has a maximum depth and width of 7 feet and 2 feet. The channel allows for 2 feet of freeboard, providing a maximum flow depth of 5 feet. However, to support a functioning HGL, the channel elevation was raised by 2 feet, increasing the maximum depth to 9 feet. This design accounts for 2 feet of freeboard, which provides a maximum depth of flow at 7 feet.

The cross-sectional flow area of the channel was measured by multiplying the maximum flow depth by the channel width. The maximum allowable velocity in the channel for Phase 2 and Phase 3 was computed by using Manning's Equation (**Error! Reference source not found.**). However, the slope and channel roughness were assumed to be 0.0007 and 0.012, respectively.

Equation 7: Manning's Equation [29]

$$Q = VA = \left(\frac{C}{n}\right) AR^{\frac{2}{3}}\sqrt{S}$$

Where:

V = Velocity (ft/s)

C = 1.49 $ft^{1/3}/s$

n = Mannings Roughness Coefficient

R = Hydraulic Radius (ft)

S = Slope

The channel reconstruction was determined by computing the headloss through the existing headworks channel (Equation 8), proposed bar screens, grit chamber, and the primary splitter box. The headloss was summed and then subtracted by the existing bar screen headloss of 1.2 feet, respectively. The HGL profile elevations were adjusted for the improved preliminary units. The headworks channel elevations are required to be raised by at least 1.5 feet to ensure constant flow. However, for the expansion of the plant, the channel operating floor will be raised to 2 feet. Table 31 displays the headloss computations that were utilized for the HGL adjustment and channel reconstruction.

Equation 8: Headloss in Channel [29]

$$h_L = L \times S$$

Where:

h_L = Headloss (ft)

L = Channel Length (ft)

S = Slope (ft/ft)

Table 31: Computed Headloss for HGL Profile Adjustments and Channel Reconstruction

HGL Adjustments and Channel Reconstruction	
Headloss through Existing Bar Screens (ft)	1.2
Headloss through Headworks Channel (ft)	0.06
Headloss through Grit Chamber (ft)	0.20
Headloss through Proposed Bar Screens (ft)	2.25
Headloss from Primary Splitter Box to Headworks (ft)	0.80
Total Headloss Before Adjustment (ft)	2.1
Total Adjusted Headloss (ft)	1.5

The required bar screen expansion was determined by calculating approach velocities for both phases. The approach velocity was determined by dividing the peak flow rate by the cross-sectional area of the channel (Equation 9). The velocity through the bar screens and headloss was computed from Equation 10 and 11.

Equation 9: Continuity Equation [29]

$$Q = VA$$

Where:

Q = Design Flow Rate (cfs)

A = Cross-Sectional Area of Channel (ft^2)

Equation 10: Continuity Equation for Velocity Through Bar Screens [30]

$$V_b = \frac{V_a \times A_a}{A_{net}}$$

Where:

V_b = Velocity Through Bar Screen (ft/s)

V_a = Maximum Velocity in Channel (ft/s)

A_a = Flow Aera of Channel (ft_2)

A_{net} = Net Area of Bar Screen (ft^2)

Equation 11: Headloss through Bar Screens [29]

$$h_L = \frac{\left(0.7\left(V_{thru}^2 - V_{approach}^2\right)\right)}{2g}$$

Where:

V_{thru} = Velocity through Bar Screens (ft/s)

$V_{approach}$ = Approach Velocity (ft/s)

The final design for Phase 2 will require two bar screens to support the varied flows and predicted peak flow of 20 MGD, as one will be used for redundancy. Phase 3 will require constructing one additional channel with a width and maximum channel depth of 2 feet and 9 feet. Phase 3 will utilize both bar screens that were implemented in Phase 2. To allow for redundancy, one additional bar screen will be added to the constructed channel. Table 32 and Table 33 display the measurements for required expansion.

Table 32: Phase 2 – Headworks Expansion Computations

Phase 2: Bar Screen Influent Parameters for 2025-2037	
Peak Flow (MGD)	20
Flow (cfs)	30.9
Channel Dimensions	
Channel Width (ft)	2
Freeboard (ft)	2
Channel Depth (ft)	9
Maximum Channel Water Depth (ft)	7
Cross-Sectional Channel Flow Area (ft^2)	14
Headloss in Channel (in)	0.70
Phase 2: Bar Screen Expansion for 2025-2037	
Maximum Design Velocity for One Channel (ft/s)	2.21
Maximum Design Velocity for Two Channels (ft/s)	1.11
Velocity Through Bars for One Channels, One Screen (ft/s)	2.23
Redundancy: Velocity Through Bars for Two Channels, Two Screens (ft/s)	1.12
Headloss Through Bar Screen (in)	0.89

Table 33: Phase 3 – Headworks Expansion Computations

Phase 3: Bar Screen Influent Parameters for 2037-2050	
Peak Flow (MGD)	33
Flow (cfs)	51.1
Channel Dimensions	
Channel Width (ft)	2
Freeboard (ft)	2
Channel Depth (ft)	9
Maximum Channel Water Depth (ft)	7
Cross-Sectional Channel Flow Area (ft^2)	14
Headloss in Channel (in)	0.70
Phase 3: Bar Screen Expansion for 2037-2050	
Maximum Design Velocity for One Channel (ft/s)	3.65
Maximum Design Velocity for Two Channels (ft/s)	1.83
Maximum Design Velocity for Three Channels (ft/s)	0.91
Velocity Through Bars for One Channels, One Screen (ft/s)	4.74
Velocity Through Bars for Two Channels, Two Screens (ft/s)	2.37
Redundancy: Velocity Through bars for Three Channels, Three Screens (ft/s)	1.18
Headloss Through Bar Screen (in)	2.44

Grit Removal

The Huber Vortex Grit Chamber is specified to hold 20 MGD. The capacities of each unit were compared and appropriately duplicated based on the maximum design flow. For Phase 2, the facility will need to implement two units of the vortex grit chamber to support the maximum design flow of 20 MGD and allow for redundancy. For Phase 3, the facility will need to add one additional vortex grit chamber to support the maximum design flow rate of 33 MGD and allow for redundancy.

Table 34: Phase 2 - Grit Chamber Expansion

Phase 2: Vortex Grit Chamber Influent Parameters	
Flow (MGD)	20
Flow (cfs)	30.9
BOD (lb/day)	37,113.89
COD (lb/day)	67,479.81
TSS (lb/day)	28,046.30
TDS (mg/L)	780.95
Phase 2: Vortex Grit Chamber Expansion	
Maximum Flow Rate (MGD)	20
One Unit (MGD)	20
Redundancy: Two Units (MGD)	40
Phase 2: Vortex Grit Chamber Effluent Parameters	
Flow (MGD)	20
Flow (cfs)	30.9
BOD (lb/day)	37,113.89
COD (lb/day)	67,479.81
TSS (lb/day)	28,046.30
TDS (mg/L)	780.95

Table 35: Phase 3 - Grit Chamber Expansion

Phase 3: Grit Chamber Influent Parameters for 2037-2050	
Flow (MGD)	33
Flow (cfs)	51.1
BOD (lb/day)	62,169
COD (lb/day)	113,035.51
TSS (lb/day)	46,980.38
TDS (mg/L)	941.50
Grit Chamber Expansion for 2037-2050	
Maximum Flow Rate (MGD)	33
One Unit (MGD)	20
Two Units (MGD)	40
Redundancy: Three Units (MGD)	60
Phase 2: Vortex Grit Chamber Effluent Parameters	
Flow (MGD)	33
Flow (cfs)	51.1
BOD (lb/day)	62,169.0
COD (lb/day)	113,035.51
TSS (lb/day)	46,980.38
TDS (mg/L)	941.50

Primary Sedimentation Basin

There is currently only one primary sedimentation basin at the CCWRP. The lack of redundancy resulted in a plant shut-down when maintenance was required on the primary clarifier. The diameter is determined based off the detention time, the side water depth, and the flow slope. Phase 2 will include construction of 2 additional primary clarifiers for a total of 3: 2 for treatment and 1 for redundancy. Phase 3 will add 3 primary clarifiers for a total of 4 – 3 for treatment and 1 for redundancy. The three new ones will be uniformly sized to allow for easy maintenance.

Table 36: Flow Estimations of CCWRP Primary Sedimentation Basin

CCWRP - Primary Clarifier Flow Parameters			
Flow Parameters	Phase 1	Phase 2	Phase 3
Years	2008-N/A	2025-2037	2037-2050
Q with Peaking Factor (of 1.85)	14,800,000	20,000,000	33,000,000
Additional Q to be Treated	-	5,200,000	13,000,000
HDT (hr)[a]	2.5	2.5	2.5
Volume Total (gal)	1,541,667	2,083,333	3,437,500
Additional Required Vol (gal)	-	541,667	1,354,167
Sidewater Depth (ft)	16	16	16
Flow Slope (in/ft)	1	1	1
Surface Area (ft^2)	12,880	9,503	9,503
Diameter (ft)	128	110	110

[a] Average hydraulic detention time ranges from 1.5 to 2.5 [29].

The required diameter was determined by Equation 2 below using an average HDT of 2.5 hours and a side water depth of 16 feet (the side water depth of the current sedimentation basin).

Equation 12: Hydraulic Detention Time of Primary Sedimentation Basin

$$Detention\ Time = \frac{Volume\ of\ Primary\ \ Settling\ Tank}{Flow\ Rate}$$

Table 37: Loading Estimations of CCWRP Primary Sedimentation Basin

CCWRP - Primary Clarifier Loading Parameters				
Loading Parameters		Phase 1	Phase 2	Phase 3
Q	Q (MGD)	8	20	33
	COD Loading (lb/day)	32,000	67,480	113,036
BOD	Loading (lb/day)	17,600	33,098	62,170
	Effluent of Primary (lb/day)	11,440	21,514	40,410
	Removed[1] (lb/day)	6,160	11,584	21,759
	% BOD Removed[a]	35%	35%	35%
TSS	Loading (lb/day)	13,300	28,046	46,980
	Effluent of Primary (lb/day)	5,985	12,621	21,141
	Removed[2] (lb/day)	7,315	15,425	25,839
	% TSS Removed[a]	55%	55%	55%

[a] BOD and TSS removal efficiency provided by manufacture [29].

Aeration Basin

The CCWRP had one rectangular aeration basin with 8 zones that come on as demand requires. There are course and fine air diffusers and "champagne" bubbles to prevent break-up of solids. The zones serve different treatment purposes and alternate the wastewater between aerobic and anaerobic zones. Return Activated Sludge (RAS) is pumped from the secondary clarifier to maintain a proper food to mass ratio, as well as intermediate mixed liquor pumps (IMLR) to feed the microorganisms back to the first zone. The IMLR pumps are critical infrastructure, thus their failure can result in a plant shut down [29].

It was determined that Phase 2 will require 2 additional basins for a total of 3 basins: 2 of the basins are needed for flow and 1 for redundancy. Phase 3 will require a total of 4 aeration basins: 3 for flow and one for redundancy. The redundant basins will prevent plant shut down if any critical infrastructure fails or requires maintenance.

The BOD removal was calculated using Equation 13. The TSS removal was determined using an average TSS effluent concentration.

Table 38: Flow and Loading Estimations for CCWRP Aeration Basin

CCWRP - Aeration Basin Loading Parameter				
Loading Parameters		Phase 1	Phase 2	Phase 3
Q	Influent (MGD)	8	20	33
BOD	Influent (lb/day)	11,440.00	21,513.67	40,410.19
	Influent (mg/L) [Ce]	171.370	129	147
	Effluent (mg/L)[a] [C_e]	16.001	12.037	13.702
	Effluent (lb/day)	1,068.19	2,008.80	3,773.22
	% BOD Removed	91%	91%	91%
TSS	Influent (lb/day)	5,985.00	12,620.83	21,141.17
	Influent (mg/L)	89.655	76	77
	Effluent (mg/L)[2]	40	40	40
	Effluent (lb/day)	2670.2	6675.6	11014.7
	% TSS Removed	55%	47%	48%

[a] BOD removal calculated using EPA design conditions for aerated mix lagoon and Equation 3 [31]. [2] Average TSS concentration in aeration basin effluent [31].

Equation 13: Aeration BOD treatment model [31]

$$C_e = C_0/[1 + \frac{K_{20}(t)}{n}]^n$$

Where:

C_e = Effluent BOD (mg/L)

C_0 = Influent BOD (mg/L)

K_{20} = Rate constant at 20 C

T = Solids detention time in system (days)

N = Number of equal sized cells in system

Table 39: Average EPA Design Conditions for Aerated Mix Lagoon

EPA Design Conditions [31]		
Temp Dependent Rate Constant	Kt	0.276
Solids Detention Time (days)	t	10
Number of Equal sized Cells	n	8

The aeration tank volume required for Phase 2 and 3 was determined by Equation 14 below. Assumptions made for this calculation are listed under Table 40.

Equation 14: Aeration Tank Volume

$$V = \frac{8.34 * S_0 * Q_0}{VL} * 1000$$

Where:

S_0= Aeration basin influent BOD concentration, mg/L

Q_0= Aeration basin influent flowrate, MGD

VL = Design volumetric flowrate, lb BOD/day/1000 ft^3

The hydraulic detention time was calculated using Equation 15 below. HDT is important to maintain adequate treatment.

Equation 15: Aeration Basin Hydraulic Detention Time

$$HRT = 24 * \frac{V_{MG}}{Q_0}$$

Where:

V_{MG} = Volume of aeration basin, million gallons

Q_0 = Aeration basin influent flowrate, MGD

The food to mass ratio was calculated using

Equation 16 below. The F:M is important to determine the amount of return activated sludge necessary to

Equation 16: Food to Mass Ratio for Aeration Basin

$$F{:}M = \frac{S_0 * Q_0}{\%\,Vol * X' * V_{MG}}$$

Where:

S_0= Aeration basin influent BOD concentration, mg/L

Q_0= Aeration basin influent flowrate, MGD

% Vol = Percent volatile MLSS

X' = MLSS concentration, mg/L

V_{MG} = Volume of aeration basin, million gallons

Table 40 - CCWRP Aeration Basin Flow and Sizing

Aeration Basin - Flow and Sizing			
Criteria	Phase 1	Phase 2	Phase 3
Primary Eff. Flowrate, Q_0 (MGD)	8	20	33
Prim Effluent BOD, S_0 (mg/L)	171.370	128.909	146.749
MLSS Conc. (X') (mg/L)[a]	2100	2100	2100
Design Vol. Loading, VL (lb BOD/day/1,000 ft^3)[b]	20	20	20
% Volatile MLSS, % Vol[c]	80%	80%	80%
Aeration Tank Volume (ft^3)	571,692	1,075,103	2,019,420
Aeration Tank Volume, V_{MG} (MG)	4.276	8.042	15.105
HRT (hr)	12.829	9.650	10.986
F:M (lb BOD/lb MLVSS)	0.19	0.19	0.19

Assumptions: [a] Value assumed based on typical wastewater quality [32]. [b] Design volumetric loading typical range for completely mixed aeration basins, [c] Within typical range for completely mixed aeration basins

Secondary Treatment

There are currently 2 secondary clarifiers at the CCWRP. The clarifiers have a diameter of 130 feet and a sidewall depth of about 18 feet and a volume of 1,786,326 gallons each. The hydraulic detention time (HDT) of secondary clarifiers range from 3-4 hours for proper treatment [33]. The HDT was determined using Equation 15. An average removal efficiency of 35% was used for BOD and 55% for TSS assuming proper HDT, as per manufacture specifications [33]. Based on the flow and loading, Phase 2 will require two additional clarifiers for flow and a third for redundancy, for a total of 5. Phase 3 will require an additional two secondary clarifiers for a total of 7. The new clarifiers will have a diameter of 110 feet to ensure proper HDT.

Table 41: Secondary Treatment, Flow, Loading, and Phasing

	CCWRP - Secondary Clarifier Loading Parameters			
	Loading Parameters	Phase 1	Phase 2	Phase 3
Q	Influent (MGD)	8	20	33
BOD	Influent (lb/day)	1,150.4	2,163.3	4,063.5
	Influent (mg/L)	17.2	13.0	14.8
	Effluent (lb/day)	747.73	1,406.16	2,641.26
	Effluent (mg/L)	11.20	8.43	9.59
	% BOD Removed	35%	35%	35%
TSS	Influent (lb/day)	2,670.2	6,675.6	11,014.7
	Influent (mg/L)	40	40	40
	Effluent (lb/day)	1,201.61	3,004.02	4,956.64
	Effluent (mg/L)	18	18	18
	% TSS Removed	55%	55%	55%

Table 42 - CCWRP Secondary Clarifier Dimensions

CCWRP - Secondary Clarifier Dimensions			
Flow Parameters	Phase 1	Phase 2	Phase 3
Q MGD	8	20	33
HDT (hr)[1]	4.00	4.00	4.00
Volume of One Clarifier (gal)	1,587,845	1,101,333	1,101,333
# of Clarifiers in Use	2	4	6
Volume Total (Mgal)	3.18	4.405	6.608
Sidewater Depth (ft)	16	15.5	15.5
Tank Side Wall Depth (ft)	18	17.5	17.5
Flow Slope	1:12	1:12	1:12
Surface Area (ft^2)	13,267	9,503	9,503
Diameter (ft)	130	110	110

Tertiary Treatment

The CCWRP used granular filters for tertiary treatment with anthracite and black sand mixture. Tertiary treatment is the final filtration before disinfection, so it is vital that suspended solids are adequately removed to prevent inference with the UV rays. There is currently one tertiary filtration system with 8 cells and 4 filters per cell. Based off future flow and loading data, it is predicted that an additional system will be required for Phase 2 flows, for a total of 2. Phase 3 will require one more additional system for a total of 3 filtration systems. The two new systems will be SuperSand systems which require less power than the current system. The systems will be arranged in parallel to allow for maintenance.

Table 43- CCWRP Tertiary Treatment Dimensions

CCWRP - Tertiary Treatment Dimensions			
Flow Parameters	Phase 1 [a]	Phase 2	Phase 3
Q MGD	8	20	33
# of Basins	8.00	16.00	24.00
Filters per Basin	4	4	4
Filter Area per Filter (ft^2)	49	50	50
Design Loading Rate (gpm/ft^2)	2.13	1.70	1.70
Peak Loading Rate (gpm/ft^2)	4.26	3.2	3.2
Bed Depth (ft)	6.5	6.56	6.56

Table 44 - CCWRP Tertiary Filter Loading Parameters

CCWRP - Tertiary Clarifier Loading Parameters				
Loading Parameters		Phase 1	Phase 2	Phase 3
Q	Influent (MGD)	8	20	33
BOD	Influent (lb/day)	747.7	1,406.2	2,641.3
	Influent (mg/L)	11.2	8.4	9.6
	Effluent (lb/day)	112.16	210.92	396.19
	Effluent (mg/L)	1.68	1.26	1.44
	% BOD Removed[a]	85%	85%	85%
TSS	Influent (lb/day)	1,201.6	3,004.0	4,956.6
	Influent (mg/L)	18.0	18.0	18.0
	Effluent (lb/day)	367.16	917.90	1,514.53
	Effluent (mg/L)[b]	5.5	5.5	5.5
	% TSS Removed	69%	69%	69%

[a] BOD removal efficiency based on study conducted on removal of oxygen demand using anthracite [34] [b] Average TSS effluent concentration in pilot study conducted on granular filters using anthracite and sand [35]

Salinity Removal Flow and Loading

The existing CCWRP does not have a method of salinity reduction which resulted in high TDS concentrations in the effluent. Phase 2 of the plant will implement an R.O. system to reduce TDS to a maximum concentration of 1,000 mg/L. A percentage of the total flow will be diverted through the R.O. and TDS in the diverted flow will be completed removed. It will be combined and mixed with the non-treated flow to result in ≤ 1,000 mg/L TDS. The system is the PureAqua TW-900K-18780 which contains 128 membranes of 8" size and allows a permeate flowrate of 900,000 GPD.

About 47% of the flow will be diverted for Phase 2, which will require 11 PureAqua systems, 10 for flow and 1 for redundancy. Based on increased water softener use in the sewershed, the TDS entering the plant will increase and therefore more of the flow will need to be diverted for Phase 3. Phase 3 will require 50% diverted flow and 19 systems, 18 for flow and 1 for redundancy.

Table 39 provides flow and loading for treated, untreated, and combined flow. The membrane concentrate will be discharged to an evaporation pond and periodically the salt must be emptied and disposed of in a landfill.

Table 45: Flow Distribution for Reverse Osmosis for TDS Removal in CCWRP

Diverted Flow		Phase 2	Phase 3
Total In Flow	Q (MGD)	20	33
	TDS (mg/L)	1,696.2	1,775.0
Treated Flow (Feed water into RO)	% Diverted	46.96%	50%
	Qin (MGD)	9.33	16.53
	TDS in (mg/L)	1696.22	1775
	% Removed	99%	99%
	TDS Removed (mg/L)	1679.3	1757.3
	TDS Effluent (mg/L)	17.0	17.75
	Qout (MGD)	7.5	13.2
Membrane	Q (GPD, Per system)	900,000	900,000
	# of Systems	10	18
	Q (Out Total) MGD	1.9	3.3
	Concentrated TSS (mg/L)	1679.3	1757.3
Non-Treated Flow	Q	10.5	16.8
	TSS	1696.22	1775
Final Combined Flow	Q	18.0	30.0
	TDS (mg/L)	1000.0	1000.0

Equation 17: % Recovery of R.O. System [36]

$$\% \, Recovery = \frac{Permeate \ Flow \ Rate \ (gpm)}{Feed \ Flow \ Rate \ (gpm)} * 100$$

Equation 18: Salinity Concentration Factor of R.O. System [36]

$$Concentration \ Factor = (\frac{1}{1 - Recovery \ \%})$$

Equation 19: Flux into R.O. System [36]

$$Flux \ (Gfd) = \frac{gpm \ of \ permeate \ of \ 1{,}440 \ min/day}{\# \ RO \ elecments \ in \ system * square \ footage \ of \ each \ RO \ element}$$

Table 46: Proposed R.O. Membrane Speculations for CCWRP

Proposed R.O. Membrane Speculations	
% Recovery	80%
Concentration Factor	5
Reject Concentration (mg/L)	8481.1
Permeate Flowrate (MGD)	15.9
Permeate Flowrate (gpm)	11037.8
Area (ft^2)	400.0
Flux (GFD)	49.0

Appendix G: Standards for Effluent Reuse

Table 47: Standards for Effluent Use

Standards For Effluent Use						
Designated Use	Arizona Administrative Code	Coliform (cfu/100 ml)[a]	Total Nitrogen (mg/L)	Dissolved Oxygen[b] (mg/L)	pH	TSS[c]
Discharge into Cave Creek - A&Ww	R18-11-109(A), R18-11-109(E)	126	1.6-1.8	3	6.5-9	80 mg/L
Domestic Water Source (Discharge into source of potable water)	R18-11-108.03 D	0 e	1.2-1.5	-	5.0-9.0	
Groundwater Recharge: Aquifer Water Quality	R18-11-406	0 e	10	-		1 (NTU)
A+ Standardsg - Direct Reuse (Irrigation)	R18-11-303	23h	10	N/A		2(NTU)d

a= Geometric mean (minimum of four samples in 30 days) [8] , b = Minimum concentration, Sample must be taken from a depth no greater than one meter, c = Median value determined from a minimum of four samples collected at least seven days apart, d= five or fewer may be allowed as long as it does not interfere with disinfection, e= If a sample is total coliform positive, a 100 mL repeat sample shall be taken within two weeks, f = Source: R18-11 Appendix B, g= Can be used for any time of direct reuse listed in Table A, h= No detectable fecal coliform in 4 out of last 7 daily samples, standard is for single max concentration.

Table 48: Salinity Effect on Crop Yield [8]

Salinity Effects	
Effect on Crop Yield	TDS (mg/L)
No detrimental effects on crop yield	<500
Can affect crop yield and sensitive plants	500-2000
Severe problems with crop yield	>2000

Table 49: Salinity Limits for Effluent Use [8]

Potential Salinity Limits for End Use	
End Use	TDS mg/L
Stream Discharge	800-1,500
Agricultural Reuse	800-1,500
Turf Irrigation	500-1,200
Groundwater Recharge	500-1,000

Table 50: Types of reuse appropriate for increasing levels of treatment – EPA [9]

Treatment Level	Increasing Levels of Treatment → Primary	Secondary	Filtration and Disinfection	Advanced
Processes	Sedimentation	Biological oxidation and disinfection	Chemical coagulation, biological or chemical nutrient removal, filtration, and disinfection	Activated carbon, reverse osmosis, advanced oxidation processes, soil aquifer treatment, etc.
End Use	No Uses Recommended	Surface irrigation of orchards and vineyards	Landscape and golf course irrigation	Indirect potable reuse including groundwater recharge of potable aquifer and surface water reservoir augmentation and potable reuse
		Non-food crop irrigation	Toilet flushing	
		Restricted landscape impoundments	Vehicle washing	
		Groundwater recharge of nonpotable aquifer	Food crop irrigation	
		Wetlands, wildlife habitat, stream augmentation	Unrestricted recreational impoundment	
		Industrial cooling processes	Industrial systems	
Human Exposure	Increasing Acceptable Levels of Human Exposure →			
Cost	Increasing Levels of Cost →			

Appendix H: Solar Panel Sizing

Table 51: Electrical Demand for Trojan SIGNA UV Disinfection System

Trojan SIGNA UV Disinfection System Electrical Demand				
Phase	Phase 2: 20 MGD		Phase 3: 33 MGD	
Variable Output	30%	100%	30%	100%
Watts per Lamp (W)	1000	1000	1000	1000
Lamps per Bank	16	16	16	16
Banks Per Module	6	6	8	8
Hours of Operation	24	24	24	24
Watt Hours (Wh) per Day	691200	2304000	921600	3072000
Kilo Watt Hours per Day per Unit (kWh/unit)	691	2304	922	3072
Electrical Cost per Day ($0.10/kWh)	$ 69.12	$ 230.40	$ 92.16	$ 307.20
Days of Operation	365	365	365	365
kWh per Year (kWh/year)	252,288	840,960	336,384	1,121,280

Table 52: Sizing for Solar Panels

Solar Panel Sizing Chart			
Length of panel (ft)		6.67	
Width of panel (ft)		3.33	
Area of panel (ft^2)		22.22	
Trojan UV Signa Phase 2		Trojan UV Signa Phase 3	
Maximum KWH (Wh)	2304000	Maximum KWH (KWh)	3072000
Round up (Wh)	2400000	Round up (KWh)	3100000
Watts/panel (W/unit)	400	Watts/panel (W/unit)	400
Daily hours of panel operation (hr)	12	Daily hours of panel operation (hr)	12
Number of panels	500	Number of panels	646
Efficiency of panels	15%	Efficiency of panels	15%
Actual number of panels	3333	Actual number of panels	4306
Round up panels	3400	Round up panels	4400
Estimated Energy Production (kWh)	2448	Estimated Energy Production (kWh)	3168
Estimated needed area (ft^2)	75,556	Estimated needed area (ft^2)	97,778
GIS solar panel area (ft^2)	600,000	GIS solar panel area (ft^2)	600,000

Appendix I: Staffing Levels

Table 53: Adjustment for Local Conditions

CATEGORY	LOCAL CONDITION	ADJUSTMENT					
		Operation	Maintenance	Supervisory	Clerical	Laboratory	Yardwork
Plant Layout	Average	0%	0%				0%
Unit Process	Std. Equipment/ Different Mfr	0%	0%				
Level of Treatment	Advanced	10%	-20%	2%	2%	2%	10%
Type of Waste Removal Requirement	Effluent Concentration	5%				10%	
Industrial Wastes	None or Constant	0%				0%	
Productivity	Average	0%	0%				
Climate	Moderate Winters		0%				
Training	Certification & Continuing Ed.	-5%		-10%			
Auto Monitoring	Monitoring With Feedback	-15%	10%				
Auto Sampling	Throughout Plant	-5%				-10%	
Off-plant Lab	None					0%	
Off-plant Maintenance	None		0%				
Age of Equipment	Relatively new & well cared for		0%				
Total		-10%	-10%	-8%	2%	2%	10%

Table 54: Annual Man-hours

Unit Process/Category	Exists at Plant?	Operation	Maintenance	Supervisory	Clerical	Laboratory	Yardwork
Supervisory & Administrative				2,660			
Clerical					1,060		
Laboratory						2,510	
Yardwork							2,450
Raw Sewage Pumping at Plant	Yes		470				
Screening & Grinding	Yes	1,280	40				
Grit Removal	Yes	770	70				
Primary Clarification	Yes	2,600	540				
Aeration	Yes	1,830	2,050				
Secondary Clarification for Activated Sludge	Yes	2,420	430				
Chlorination	Yes	370	410				
Tertiary Filtration	Yes	2,030	1,440				
Subtotal		11,300	5,450	2,660	1,060	2,510	2,450
Adjustment		-10%	-10%	-8%	2%	2%	10%
SUBTOTAL ADJUSTED FOR LOCAL CONDITIONS		10,170	4,910	2,450	1,080	2,560	2,700
Number of Workers		6.8	3.3	1.6	0.7	1.7	1.8

Appendix J: Cost Estimates

Table 55 shows the capital cost for the phase 2 expansion. The 2025 future cost was calculated using Equation 20 below.

Equation 20: Future Worth

$$FW = PW * (1 + i)^n$$

Where: FW = Future Worth, PW = Present Worth, i = Interest Rate, n = Number of Years

Table 55: CCWRP Unit Cost Table

Phase 2 Capital Cost Estimation								
Item #	**item**	**Size/Description**	**unit**	**Quantity**	**2019 Cost/Unit**	**2025 Cost/unit**	**2019 Capital Cost**	**2025 Capital Cost**
Influent Pump Station							**$ 599,150**	**$ 787,060**
1	Pump	12in slurry pump from eddy pumps, Model 12-Inch HD 12000	EA	3	$ 117,000	$ 139,704	$ 351,000	$ 419,112.36
2	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 105,300	$ 125,734
3	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 122,850	$ 146,689
4	Removal Existing	Remove existing Pumps	EA	4	$ 5,000	$ 23,881	$ 20,000	$ 95,524.18
Headworks Building							**$ 807,363**	**$ 964,033**
1	Bar Screen	EscaMax Perforated Plate Band Screen, Model 4000x352/6	EA	2	$ 224,000	$ 267,468	$ 448,000	$ 534,935.43
2	Metal Grate Walkway	Elevated Metal Grate Walkway, 1" bar spacing, 1" height, 4'x4' panels W/ supports	EA	65	$ 450	$ 537	$ 29,250	$ 34,926.03
3	Concrete	Normal Weight Reinforced Concrete	CY	12	$ 500	$ 597	$ 6,000	$ 7,164.31
4	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 144,975	$ 173,108
5	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 169,138	$ 201,959
6	Removal Existing	Remove existing bar screens	EA	2	$ 5,000	$ 5,970	$ 10,000	$ 11,940.52
Grit Removal							**$ 876,975**	**$ 1,047,154**
1	Grit Chamber	Huber Technology Varimax Grit Chamber, Model size 5	EA	4	$ 131,000	$ 156,421	$ 524,000	$ 625,683.40
2	Concrete	Normal Weight 6" Reinforced Concrete Slab	CY	15	$ 500	$ 597	$ 7,500	$ 8,955.39
3	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)				$ 47,105	$ 159,450	$ 190,392
4	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)				$ 54,956	$ 186,025	$ 222,124
Primary Clarifiers							**$ 896,775**	**$ 1,070,796**
1	Primary Clarifier Basin	110' Primary Clarifier Basin W/ weirs, baffles, and mechanical mechanisms, Model COPC2G	EA	2	$ 253,000	$ 302,095	$ 506,000	$ 604,190.46
2	Excavation	Excavation and earthwork for instillation of primary clarifier basins	CY	750	$ 50	$ 60	$ 37,500	$ 44,776.96
3	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 163,050	$ 194,690
4	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 190,225	$ 227,139
Aeration Basins							**$ 54,450,000**	**$ 65,016,148**
1	Aeration Basin	Duplicate of existing rectangular 8 zone system	EA	2	$ 15,000,000	$ 17,910,784	$ 30,000,000	$ 35,821,569
2	Excavation	Excavation and earthwork for instillation of Aeration Basins	CY	60,000	$ 50	$ 60	$ 3,000,000	$ 3,582,157
3	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)				$ 5,373,253	$ 9,900,000	$ 11,821,118
4	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)				$ 6,268,795	$ 11,550,000	$ 13,791,304
Secondary Clarifiers							**$ 1,907,400**	**$ 2,277,535**
1	Secondary Clarifier Basin	110' Secondary Clarifier Basin W/ weirs, baffles, and mechanical mechanisms, Model COPC2G	EA	4	$ 264,000	$ 315,230	$ 1,056,000	$ 1,260,919
2	Excavation	Excavation and earthwork for instillation of primary clarifier basins	CY	2,000	$ 50	$ 60	$ 100,000	$ 119,405
3	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 346,800	$ 414,097
4	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 404,600	$ 483,114

Item #	item	Size/Description	unit	Quantity	2019 Cost/Unit	2025 Cost/unit	2019 Capital Cost	2025 Capital Cost
Tertiary Filters							**$ 3,526,875**	**$ 4,211,273**
1	Tertiary Filter	Super sand tertiary system W/ 8 basins & 4 filters per basin	EA	1	$ 1,996,000	$ 2,383,328	$ 1,996,000	$ 2,383,328
2	Excavation	Excavation and earthwork for instillation of primary clarifier basins	CY	2,410	$ 50	$ 60	$ 120,500	$ 143,883
3	CMU Wall	Concrete Masonry Unit wall around the top of filter basin	SF	840	$ 25	$ 30	$ 21,000	$ 25,075
4	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 641,250	$ 765,686
5	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 748,125	$ 893,300
UV Disinfection							**$ 2,003,100**	**$ 2,391,806**
1	Trojan UV Signa Bank	Trojan UV Signa bank with 16 1000W bulbs per bank W/ controls, sluice gate, and connections	EA	14	$ 86,000	$ 102,688	$ 1,204,000	$ 1,437,638.97
2	Removal Existing	Remove existing UV system and controls	EA	2	$ 5,000	$ 5,970	$ 10,000	$ 11,940.52
3	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 364,200	$ 434,874
4	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 424,900	$ 507,353
Desalination							**$ 7,590,000**	**$ 9,062,857**
1	Reverse Osmosis Membrane	Pure Aqua TW-900K-18780	EA	11	$ 400,000	$ 477,621	$ 4,400,000	$ 5,253,830.10
2	Concrete	Normal Weight 6" Reinforced Concrete Slab	CY	200	$ 500	$ 597	$ 100,000	$ 119,405.23
3	Structural Masonry Building	construction of a Brick building 100'X50'	SF	5000	$ 20	$ 24	$ 100,000	$ 119,405.23
4	Pipe, Valves & Fittings	Estimation of pipes, valves and fittings (30% of unit cost)					$ 1,380,000	$ 1,647,792
5	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 1,610,000	$ 1,922,424
Solar Power							**$ 6,920,000**	**$ 8,262,842**
1	Solar Panels	LG 400W Solar Panel W/ Mounting System, connections and controls	EA	3400	$ 1,500	$ 1,791	$ 5,100,000	$ 6,089,666.71
2	Fencing	Chain-link fence to enclose the solar array	LF	1400	$ 25	$ 30	$ 35,000	$ 41,791.83
3	Electrical	Estimation of electrical connections and instrumentation (35% of unit cost)					$ 1,785,000	$ 2,131,383

Table 56: CCWRP Engineering Design Firm Hours

Task		Staff (hr)				Total Task Hours
		Senior Engineer	Project Engineer	EIT	Intern	
1.0 Project Introduction	1.1 Become Members of WEF & AZ Waters	1	1	1	1	4
	1.2 Send in Competition Registration Form	1	1	1	1	4
	1.3 Send in Competition Entry Form	1	1	1	1	4
2.0 Existing Condition Site Assessment	2.1 Analysis of Existing Conditions	12	18	18	8	56
	2.2 Historical Wastewater Flow & Loading	3	8	8	4	23
	2.3 Hydraulic Analysis	4	15	15	7	41
3.0 Projected Growth Analysis	3.1 Projected Population Analysis	4	6	8	4	22
	3.2 Projected Flow Analysis	4	8	8	4	24
	3.3 Projected Loading Analysis	10	10	10	6	36
4.0 Proposed Design Solutions	4.1 Effluent Usage	5	4	4	4	17
	4.2 Plant Upgrade Alternatives	10	20	30	14	74
	4.3 Design Criteria	10	20	20	10	60
	4.4 Proposed Costs	16	12	10	10	48
5.0 Selection of Design Improvements	5.1 Implementation of Construction and Phasing	6	4	4	5	19
	5.2 Proposed Staffing Levels	5	3	3	3	14
6.0 Project Management	6.1 Meetings	20	20	20	20	80
	6.2 Travel	10	10	10	10	40
	6.3 Reports	40	40	40	40	160
	6.4 Presentation	10	10	10	10	40
Total Staff Hours		**172**	**211**	**221**	**162**	**766**

Table 57: Contractor Construction Costs

General Contractor Costs		
Labor Cost		
Laborer		8
hr/week		50
charge rate ($)		45
weeks worked per year		49
length of project (Years)		2
Total	$	1,764,000
Equipment Costs		
construction equipment	$	750,000
rental equipment	$	200,000
Total	$	950,000
Contractor Cost Breakdown		
General Conditions (9%)	$	8,558,235
Overhead (5%)	$	4,754,575
Profit (10%)	$	9,509,150
Privilege Tax (5.395%)	$	5,130,187
Bond (1.2%)	$	1,141,098
Total	$	31,807,246
Sub-Contractor Costs		
Sub-Contractor Cost (30% GC)	$	10,356,374

Table 58: Electrical Savings Cost Estimate

CCWRP Solar Power Yearly Electrical Savings		
Criteria	Phase 2	Phase 3
Number of panels	3,400	4,400
Watts/panel	400	400
Efficiency (%)	0.15	0.15
Hours of operation (hr/day)	12	12
Wh/day	2,448,000	3,168,000
kwh/day	2,448	3,168
kwh/year	820,080	1,156,320
$/year	$ 410,040	$ 578,160

Appendix K: Proposed Plant Expansion

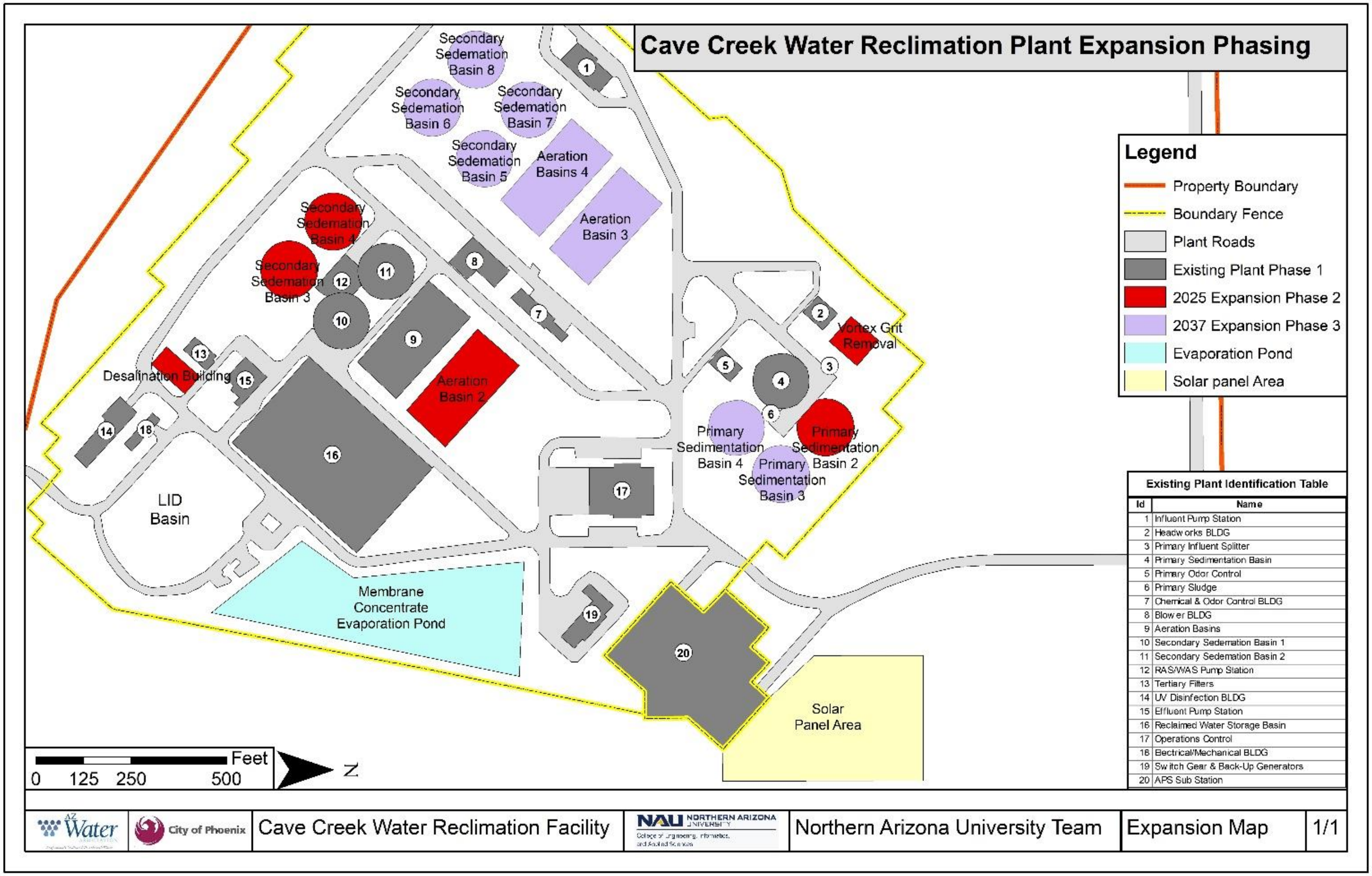


Figure 17: CCWRP Proposed Expansion Map

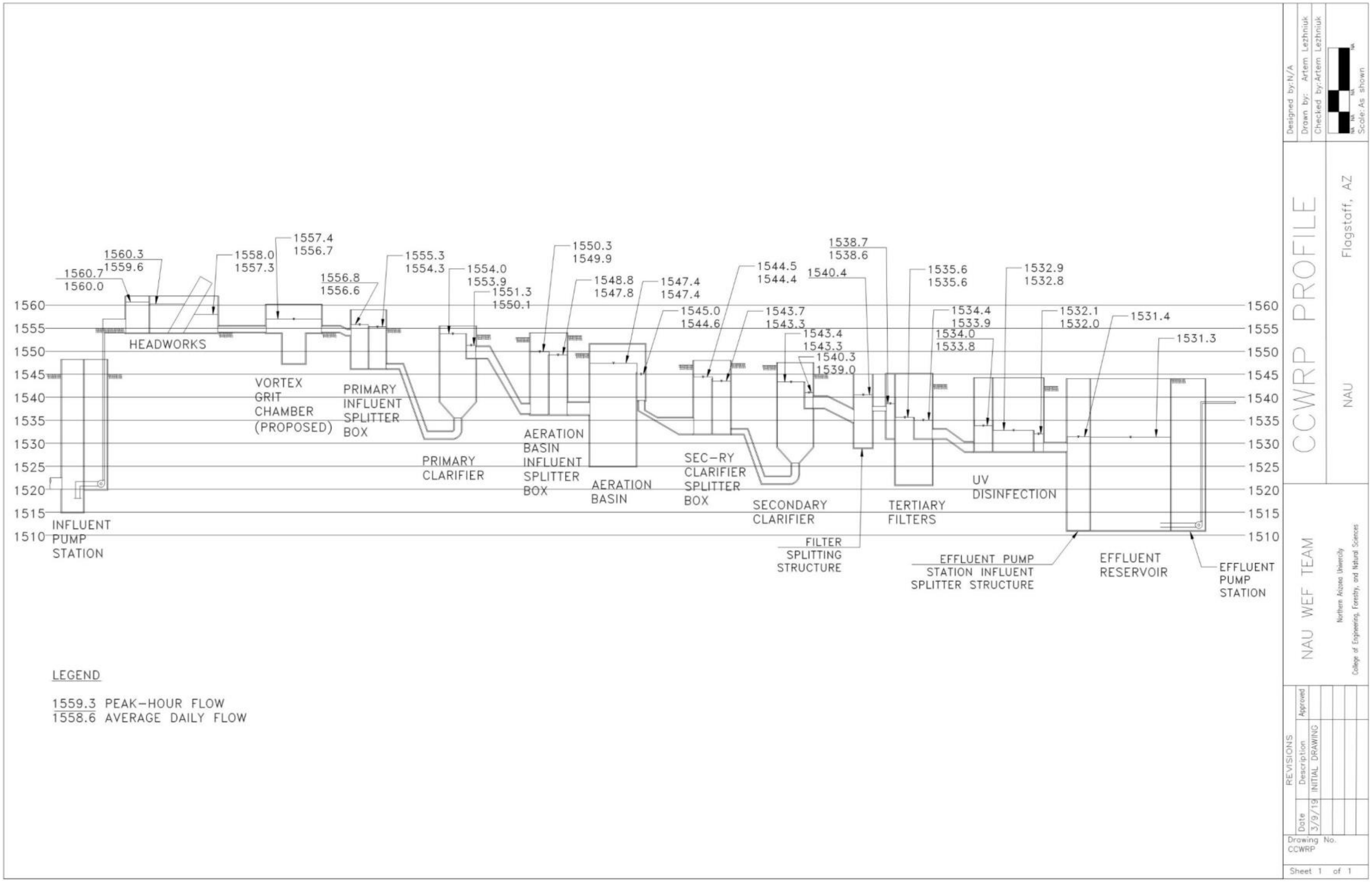


Figure 18: Proposed Phase 2 CCWRP Hydraulic Profile