

AGENDA



3:00 P.M.

CITY COUNCIL MEETING

WEDNESDAY, JULY 9, 2025

NOTES:

- Meeting room is wheelchair accessible. American Disabilities Act (ADA) accommodations are available upon request. Please phone 928-282-3113 at least two (2) business days in advance.
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GUIDELINES FOR PUBLIC COMMENT

PURPOSE:

- To allow the public to provide input to the City Council on a particular subject scheduled on the agenda.
- This is not a question/answer session.
- No disruptive behavior or profane language will be allowed.

PROCEDURES:

- Fill out a "Comment Card" and deliver it to the City Clerk.
- When recognized, use the podium/microphone.
- State your:
 1. Name and
 2. City of Residence
- Limit comments to **3 MINUTES**.
- Submit written comments to the City Clerk.

1. CALL TO ORDER/PLEDGE OF ALLEGIANCE/MOMENT OF SILENCE

2. ROLL CALL

3. SPECIAL BUSINESS

LINK TO DOCUMENT =

- a. AB 3172 **Discussion/possible direction** regarding the Wastewater Reclamation Plant (WWRP) Facility Plan on PFS treatment, and possible options for reclaimed water delivery and Advanced Water Purification.
- b. **Discussion/possible action** regarding ideas for future meetings/agenda items.

4. EXECUTIVE SESSION

Upon a public majority vote of the members constituting a quorum, the Council may hold an Executive Session that is not open to the public for the following purposes:

- a. To consult with legal counsel for advice regarding matters listed on this agenda per A.R.S. § 38-431.03(A)(3).
- b. Return to open session. Discussion/possible action regarding executive session items.

5. ADJOURNMENT

Posted: 7/02/2025

By: MG

JoAnne Cook, CMC, City Clerk

Note: Pursuant to A.R.S. § 38-431.02(B) notice is hereby given to the members of the City Council and to the general public that the Council will hold the above open meeting. Members of the City Council will attend either in person or by telephone, video, or internet communications. The Council may vote to go into executive session on any agenda item, pursuant to A.R.S. § 38-431.03(A)(3) and (4) for discussion and consultation for legal advice with the City Attorney. Because various other commissions, committees and/or boards may speak at Council meetings, notice is also given that four or more members of these other City commissions, boards, or committees may be in attendance.

A copy of the packet with material relating to the agenda items is typically available for review by the public in the Clerk's office after 1:00 p.m. the Thursday prior to the Council meeting and on the City's website at www.SedonaAZ.gov. The Council Chambers is accessible to people with disabilities, in compliance with the Federal 504 and ADA laws. Those with needs for special typeface print, may request these at the Clerk's Office. All requests should be made **forty-eight hours** prior to the meeting.

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CITY COUNCIL CHAMBERS
102 ROADRUNNER DRIVE, SEDONA, AZ

The mission of the City of Sedona government is to provide exemplary municipal services
that are consistent with our values, history, culture and unique beauty.



CITY COUNCIL AGENDA BILL

AB 3172
July 9, 2025
Special Business

Agenda Item: 3a

Proposed Action & Subject: Discussion/possible direction regarding the Wastewater Reclamation Plant (WWRP) Facility Plan on PFS treatment, and possible options for reclaimed water delivery and Advanced Water Purification.

Department	Wastewater
Time to Present	1 hour
Total Time for Item	2 hours
Other Council Meetings	August 13, 2024 - AB 3093; January 15, 2025 – AB 3172
Exhibits	A. Wastewater Reclamation Facility Plan

Finance Approval	Reviewed 6/30/25 BGW	
City Attorney Approval	Reviewed 6/19/25 KWC	Expenditure Required
		\$ n/a
City Manager's Recommendation	Provide feedback to staff 7/1/25 ABS	Amount Budgeted
		\$ n/a
		Account No. (Description)

SUMMARY STATEMENT

Background: During the FY2025 budget planning process, the Wastewater Department proposed two options for long term effluent management. The first option is to keep irrigation but make costly, necessary upgrades. The second option is to abandon irrigation on 200 acres of the Dells land and construct two new recharge wells. The projected costs for the second option are substantial and additional wells may be needed to meet future capacity.

The Environmental Protection Agency (EPA) has imposed limitations on Per- and Polyfluoroalkyl Substances (PFAS) in drinking water and is proposing limitations on wastewater effluent and biosolids. The limitations proposed would require the Wastewater Reclamation Plant (WWRP) to add additional treatment processes to meet the regulatory requirements for PFAS removal, requiring substantial capital improvements.

The WWRP Facility Plan is essential to establish operational and capital improvement needs for impending regulatory requirements, options for effluent management, and potential future capacity needs of the treatment process. The plan provides options on future needs for expansion at the WWRP, for both treatment processes and effluent management strategies.

At the January 15, 2025, City Council Special Meeting, Carollo Engineers presented the findings of the WWRP Facility Plan including the following:

- Hydraulic and treatment capacity analysis of the WWRP to determine if additional capacity will be needed for future increases in flow and/or increases in the strength of the wastewater influent. The capacity analysis indicated that capacity is adequate for projected build-out flows at the WWTP, with no major capacity increases required.
- Additional treatment processes which may be needed to meet proposed regulatory requirements on PFAS. Timing on the improvements needed for PFAS removal are unknown, until regulatory requirements are released. Cost estimates for implementing PFAS removal are estimated to range from \$28.3 - \$34.4 million dollars.
- Evaluation of alternative uses for long term effluent management including delivering reclaimed water to city limits for irrigation or other approved uses and Advanced Water Purification (AWP), with planning level cost estimates. These alternatives would supplement or replace current effluent management strategies that include spray irrigation, aquifer recharge, and wetlands.
- Decision criteria for alternatives to aid in developing water policy and future capital improvement and operational needs.

City Council directed staff to finalize the plan to include all effluent management options presented. Carollo Engineers and city staff have developed a ranking and scoring system to evaluate the five alternatives for effluent management:

- Alternative 1: Keep existing irrigation (approximately 265 acres), wetlands, and injection wells and add two injection wells for needed future effluent management capacity.
- Alternative 2: Abandon 200 acres of irrigation and add two new injection wells in the short term; plan for two additional wells in the long term for needed capacity.
- Alternative 3: Deliver reclaimed water to parks and keep current irrigation, wetlands, and injection wells.
- Alternative 4: Deliver reclaimed water to parks, keep wetlands and existing injection wells, and abandon irrigation except Area 3 by adding two new injection wells.
- Alternative 5: Implement AWP and potable delivery.

Evaluation criteria for ranking the alternatives included costs, environmental and economic impacts, water resiliency, dependability, health and safety, public perception, infrastructure and land requirements, implementation time, ease of expansion, operations and maintenance considerations, and personnel requirements. Weighting factors for the criteria were applied using a pairwise comparison approach. Each alternative was then scored on each criterion using a scale of 1 to 10 and subsequently calculated an average score based on the weighting factor for each criterion. The final score for the alternatives, along with estimated capital costs, are summarized below:

	Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5
Overall Score	5.70	7.66	5.92	7.51	6.14
Capital Cost (\$, million)	\$11	\$22	\$28.6	\$39.6	\$161

Capital costs for PFAS removal and effluent management alternatives 1 and 2 are incorporated into the 10-year wastewater CIP budget. Funding sources for PFAS removal and all effluent

management alternatives may include utilizing the wastewater fund, bonds, grants, and potential cost sharing with Arizona Water Company.

Staff is looking for possible direction on next steps for incorporating alternatives into water policy, including coordination with Arizona Water Company, and further developing the capital improvement plan.

Climate Action Plan/Sustainability Consistent: ☒Yes - ☐No - ☐Not Applicable

The WWRP Facility Plan will meet the goals related to the Water Resources Management Plan action for creating an integrated water resources management plan that ensures a long-term sustainable supply of water.

Board/Commission Recommendation: ☐Applicable - ☒Not Applicable

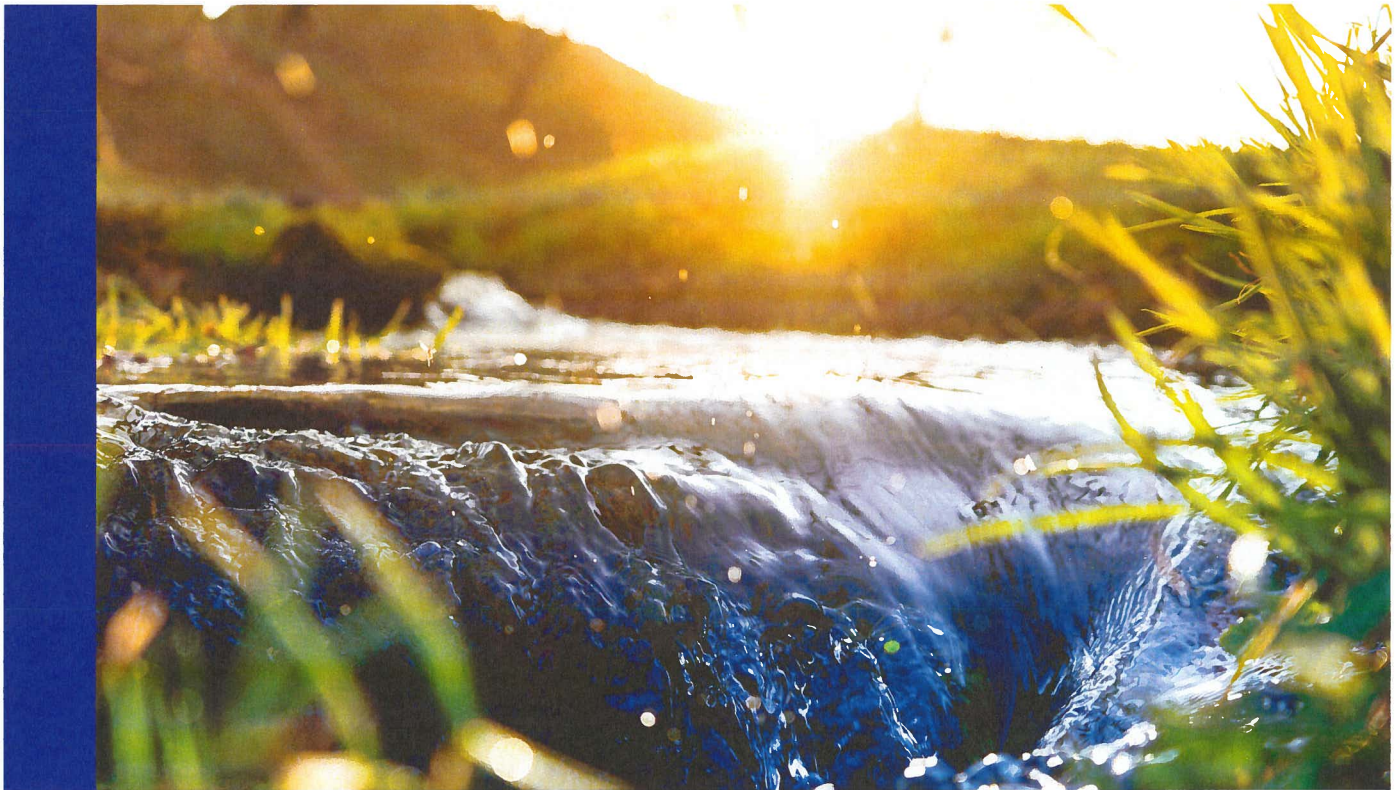
Alternative(s): n/a

MOTION

I move to: Discussion/possible direction only.



CITY OF SEDONA



WASTEWATER RECLAMATION PLANT FACILITY PLAN

FINAL / June 2025





CITY OF SEDONA

WASTEWATER RECLAMATION PLANT FACILITY PLAN

FINAL / June 2025



Digitally signed by Jessica A. Dresang
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Abbreviations

°C	degrees Celsius
µg/kg	micrograms per kilogram
µg/L	micrograms per liter
A.A.C.	Arizona Administrative Code
AACE	Association for the Advancement of Cost Engineering
AADF	annual average daily flow
AADL	annual average daily load
ADEQ	Arizona Department of Environmental Quality
APP	Aquifer Protection Permit
AWP	Advanced water purification
AWPF	advanced water purification facility
AWQS	Aquifer Water Quality Standards
AWT	advanced water treatment
BADCT	best available demonstrated control technology
BAF	biologically active carbon filtration
BOD	biochemical oxygen demand
BOD ₅	5-day biochemical oxygen demand
CaCO ₃	calcium carbonate
Carollo	Carollo Engineers
CBAT	carbon-based advanced treatment
CBOD ₅	5-day carbonaceous biochemical oxygen demand
CCPP	calcium carbonate precipitation potential
CFU	colony forming units
City	City of Sedona
CMF	concentration management facility
COD	chemical oxygen demand
CSF	clarifier safety factor
CSMR	chloride-to-sulfate mass ratio
<i>E. Coli</i>	<i>Escherichia coli</i>
EBCT	empty bed contact time
EQ	equalization
Facility Plan	Wastewater Reclamation Plant Facility Plan
fps	feet per second
GAC	granular activated carbon
GenX	hexafluoropropylene oxide dimer acid
gpd	gallons per day

gpm	gallons per minute
gpm/sq ft	gallons per minute per square foot
HI	Hazard Index
hp	horsepower
HRT	hydraulic residence time
ID	identification
IMLR	internal mixed liquor return
ISWC	initial source water characterization
IX	ion exchange
kg	kilograms
kwh	kilowatt per hour
L	liter
lb/day	pounds per day
LS	lime softening
MCL	maximum contaminant level
MCLG	maximum contaminant level goal
MF	microfiltration
MG	million gallons
mg/L	milligrams per liter
mgd	million gallons per day
mL	milliliter
mL/g	milliliter per gram
MLSS	mixed liquor suspended solids
MMADF	maximum monthly average daily flow
MMADL	maximum monthly average daily load
mV	millivolts
MVRE	mechanical vapor recompression evaporator
ND	non detect
NDMA	N-nitrosodimethylamine
NF	nanofiltration
ng/L	nanogram per liter
NH ₃ -N	ammonia nitrogen
NH ₄	ammonium
NO ₂ -N	nitrite nitrogen
NO ₃ -N	nitrate nitrogen
NO _x	nitrogen oxides
NR	not required
NTU	nephelometric turbidity unit
O&M	operations and maintenance

PDF	peak daily flow
PFAS	per- and polyfluoroalkyl substances
PFBS	perfluorobutane sulfonic acid
PFD	process flow diagram
PFHxS	perfluorohexanesulfonic acid
PFNA	perfluorononanoic acid
PFOA	perfluorooctanoic acid
PFOS	perfluorooctyl sulfonate
pH	potential of hydrogen
PHF	peak hour flow
PHPF	peak hour peaking factor
POC	point of compliance
ppt	parts per trillion
RAS	return-activated sludge
RBAT	reverse osmosis-based advanced treatment
RO	reverse osmosis
RSSCT	rapid small-scale column testing
SC	secondary clarifier
SO _x	sulphur oxides
SRT	solids retention time
SVI	sludge volume index
TDH	total dynamic head
TDS	total dissolved solids
TIN	total inorganic nitrogen
TKN	total Kjeldahl nitrogen
TN	total nitrogen
TOC	total organic carbon
TS	total solids
TSS	total suspended solids
UF	ultrafiltration
uS/cm	microsiemens per centimeter
USEPA	United States Environmental Protection Agency
USF	underground storage facility
UV	ultraviolet
UV AOP	ultraviolet advanced oxidation process
UVT	ultraviolet light transmission
UWSRA	Underground Water Storage and Recovery Act
VSS	volatile suspended solids
WAS	waste-activated sludge

WRA	Whitman, Requardt & Associates, LLP
WS	water storage
WWRP	wastewater reclamation plant
X	required
µm	micrometers

CHAPTER 1 INTRODUCTION

1.1 Project Background

The City of Sedona (City) owns and operates the Sedona Wastewater Reclamation Plant (WWRP) in Sedona, Arizona. The WWRP currently produces Class A+ effluent, which is delivered to a combination of injection/recharge wells, wetlands, irrigation, and reclaimed water. A full plant capacity evaluation was completed as the initial task of this study to identify treatment and hydraulic bottlenecks throughout the facility at current and buildout flows and loads. With recent developments in federal and state regulations related to per- and polyfluoroalkyl substances (PFAS), the City is evaluating the technical and economic feasibility of implementing treatment alternatives that would treat drinking water PFAS maximum contaminant levels (MCL). Additionally, the City is considering effluent management strategies and expansion needs, including the possible addition of injection wells to increase the existing effluent management capacity and flexibility. Alternative effluent management approaches were also considered as part of this study, including delivering reclaimed water to West Sedona for use in the park's irrigation and implementing advanced water purification (AWP) treatment and delivery to produce a drought-resilient potable water supply.

The purpose of the Wastewater Reclamation Plant Facility Plan (Facility Plan) project is to complete a feasibility phase study that assists the City with evaluating technical feasibility and planning level costs associated with each treatment and delivery alternative considered to inform City capital improvements planning.

1.2 References to Prior Work

The WWRP was originally constructed in 1990 as a sequencing batch reactor facility. Since its original construction in 1990, the WWRP has undergone several major projects, including:

1. Addition of automatic backwash filters (1995).
2. Effluent pump station upgrades (1995, 1996).
3. Reservoirs (1996)
4. Upgrade to conventional activated sludge process (1999).
5. Effluent disposal design modifications (2005).
6. Solar drying bed improvements (2010).
7. Ultraviolet (UV) disinfection and Wetlands upgrades (2011).
8. WWRP Class A+ upgrades (2015).
9. WWRP headworks bar screen replacement 2018.
10. Injection wells (2017).
11. Tertiary filter upgrades (2019).
12. WWRP grit classifier replacement 2020.

Additionally, a plant capacity evaluation was performed as part of the WWRP A+ Upgrades project, and included in the Design Concept Report (Carollo Engineers [Carollo], May 2013). This capacity evaluation was revisited and updated by Carollo in 2017. Due to another increase in influent strength, the plant capacity was re-evaluated as part of this Facility Plan.

1.3 Overview of Existing Facilities

A site plan of the existing facilities is included in Appendix A. The WWRP receives wastewater from the City's wastewater collection system. Raw sewage is pumped to the headworks of the WWRP. The septage facility consists of a receiving station, a septage holding tank, two transfer pumps, a level control system, and a flow meter. Septage flows by gravity from the receiving station into the holding tank, which is equipped with submersible centrifugal chopper pumps. Wastewater then flows through two influent screening channels, where coarse bar screens are used to remove large debris. After passing through the screens, the wastewater flows through the Parshall flume for flow measurement before it enters the vortex grit removal system. Here, the wastewater enters tangentially, creating a vortex that allows grit to settle by gravity into a separator hopper while effluent exits from the top of the tank.

Wastewater then flows by gravity to the aeration basins, which are designed to allow plug flow or step-feed operation. Each aeration basin includes two anoxic zones, followed by one aerobic zone to allow nitrification and denitrification. Mixed liquor from the aeration basins then flows by gravity to circular secondary clarifiers (SC) where solids settle and are pumped out of the system by a set of return-activated sludge/waste-activated sludge (RAS/WAS) pumps. From the clarifiers, secondary effluent flows to cloth media disk filters located in the filter complex. The effluent from the filtration system flows through parallel UV channels for disinfection. The UV system is currently in the process of being replaced with a new system. The final effluent is reused within the vicinity of the WWRP for non-potable water uses, including discharge to the Sedona Preserve Wetlands, injection to the aquifer via two recharge wells, reclaimed water, and spray irrigation.

Solids are removed from wastewater at various stages of the treatment processes. In preliminary treatment, grit and large debris are removed and disposed of in a landfill. WAS from SCs is pumped to the aerobic digesters for stabilization. The digesters also serve as sludge-holding basins, providing retention time before the sludge is either dewatered by centrifuges or sent to solar drying beds on site. Dewatered sludge from both the drying beds and centrifuges is hauled off-site for landfill disposal.

CHAPTER 2 REGULATORY SUMMARY

The WWRP currently operates under an Aquifer Protection Permit (APP), Underground Storage Facility (USF) Permit, Water Storage (WS) Permit, and Type 3 Recycled Water General Permit and produces Class A+ reclaimed water. Treatment requirements for alternatives considering AWP or PFAS treatment must comply with applicable regulatory requirements as outlined in this section. Arizona's water statutes are complex but necessary to help protect and efficiently use this valuable resource. The combination of statutes, rules, court decisions, and other legal agreements collectively govern the quantity of water available, how it can be conveyed, and how and where it can be used by the City.

2.1 APP Regulations

In Arizona, an APP is required for any facility that discharges a pollutant either directly to an aquifer or to the land surface or the vadose zone (the area between an aquifer and the land surface) in such a manner that there is a reasonable probability that the pollutant will reach an aquifer. Wastewater treatment facilities and injection wells are discharging pollutants and, therefore, require permits. Aquifer Water Quality Standards (AWQS) are outlined in Arizona Administrative Code (A.A.C.) Title 18, Chapter 11, Article 4. The article provides numeric water quality standards for discharges to aquifers that are designated for drinking water-protected uses. The standards establish water quality standards for inorganics, metals, organics, pesticides, polychlorinated biphenyls, and radionuclides. AWQS applies to aquifers that are classified for drinking water-protected use. There are numerous requirements specified in the A.A.C regarding aquifer protection; however, the following are the most critical:

- The best available demonstrated control technology (BADCT) must be used by the facility (see Section 2.2.1.1).
- The facility must demonstrate that AWQS will not be violated in the aquifer at a point of compliance (POC) because of discharge from the facility. If the level of a pollutant in the aquifer already exceeds the AWQS at the time of permit issuance, the aquifer must not be further degraded.

The City currently operates under APP permit no. P-102298.

2.2 Arizona Department of Environmental Quality and Arizona Department of Water Resources Regulations for Non-potable Reuse and Indirect Potable Reuse

2.2.1 Non-Potable Reuse

In the state of Arizona, non-potable reuse applications are restricted based on the Class of reclaimed water produced by the WWRP. The Sedona WWRP produces Class A+ effluent, for which requirements are summarized in the following section.

2.2.1.1 ADEQ BADCT Requirements (A+ Effluent)

Arizona Department of Environmental Quality (ADEQ) sets forth the regulations pertaining to wastewater treatment effluent quality and effluent management in Arizona. The ADEQ rules require that wastewater treatment plants in the State of Arizona must meet the conditions of BADCT. Treated effluent must (at a minimum) meet or exceed the current standards set forth in the A.A.C., specifically as defined in R18-9 and R18-11. The BADCT treatment performance requirements are presented in Table 1.

The ADEQ reuse regulations categorize reclaimed water into three main classes: A, B, or C effluent. In addition, if nitrogen removal is provided, then the water can be classified as A+ or B+. Class A+ water can be used for a wide variety of non-potable uses, including food crop and residential/school irrigation, toilet and urinal flushing, fire protection systems, commercial closed-loop air conditioning systems, vehicle and equipment washing, and snowmaking. The Sedona WWRP is currently permitted to produce Class A+ quality effluent, and currently disposes of treated effluent to wetlands, groundwater recharge, and for irrigation use. Reclaimed water is available for public purchase at the WWRP and must be hauled off. Reclaimed water quality standards are summarized in Table 2.

Table 1 Arizona Regulated Wastewater Treatment Requirements for BADCT

Regulated Parameter	Limit (mg/L)
BOD ₅	30 mg/L (30-day average) 45 mg/L (7-day average)
TSS	30 mg/L (30-day average) 45 mg/L (7-day average)
pH	6.0 - 9.0
Removal efficiency for BOD ₅ , CBOD ₅ and TSS	85%
TN	10 mg/L (5-month geometric mean)
Fecal coliform or <i>E. coli</i> bacteria	Non-detectable (4 of 7 daily samples)
Fecal coliform	23 CFU/100 mL (single sample max)
<i>E. coli</i> bacteria	15 CFU/100 mL (single sample max)

Source: A.A.C. R18-9-B204, September 30, 2019

BOD₅ - 5-day biochemical oxygen demand; TSS - total suspended solids; pH - potential of hydrogen;
CBOD₅ - 5-day carbonaceous biochemical oxygen demand; TN - total nitrogen; *E. Coli* - *Escherichia coli*;
mg/L - milligrams per liter; CFU - colony forming units; mL - milliliter

Table 2 ADEQ Reclaimed Water Quality Standards WWRP Evaluation City of Sedona, Arizona

Parameter	Effluent Limits		
	Class A+ ⁽¹⁾	Class B+ ⁽²⁾	Class C ⁽³⁾
Secondary treatment	X	X	Stabilization ponds with 20-day detention
Filtration	X	NR	NR
Denitrification	X	X	NR
Disinfection	X	X	With or without
TN (as N)(4)	< 10 mg/L	< 10 mg/L	N/A
Turbidity			
▪ Daily (24-hour) average	▪ 2 NTU	▪ N/A	▪ N/A
▪ Single sample maximum	▪ 5 NTU	▪ N/A	▪ N/A
Fecal Coliform			
▪ Single sample maximum	▪ < 23 cfu/100 mL	▪ < 800 cfu/100 mL	▪ < 4,000 cfu/100 mL
▪ Four out of last seven daily samples	▪ ND	▪ < 200 cfu/100 mL	▪ < 1,000 cfu/100 mL

Notes:

X - required; ND – non detect; NR - not required; NTU - nephelometric turbidity unit

(1) Reference: A.A.C. R18-11-303.

(2) Reference: A.A.C. R18-11-305.

(3) Reference: A.A.C. R18-11-307.

(4) Five sample geometric mean.

2.2.2 Aquifer Recovery and Aquifer Storage and Recovery

In 1986, the State of Arizona promulgated the Underground Water Storage and Recovery Act (UWSRA) to regulate recharge, storage, and recovery of all classes of water. The UWSRA allows renewable supplies to be recharged and stored underground to meet future demands and encourages the use of excess renewable supplies, such as reclaimed water, that would otherwise remain unused.

Underground storage projects involve two types of permits, the USF permit and the WS permit. The USF permit regulates the "how" of underground storage, and the WS permit is issued for the purposes of accruing and accounting for credits. The USF covers two types of physical facilities: constructed and managed. Constructed facilities include facilities specifically built for facilitating storage, such as spreading basins, aquifer storage recovery wells, and vadose zone wells. The managed facility permit applies when water is discharged to a natural stream and water is allowed to infiltrate through natural processes. Sedona currently has both constructed facilities' USF and WS permits. Storage and recovery can be used as an approach to indirectly convert reclaimed water to drinking water without the expense of direct potable reuse.

2.3 PFAS National Primary Drinking Water Regulations

On April 10, 2024, the United States Environmental Protection Agency (USEPA) finalized National Primary Drinking Water Regulations establishing legally enforceable MCL for six PFAS compounds in drinking water. Although these regulations do not directly relate to wastewater treatment, the AWQs that are included in APPs refer to drinking water MCLs. Therefore, it is expected that the drinking water MCLs will indirectly impact wastewater treatment in Arizona. As of the date of this study, the State of Arizona has not begun the rulemaking process to incorporate the PFAS MCLs into the AWQs, but that process is expected to begin within 2025. As such, the City is proactively considering the impact of these regulations on the WWRP operations. In lieu of enforceable wastewater standards for PFAS, treatability alternatives covered under Section 4.4 will assume compliance with the drinking water MCLs.

The 2024 National Primary Drinking Water Regulations established individual MCLs for perfluorooctanoic acid (PFOA), perfluorooctyl sulfonate (PFOS), perfluorohexanesulfonic acid (PFHxS), perfluorononanoic acid (PFNA), hexafluoropropylene oxide dimer acid (commonly known as GenX chemicals), and a regulated Hazard Index (HI), which is a PFAS mixture containing at least two or more of PFHxS, PFNA, GenX, and perfluorobutane sulfonic acid (PFBS), to account for combined and co-occurring levels. The formula for calculating the HI, shown in Figure 1, involves dividing the concentration of each of the four individual PFAS compounds by their individual Health-based Water Concentration reference value (i.e., the level at which USEPA projects that no health effects are expected for that PFAS), and then summing these four values. When calculating the HI, all analysis results with no reportable PFAS are assumed to be zero. USEPA also finalized health-based, non-enforceable maximum contaminant level goals (MCLG) for these PFAS compounds. Table 3 summarizes the MCLs and MCLGs set by the USEPA for all regulated PFAS compounds.

$$HI = \frac{[PFHxS]}{10 \text{ ppt}} + \frac{[GenX]}{10 \text{ ppt}} + \frac{[PFNA]}{10 \text{ ppt}} + \frac{[PFBS]}{2,000 \text{ ppt}}$$

Figure 1 Formula for Calculating the HI

Table 3 USEPA MCLs for PFAS

Compound/Parameter	Final Health-Based MCLG	Final MCL (enforceable levels)
PFOA	0 ppt	4 ppt
PFOS	0 ppt	4 ppt
HI	1 (unitless)	1 (unitless)
PFHxS	10 ppt	10 ppt
GenX	10 ppt	10 ppt
PFNA	10 ppt	10 ppt
PFBS	--	--

ppt - parts per trillion

The Drinking Water Health Advisories for GenX and PFBS were finalized in 2022 and are 10 ppt and 2,000 ppt, respectively. Additionally, on April 19, 2024, USEPA issued a final rule to designate PFOA and PFOS as hazardous substances under the Comprehensive Environmental Response, Compensation, and Liability Act.

2.4 ADEQ AWP Rules

ADEQ has recently finalized the rules for the AWP Program, which outline requirements for implementation and permitting, advanced water treatment (AWT), pathogen and chemical removal standards, enhanced source control, operational requirements, operator certification, monitoring and reporting, and public communication. The reference standards for the main requirements of the rules are presented in Table 4. The proposed AWP approach in this study follows the requirements of the AWP rules.

Table 4 Summary of AWP Requirement from ADEQ AWP Proposed Rules

Requirement	Reference
Communications Plan	R18-9-B811
Tier 1, 2, and 3 Chemical Lists	R18-9-E825, R18-9-E826, R18-9-E827,
Identification of Critical Control Points	R18-9-C827, subsection (D)
Enhanced Source Water Control Plan	R18-9-C824
Initial Source Water Characterization (ISWC) Plan	R18-9-C814
ISWC Sampling Completed Over One Year and Report	R18-9-C814
Pilot Study Plan	R18-9-C815, subsection (B)
Pilot Testing and Report	R18-9-C815, subsections (C), (D)
Permitting for AWP Demonstration and Full-Scale Project	R18-9-F817 to R18-9-F823
Full-scale Verification	R18-9-F835

CHAPTER 3 CAPACITY EVALUATION

3.1 Basis of Capacity Evaluation and Historical Plant Data

Development of the capacity evaluation included a review of historical plant data. The capacity of unit processes such as preliminary treatment (i.e., influent pumping, screening, and grit removal) and tertiary treatment (disinfection) are primarily governed by hydraulic peak flows. The capacity of the secondary treatment process and solids handling is primarily governed by wastewater loadings and is, therefore, heavily impacted by the wastewater constituent concentrations in the raw wastewater.

This chapter provides the flow and water quality data utilized for the rest of the evaluation, including daily influent flows, wastewater influent characteristics, wastewater effluent characteristics, and other applicable process data. The flow and process data were used for the Capacity Evaluation included in Chapter 4. Recent PFAS data and a summary of supplemental sampling are also summarized in this chapter and were used to inform the treatment evaluation included in Chapters 4 and 5.

3.1.1 Influent and Effluent Flows

Historical flow data was summarized for use in the capacity evaluation and development of treatment and delivery alternatives. The analyzed dataset ranged from January 2020 to July 2024 and included daily influent flows, wastewater influent characteristics, wastewater effluent characteristics, and other process data.

Influent wastewater flow peaking factors used for the capacity evaluation and development of treatment and delivery alternatives include:

- Maximum monthly average daily flow (MMADF) peaking factor - calculated as the ratio between the maximum 30-day average flow and the annual average daily flow (AADF).
- Peak daily flow (PDF) peaking factor - calculated as the ratio between the maximum daily flow and the AADF.
- Peak hour flow (PHF) peaking factor - calculated as the ratio between the maximum hourly flow and the AADF.

Figure 2 shows daily influent flows to the WWRP. The influent flows were analyzed to obtain an AADF over the entire analysis period (2020 to 2024) and to evaluate trends in the influent flows to the WWRP. A 30-day running average of the daily influent flows was calculated to evaluate maximum month flows. The highest MMADF was observed in March 2023, with a value of 1.38 million gallons per day (mgd), although the overall flow showed seasonal variation and an MMADF peaking factor of 1.12 for the full dataset. There was a significant decrease in WWRP influent flows during the spring of 2020, which was likely due to the COVID-19 lockdown and the resulting decrease in tourism in Sedona.

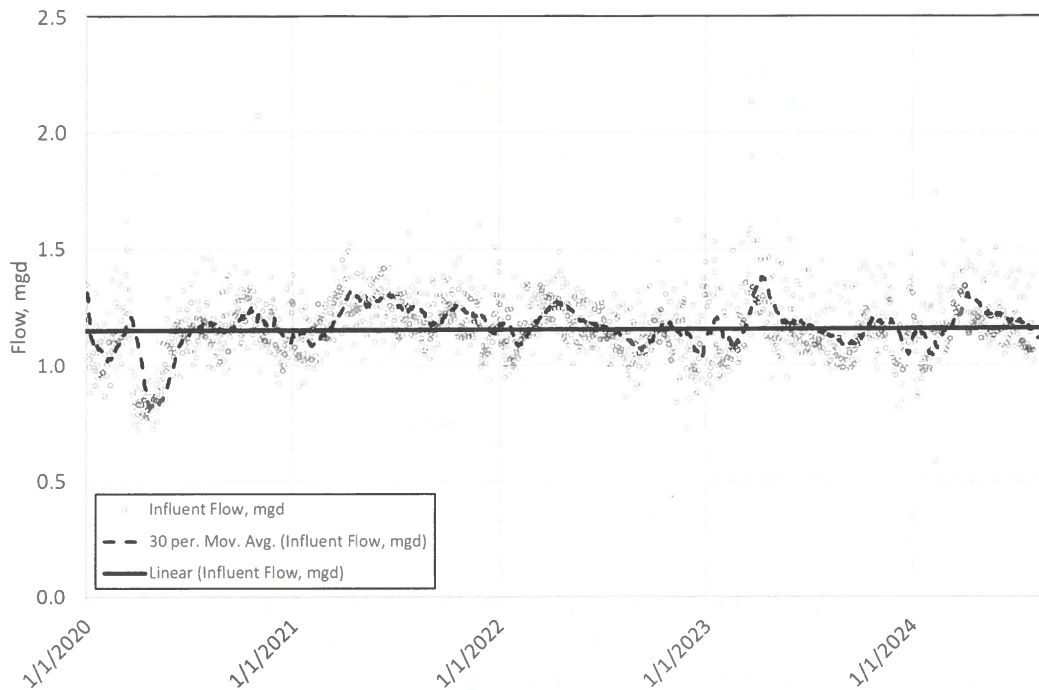


Figure 2 Sedona WWRP Daily Influent Flow (January 2020 to July 2024)

Table 5 summarizes the AADF, MMADF, PDF, and PHF flows and peaking factors for the period analyzed. The MMADF, PDF, and PHF were calculated using a value of 1.16 mgd for the AADF times the respective peaking factors. As part of this study, a peak hour peaking factor (PHPF) analysis was conducted using flow data from two dates in 2024. However, due to the lack of available data for higher flow days observed in previous years, it was determined that there was insufficient information to reduce the PHPF, and the peaking factor of 2.3 was maintained from the 2017 capacity evaluation.

Table 5 Influent Flows and Peaking Factors (January 2020 to July 2024)

Condition	Flow, mgd	Buildout Flow, mgd	Peaking Factor
AADF	1.16	1.59	---
MMADF	1.30	1.78	1.12
PDF	1.83	2.51	1.58
PHF	2.68	3.66	2.30 ⁽¹⁾

Notes:

(1) Due to insufficient recent data, the PHPF of 2.3 from the 2017 capacity evaluation was used for this evaluation.

3.1.1.1 Existing Effluent Management

Effluent flows are distributed between existing effluent management facilities, including: (1) storage in Reservoir 3 for eventual discharge via spray irrigation in Area 4, (2) discharge to the Sedona Wetlands Preserve, (3) aquifer injection via two existing recharge wells, and (4) spray irrigation of Areas 1 to 3. A schematic of the existing effluent uses is shown in Figure 3.

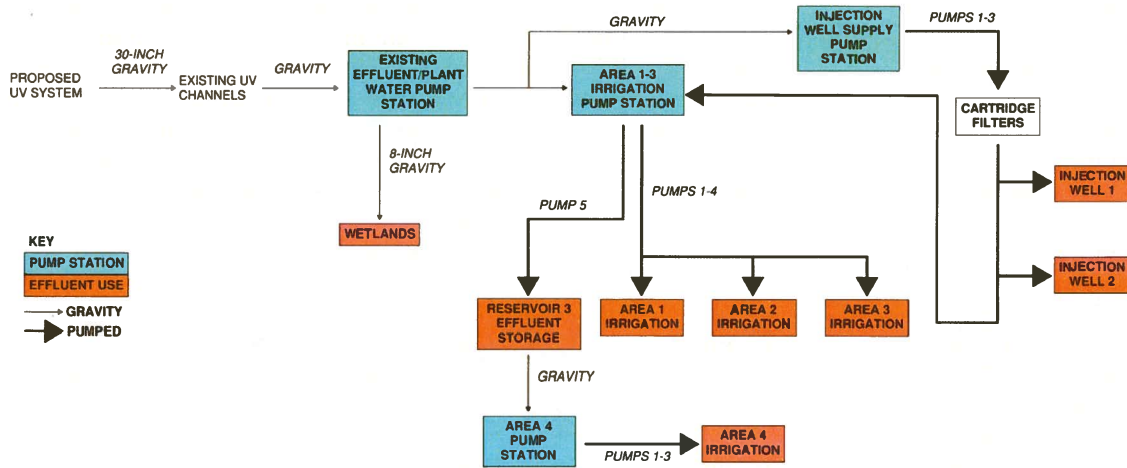


Figure 3 Existing WWRP Effluent Management Schematic

Monthly average flows to each use are plotted with monthly average WWRP influent flow in Figure 4 for months between July 2022 and July 2024. The difference between WWRP influent flow and the sum of effluent uses seen during the winter months in Figure 4 is due to storage in Reservoir 3. Water is stored in Reservoir 3 primarily during the winter months and then used to irrigate Area 4 in the late spring and early summer. This analysis will be revisited in Chapter 4 and Section 5.2.1 for the development of AWP treatment flow capacity scenarios.

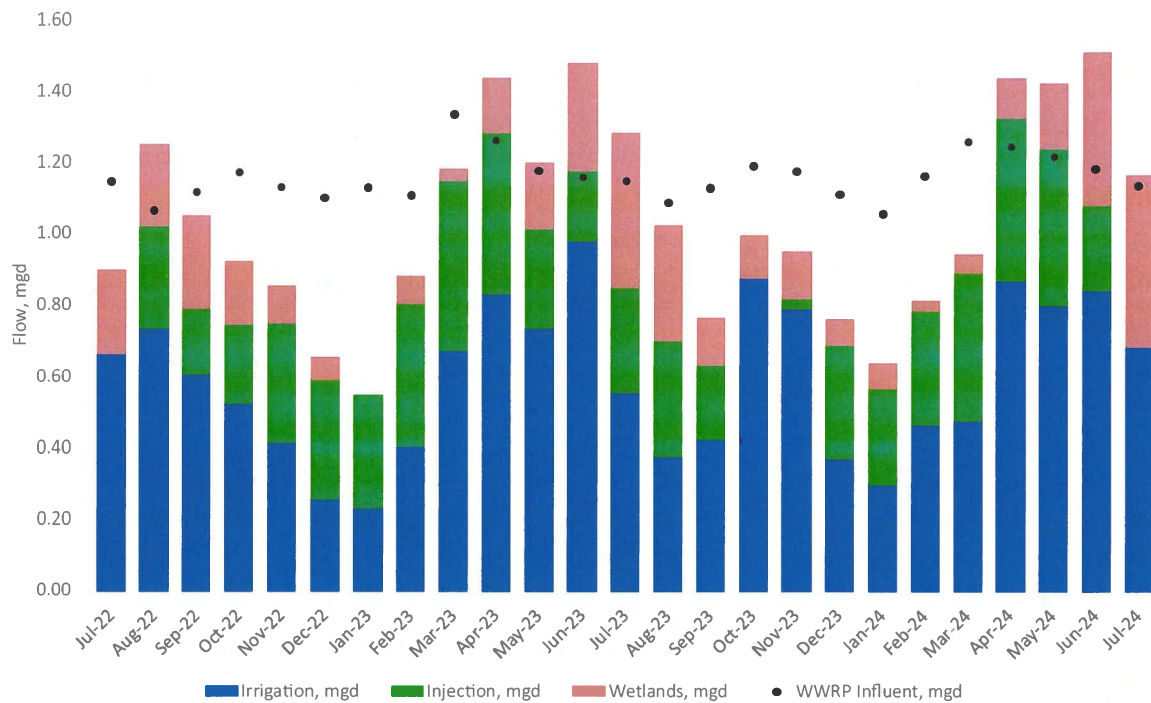


Figure 4 Monthly Average Effluent Flows by Use (July 2022 to July 2024)

3.1.2 Influent Concentrations and Loads

Influent wastewater quality parameters were collected and reviewed as part of the current evaluation. The influent wastewater characteristics were evaluated to determine the average wastewater constituent concentrations and the mass load peaking factors. These parameters drive the capacity of the secondary treatment and solids handling systems and are critical elements of the plant capacity evaluation. The measured parameters included chemical oxygen demand (COD), TSS, volatile suspended solids (VSS) and ammonia nitrogen (NH₃-N).

Influent mass load peaking factors and concentrations are summarized in Table 6 and Table 7, respectively. Subsequent Figures Figure 5 through Figure 8 show each parameter graphically. General observations are as follows:

- Influent COD concentrations and loads are presented in Figure 5. For the period evaluated between 2020 to 2024, the annual average influent COD concentration was 917 mg/L. The annual average daily load (AADL) and maximum monthly average daily load (MMADL) were 8,900 pounds per day (lb/day) and 11,600 lb/day, respectively. The highest daily COD load reported was 16,100 lb/day in March 2023.
- Influent TSS concentrations and loads are presented in Figure 6. For the period evaluated between 2020 to 2024, the annual average influent TSS concentration was 414 mg/L. The AADL and MMADL were 4,000 lb/day and 5,700 lb/day, respectively.
- Influent VSS concentrations and loads are presented in Figure 7. For the period evaluated between 2020 to 2024, the annual average influent VSS concentration was 378 mg/L. The AADL and MMADL were 3,700 lb/day and 5,200 lb/day, respectively.
- Influent NH₃-N concentrations and loads are presented in Figure 8. For the period evaluated between 2020 to 2024, the annual average influent NH₃-N concentration was 42 mg/L. The AADL and MMADL were 410 lb/day and 560 lb/day, respectively.

Table 6 Mass Loading Peaking Factors

Parameter	MMADL Peaking Factor
COD	1.31
TSS	1.41
NH ₃ -N	1.35

Table 7 Influent Wastewater Concentrations at Average Annual and Maximum Monthly Loadings

Parameter	Concentration at AADL, mg/L	Concentration at MMADL, mg/L
COD	917	1070
TSS	414	520
VSS	378	475
NH ₃ -N	42	51

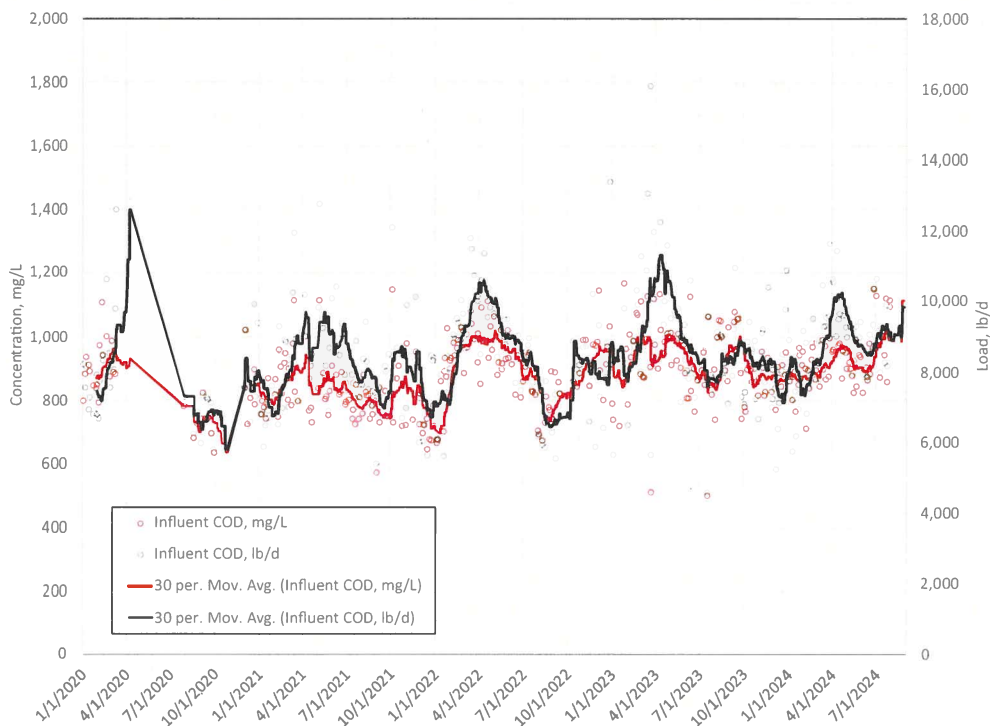


Figure 5 Sedona WWRP Influent COD Concentration and Load (January 2020 to July 2024)

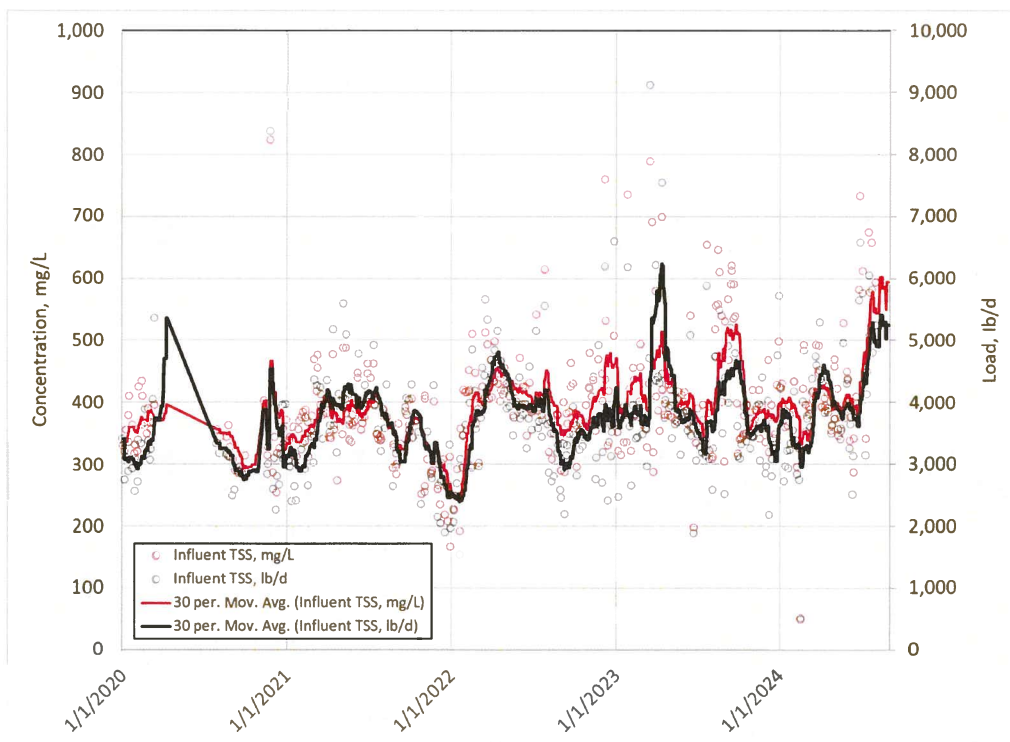


Figure 6 Sedona WWRP Influent TSS Concentration and Load (January 2020 to July 2024)

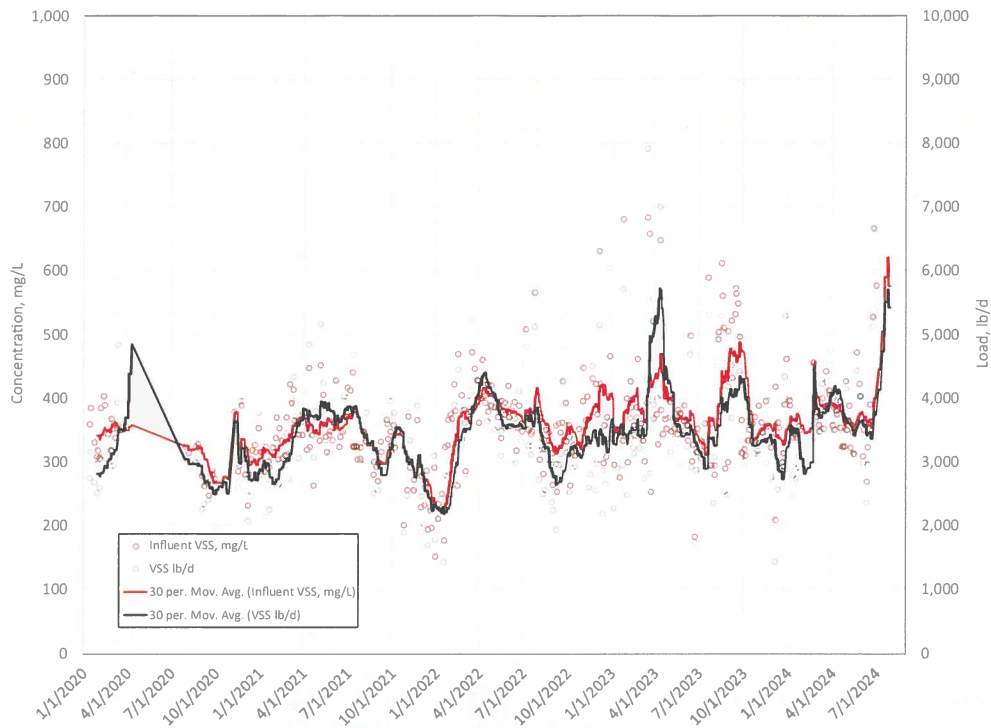


Figure 7 Sedona WWRP Influent VSS Concentration and Load (January 2020 to July 2024)

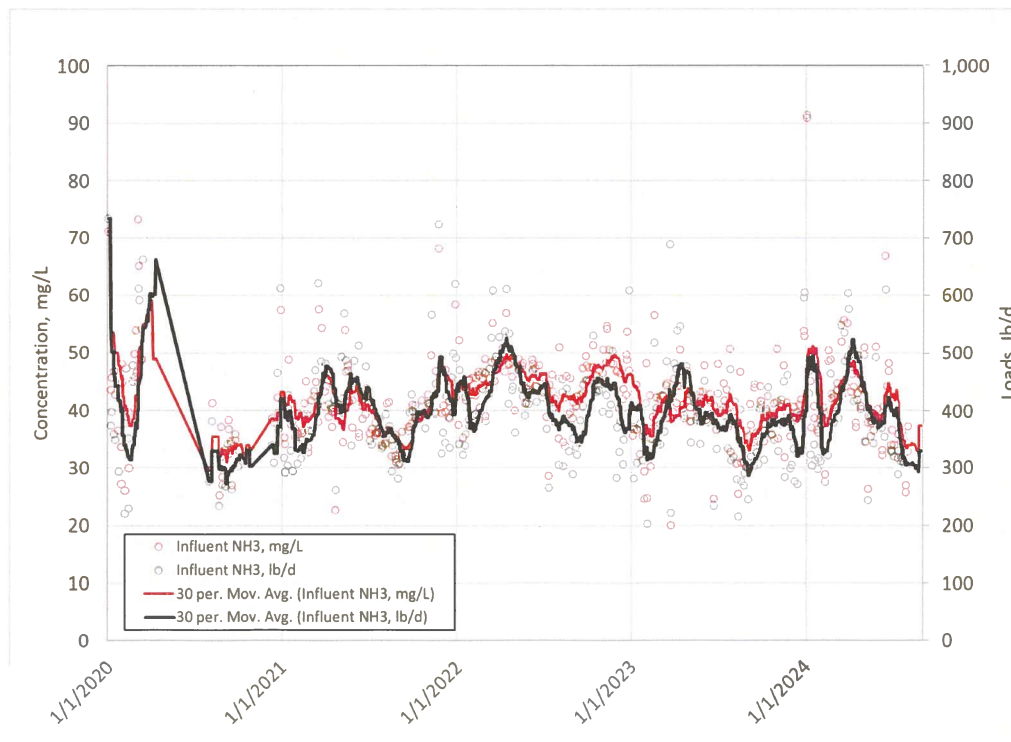


Figure 8 Sedona WWRP Influent NH3-N Concentration and Load (January 2020 to July 2024)

3.1.3 Effluent Water Quality

Wastewater effluent parameters were summarized and analyzed to evaluate process performance. The measured parameters included effluent COD, TSS, turbidity, $\text{NH}_3\text{-N}$, nitrate, and nitrite. These parameters reflect treatment efficiency and overall effluent quality. Monitoring these parameters provides the opportunity to evaluate opportunities for treatment optimization, in addition to tracking permit compliance.

Effluent COD concentrations are shown in Figure 9. The average effluent COD concentration was 100 mg/L. The COD concentration remained relatively consistent through the dataset with an average COD removal of 88 percent across the WWRP.

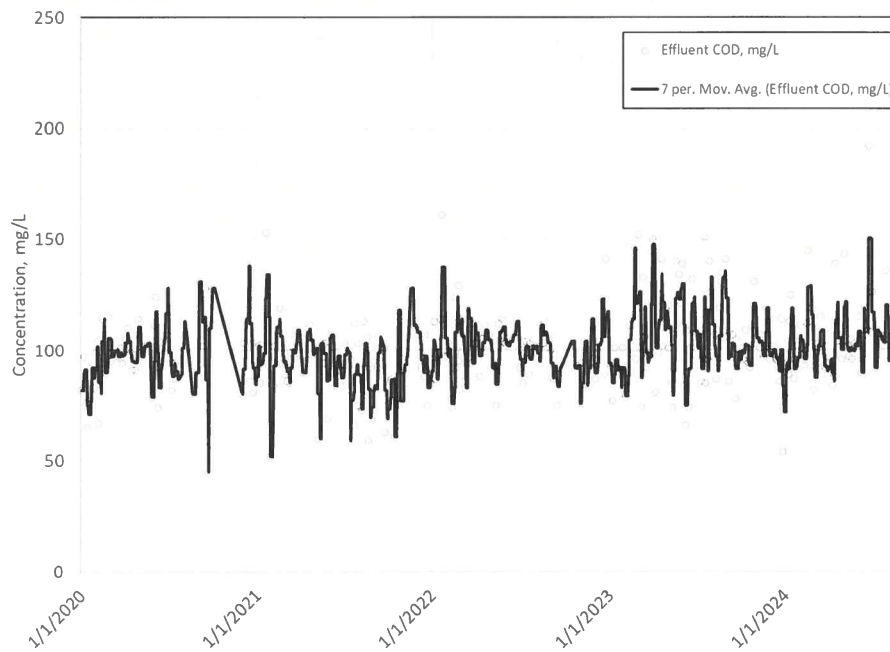


Figure 9 Effluent COD Concentrations (January 2020 to July 2024)

Effluent TSS concentrations are shown in Figure 10. The highest TSS concentrations occurred in July 2020 and June 2024. Overall, TSS concentrations remained relatively consistent with the average effluent TSS concentration of approximately 1.6 mg/L. The effluent TSS concentrations are consistently well below BADCT requirements (Table 1) for TSS of 30 mg/L on a 30-day average basis and 45 mg/L on a 7-day average basis. Additionally, the effluent TSS concentrations are consistently below the design TSS value of 5 mg/L for the existing UV system, with only two notable design exceedances observed in the data set during July 2020 (9 mg/L) and June 2024 (7 mg/L).

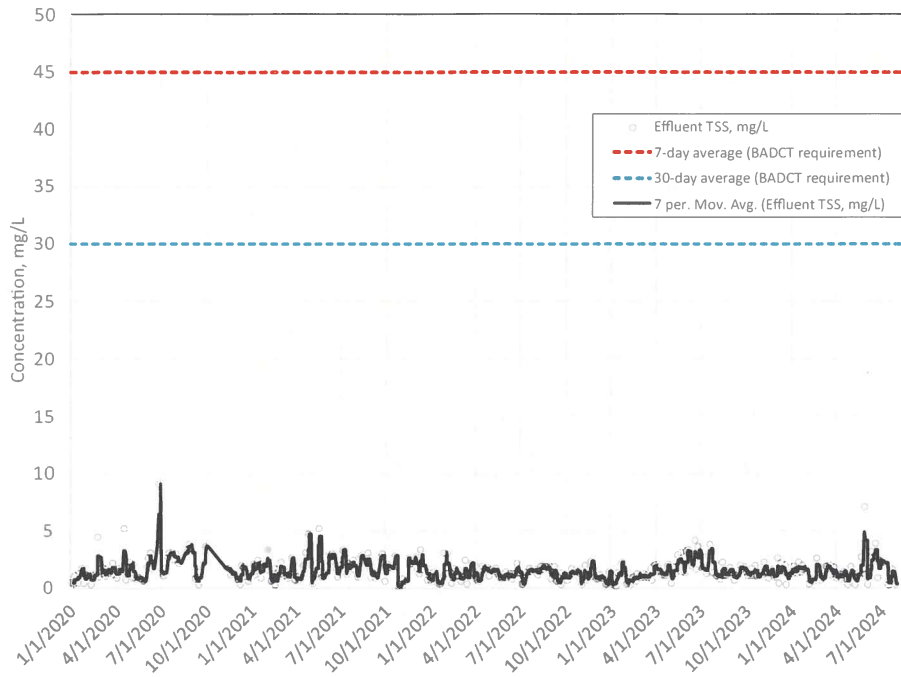


Figure 10 Effluent TSS Concentration (January 2020 to July 2024)

Effluent turbidity for filter 2A is presented in Figure 11. There are three cloth disk filters installed, but Filter 2A has been the most frequently used. Although the period evaluated was 2020 to 2024, there was a gap between May 2020 and January 2021. For the data available, Filter 2A generally treated water to effluent turbidity in the range of 0.6 NTU, well below the Class A+ requirement (Table 2) of 5 NTU for a single sample maximum and generally within the range of 2 NTU on a 24-hour average basis.

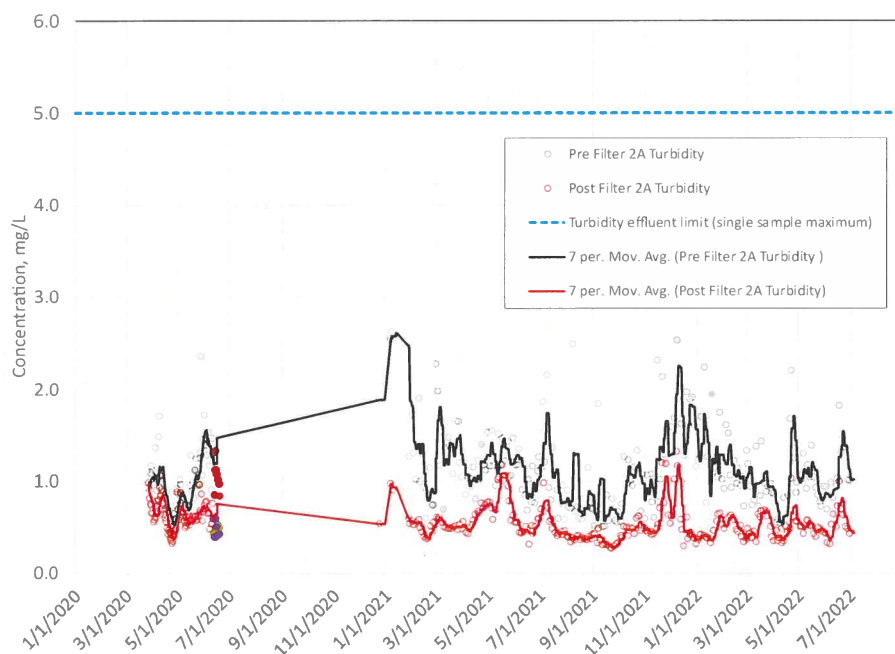


Figure 11 Filter 2A Turbidity Concentration (January 2020 to July 2022)

Nitrate, nitrite, ammonia, and total inorganic nitrogen (TIN) are presented in Figure 12 and Figure 13. TIN is the sum of NH_3 , nitrogen oxides (NO_3), and NO_2 . Nitrification is a crucial biological process that converts ammonia to nitrite and then to nitrate, while denitrification transforms nitrate into nitrogen gas. Both processes are important in evaluating wastewater treatment performance as they provide nitrogen removal across the facility. TN is comprised of TIN plus organic nitrogen, which is typically present in the range of 1 to 1.5 mg/L in municipal wastewater.

The analysis covered the period from 2020 to 2024, where the average influent NH_3 concentration was 41 mg/L and average effluent TIN concentration was 2.23 mg/L. Figure 12 shows elevated NO_2 and NH_3 during winter months, most notably in 2022 and 2024, indicating incomplete nitrification, which was likely due to low wastewater temperatures as nitrification is a temperature-dependent process. Outside of noted winter periods, NO_2 and NH_3 concentrations remained low and consistent, along with effluent TIN, indicating excellent TN removal with both full nitrification and strong denitrification performance. Based on the available data for TIN and typical values for organic nitrogen, the WWRP effluent meets the 10 mg/L TN requirement of the APP permit for Class A+ effluent.

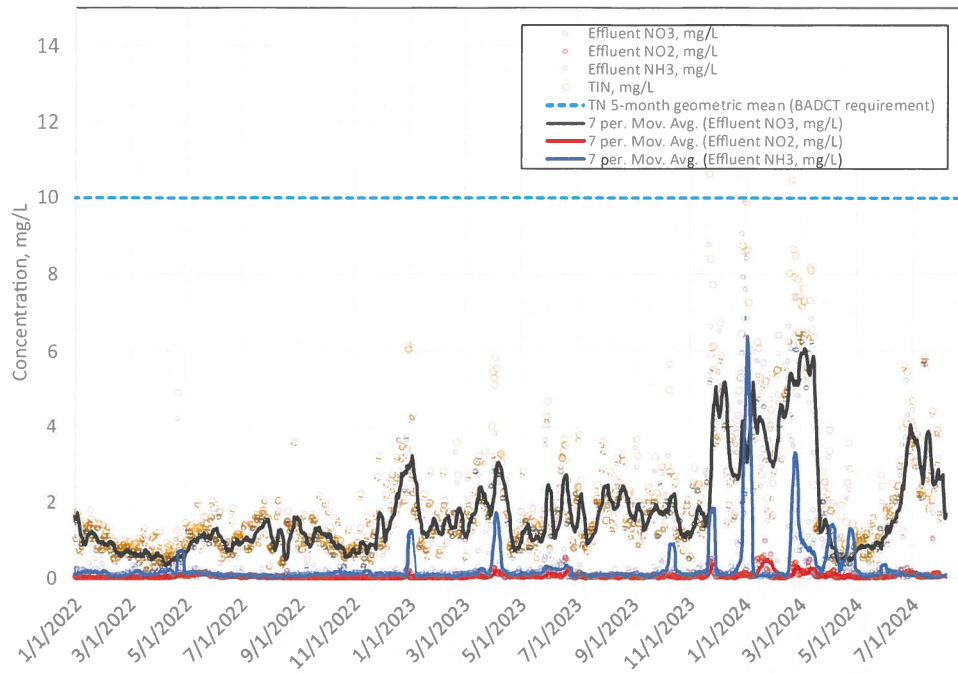


Figure 12 Effluent Nitrogen Concentrations (January 2020 to July 2024)

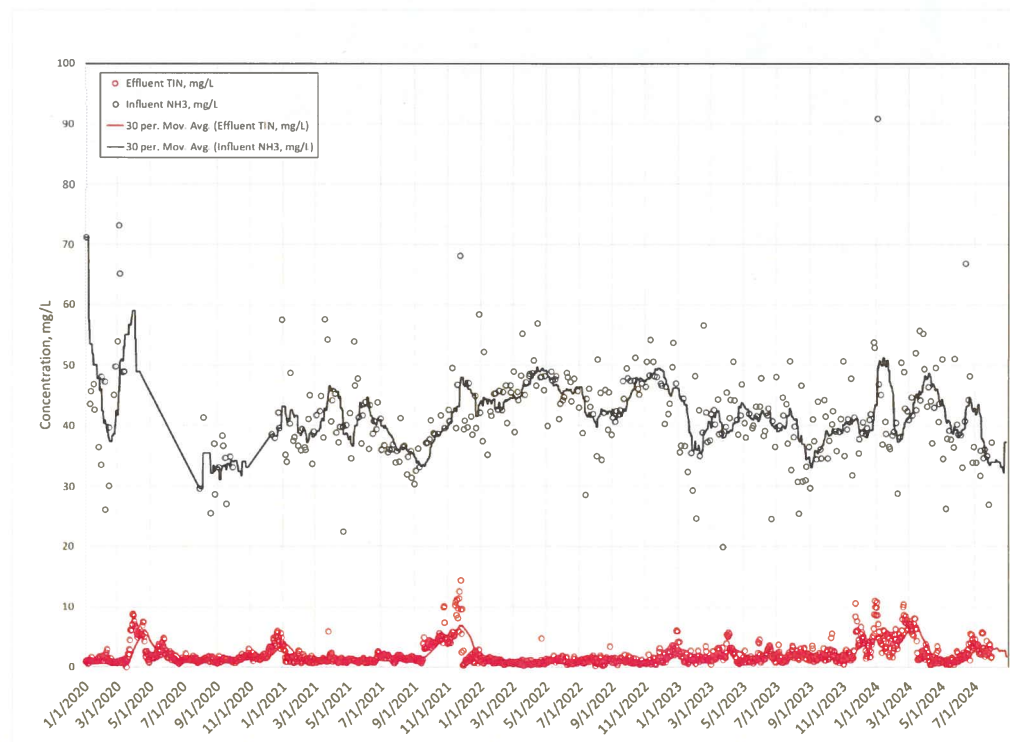


Figure 13 Influent Ammonia and Effluent TIN Concentrations (January 2020 to July 2024)

3.1.4 Influent Concentration Trends

Influent concentrations for various parameters were evaluated for wastewater data collected between 2010 and 2024, as summarized in Chapter 3 of this report. A previous plant capacity evaluation was performed as part of the WWRP A+ Upgrades project and included in the Design Concept Report (Carollo, May 2013). This capacity evaluation was revisited and updated by Carollo in 2017. Due to increases in influent concentrations, the plant capacity was re-evaluated as part of this Facility Plan.

The wastewater parameters evaluated included the concentrations of COD, TSS, VSS, and $\text{NH}_3\text{-N}$. In general, the 2010 to 2021 average concentrations were lower for all wastewater parameters compared with 2022 to 2024 average concentrations. Therefore, the water quality data from 2022-2024 was utilized for analysis, as these values more accurately represented the “strength” of influent wastewater due to its higher concentrations compared to previous years. Figures 14 through 17 show average annual influent concentrations for COD, TSS, VSS, and $\text{NH}_3\text{-N}$, respectively.

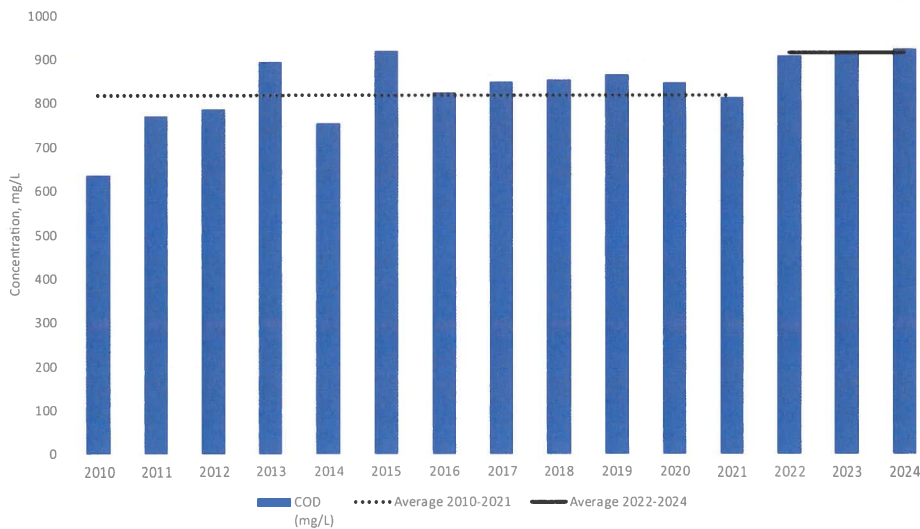


Figure 14 Influent COD Concentrations (2010 to 2024)

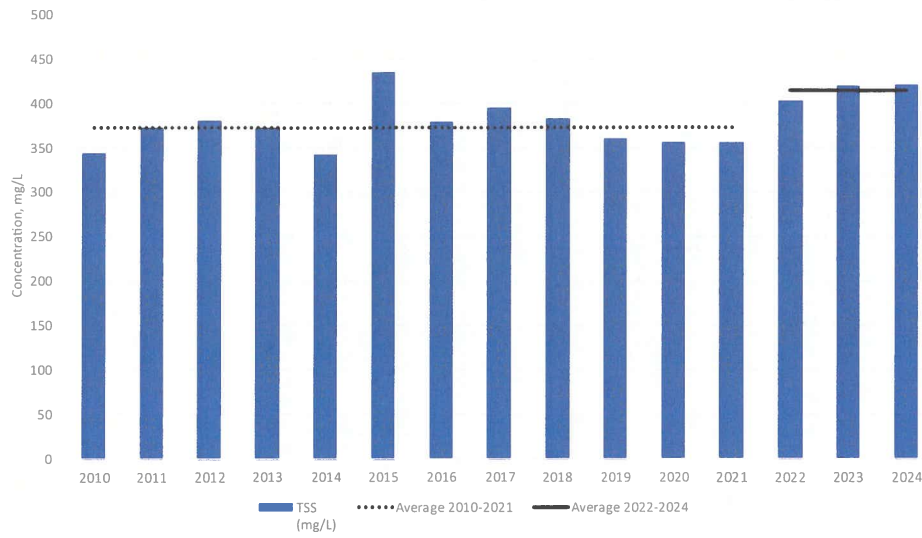


Figure 15 Influent TSS Concentrations (2010 to 2024)

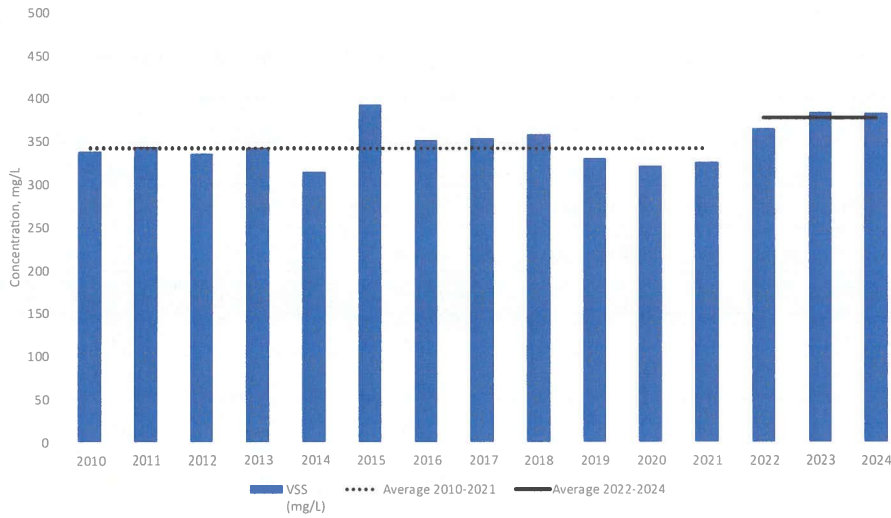


Figure 16 Influent VSS Concentrations (2010 to 2024)

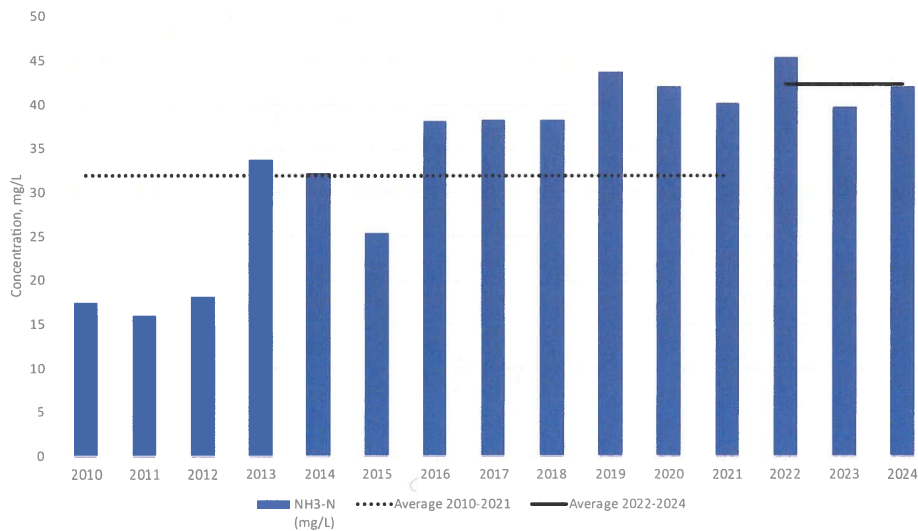


Figure 17 Influent NH₃-N Concentrations (2010 to 2024)

3.1.5 Supplemental Sampling

A supplemental sampling event was completed in Fall 2024 as part of the PFAS and AWP treatment evaluations included in Chapters 4 and 5 of this report. The purpose of this sampling event was to evaluate analytes in the secondary effluent that are not monitored as part of the plant's existing permit requirements but are important to the sizing of treatment facilities associated with the PFAS and AWP treatment trains discussed later in this report. A summary of the sample results is shown in Table 8.

Table 8 WWRP Effluent Water Quality Supplemental Sampling – November 13, 2024

Parameter	Unit	Result
Aluminum	mg/L	<0.20
Ammonia as N	mg/L	0.019
Ammonium as NH ₄	mg/L	0.04
Boron	mg/L	0.19
Bromide	mg/L	<0.50
Calcium	mg/L	54
Chloride	mg/L	157
Conductivity	uS/cm	1022
Iron	mg/L	<0.050
Magnesium	mg/L	25
Manganese	mg/L	<0.020
Nitrite	mg/L	0.09
Nitrate	mg/L	2.75
Oxidation Reduction Potential	mV	220
Orthophosphate	mg/L	1.06

Parameter	Unit	Result
pH		7.70
Potassium	mg/L	21
Total Silica	mg/L	19
Silicon	mg/L	8.7
Sodium	mg/L	110
Strontium	mg/L	0.15
Sulfate as SO ₄	mg/L	27.9
Sulfide, total	mg/L	<0.04
TDS	mg/L	623
Temperature	mg/L	20.9
TOC	mg/L	7.71
Total Alkalinity	mg/L	256

SO₄ - sulphur oxides; TOC - total organic carbon; uS/cm - microsiemens per centimeter; mV - millivolts

3.2 Secondary Treatment Process Capacity Evaluation

The purpose of the process capacity evaluation was to determine whether the existing secondary treatment process, consisting of aeration basins and SCs, has sufficient capacity to treat flows and loads through the anticipated buildout condition. Secondary treatment capacity is evaluated by balancing the biological treatment capacity of the aeration basins with the settling capacity of the SCs. A combination of data evaluation and process modeling was used to evaluate the treatment capacity, as described in the following subsections.

3.2.1 Model Setup and Calibration

A process model for the WWRP was developed using Biowin, a commercially available wastewater treatment simulation software package. The model was calibrated using historical process data from the City, adjusting internal model wastewater fractionation to match the model-predicted process performance to the operational data collected at the plant.

A schematic of the Biowin process model is shown in Figure 18.

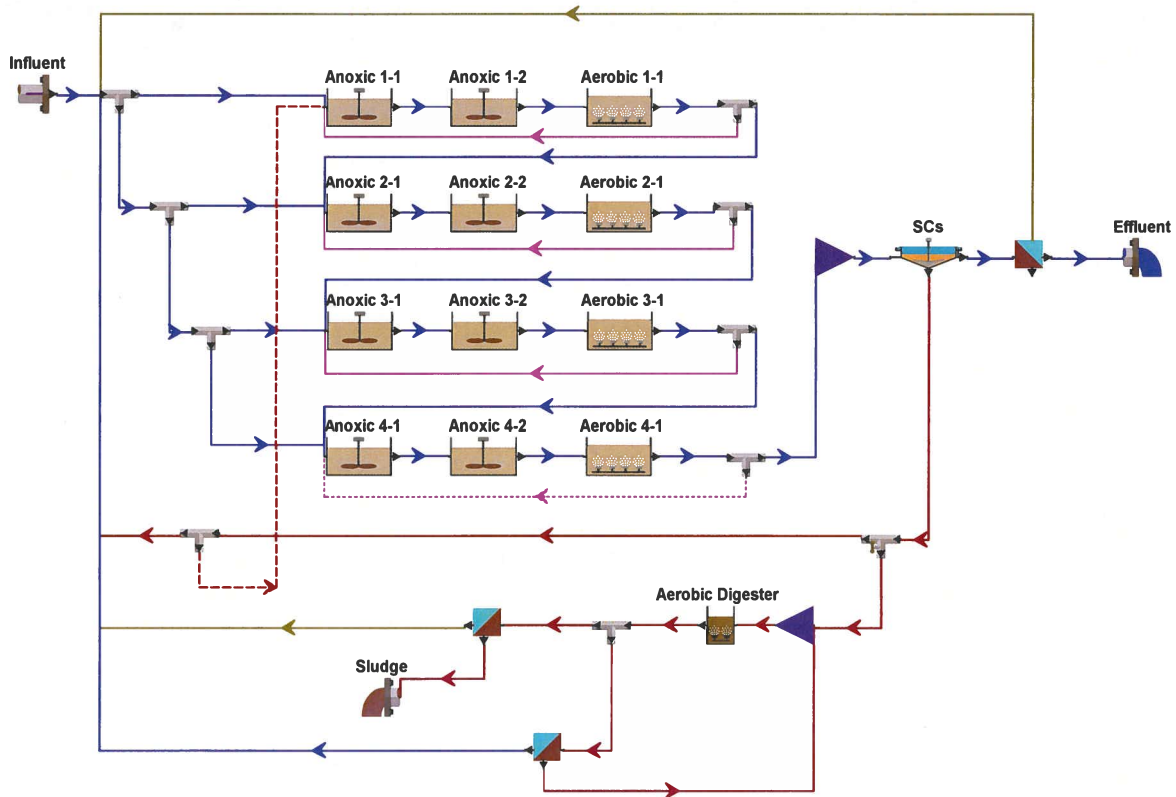


Figure 18 Biowin Process Model Schematic for the Sedona WWRP

The approach used for model calibration was to use the available plant data from 2021 to 2024 (as discussed in Section 3.1.4) as inputs to the model and compare the steady-state model predictions with annual averages of plant operating data. For the purpose of this evaluation, Biowin default values for stoichiometric and kinetic parameters were maintained. Plant data between January 2022 and December 2024 were used for the model calibration process (Table 9). This time period was selected to capture the more recent trend in increased influent concentrations. Table 9 presents the model calibration results. The model deviations were within typical industry standards. The relatively close agreement between the predicted values and the actual data averages indicates that the calibrated model can accurately predict the performance of the WWRP, which validates the simulations performed under different operating conditions and at future flows and loadings.

Table 9 Model Calibration Results

Criteria	Units	Data Average (Jan 2022 to Jul 2024)	Model Result	Model Deviation	
Influent Flow	mgd	1.16	1.16	Input	
Influent COD	mg/L	917	917	Input	
Influent TSS	mg/L	414	417	+1 percent	+3 mg/L
Influent NH ₃ -N	mg/L	42	42	Input	
WAS Flow	gpd	27,000	27,000	Input	
WAS solids	mg/L	11,135	10,719	-4 percent	-416 mg/L
	lb/day	2,500	2,400	-4 percent	-100 lb/day
MLSS	mg/L	3,857	3,922	+2 percent	+65 mg/L
Effluent NH ₃ -N	mg/L	0.32	0.14	56 percent ⁽¹⁾	-0.18 mg/L ⁽¹⁾
Effluent NO ₂ -N	mg/L	0.07	0.03	57 percent ⁽¹⁾	-0.04 mg/L ⁽¹⁾
Effluent NO ₃ -N	mg/L	1.89	1.07	43 percent ⁽¹⁾	-0.82 mg/L ⁽¹⁾
Effluent TIN	mg/L	2.28	1.24	46 percent ⁽¹⁾	-1.04 mg/L ⁽¹⁾
Digester solids	percent TS	1.6	1.7	+5 percent	+0.1 percent TS
Cake solids	wet tons	3.7	5.4	46 percent	+1.7 wet tons

Notes:

gpd – gallons per day; MLSS - mixed liquor suspended solids; NO₂-N - nitrite nitrogen; NO₃-N - nitrate nitrogen; TS - total solids

(1) Agreement between model and data is considered adequate based on the small deviation in terms of mg/L.

3.2.2 Secondary Treatment Process Capacity Evaluation Criteria

The process evaluation presented in this section is based on the following process performance criteria.

3.2.2.1 Minimum Temperature

Temperature affects the kinetics of the reactions involved in biological treatment, most importantly nitrification performance (the conversion of ammonia to nitrate). Process model simulations were performed at the plant's minimum influent average temperature of 17 degrees Celsius (°C) to account for worst-case conditions.

3.2.2.2 Solids Retention Time

The aerobic solids retention time (SRT) is an important design and operating parameter for activated sludge processes, as this parameter has a major influence on nitrification performance and nitrogen removal. Carollo's process modeling approach was to select a target minimum SRT, simulate process operation at future flows to determine the resulting MLSS and determine if the resulting MLSS can be handled by the SCs. The evaluations presented in this report were based on achieving a minimum aerobic SRT of five days at the minimum design temperature of 17 °C. Aerobic SRT values shorter than four to five days can compromise the ability of the plant to effectively and reliably remove nitrogen.

3.2.2.3 Effluent characteristics

Effluent characteristics are another important criterion in determining the capacity of the secondary process based on its ability to meet Class A+ reclaimed water quality. The WWRP's APP stipulates an effluent TN limit of 10 mg/L and an alert level of 8 mg/L, both based on a five-sample rolling geometric mean of monthly samples.

The governing criterion for this analysis was the effluent TN, which is the sum of $\text{NH}_3\text{-N}$, $\text{NO}_3\text{-N}$, $\text{NO}_2\text{-N}$, and organic nitrogen. An adequate SRT results in low effluent ammonia and nitrite concentrations. The effluent nitrate-nitrogen concentration depends on the level of denitrification achieved in the system. The denitrification capacity is directly related to the biochemical oxygen demand/total Kjeldahl nitrogen (TKN) ratio entering the activated sludge system, the anoxic volume available, and the internal mixed liquor return (IMLR) flow.

In the capacity evaluations reported in this report, a maximum TN concentration of approximately 7 mg/L was selected to provide a "buffer" of at least 1 mg/L from the alert level of 8 mg/L specified in the plant permits.

3.2.2.4 Clarifier Safety Factor

The capacity of the SCs was evaluated based on the predicted ability to produce low TSS effluent without excessive solids carryover. In contrast to using only hydraulic loading criteria to determine the capacity of the SCs, the approach presented herein considers hydraulic (i.e., flow) and solids (i.e., MLSS concentration and sludge volume index [SVI]) criteria to calculate a clarifier safety factor (CSF), which forms the basis of the clarifier evaluation.

The clarifier settling safety factor is the ratio between the maximum settling velocity of the mixed liquor and the basin overflow rate. The maximum settling velocity of the mixed liquor is determined by the settleability characteristics of the sludge and was predicted based on the SVI.

The design SVI should be conservative enough to account for less-than-ideal settling characteristics but should not be overly conservative as to account for the worst possible settling characteristics occurring at peak hydraulic conditions. For comparison, the WWRP typically has SVI values in the range of 50 to 100 milliliter per gram (mL/g), with a maximum value of approximately 200 mL/g. For this evaluation, a typical design value of 150 mL/g was chosen, which covers typical operating conditions and reflects the 98th percentile for the available dataset, as presented in Figure 19.

The minimum CSF at the PDF was established at 1.2, which includes an additional safety factor of 20 percent.

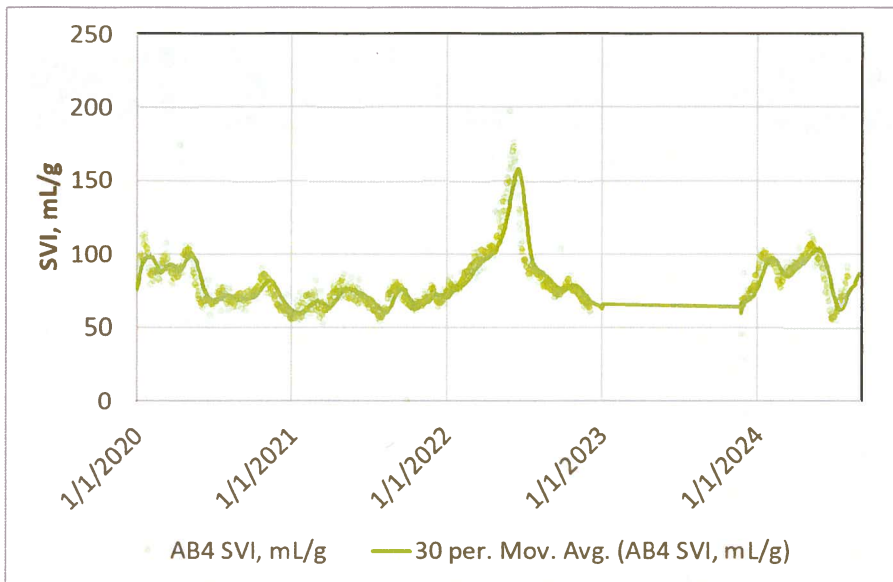


Figure 19 SVI Data (January 2020 to July 2024)

3.2.3 Secondary Treatment Capacity Evaluation Results

The secondary treatment capacity analysis was performed using the calibrated process model. Current, buildout, and maximum capacity scenarios with both AADL and MMADL conditions were evaluated. For the current and buildout scenarios, one aeration basin and one SC were assumed out of service for redundancy. For the maximum capacity scenario, all four aeration basins were assumed to be in service at MMADL, while still allowing one aeration basin to be taken out of service at AADL. For all scenarios, one redundant SC was considered out of service.

Model results are summarized in Table 10. As shown, at both current and buildout flows the CSF is well over the minimum target value of 1.2, and the aerobic SRT is well above the minimum target of days. At maximum capacity, the system would operate near the target criteria for minimum aerobic SRT (4.7 to 5 days versus target of 5 days), and minimum CSF (1.3 versus target of 1.2). All scenarios were run at MLSS concentrations similar to current operations at 3,700 to 3,800 mg/L. The predicted effluent TN for all scenarios is well below the target of 7 mg/L. Based on this evaluation, the existing aeration basins and clarifiers will be sufficient to treat flows and loads through the anticipated buildout capacity of 1.78 mgd MMADF.

Table 10 Summary of Secondary Treatment Capacity

Parameter	Unit	Current		Buildout		Capacity	
		AADL	MMADL	AADL	MMADL	AADL	MMADL
Plant Flow	mgd	1.16	1.30	1.59	1.78	1.95	2.18
Aeration Basins in Service ⁽¹⁾	#	3	3	3	3	3	4
SCs in Service ⁽²⁾	#	2	2	2	2	2	2
MLSS	mg/L	3,700	3,700	3,700	3,700	3,800	3,800
Aerobic SRT	days	14.6	8.9	9.4	5.7	4.7	5.0
SRT	days	22.2	13.6	14.4	8.7	7.1	7.5
CSF at SVI = 150 mL/g ⁽³⁾	-	2.4	2.4	1.7	1.7	1.3	1.3
Effluent TN	mg/L	3.2	3.2	2.7	3.5	3.2	3.6

Notes:

(1) The total number of aeration basins is 4.

(2) The total number of SCs is 3.

(3) The CSF is calculated at the corresponding PDF based on a PDF peaking factor of 1.5.

3.3 Solids Handling Capacity Evaluation

The existing solids handling treatment train at the Sedona WWRP includes aerobic digestion of WAS. The digested WAS (i.e., biosolids) is then dewatered using two centrifuges or distributed to solar drying beds on site. Dewatered cake from the centrifuges and drying beds is hauled to a landfill offsite.

There are two aerobic digesters at the WWRP. Digester No. 1 has a usable volume of 0.27 million gallons (MG) and employs surface aeration. Digester No. 2 has a volume of 0.33 MG and has diffused air. Digester No. 2 is currently used, with Digester No. 1 as standby to be available during maintenance or repairs to Digester No. 2. Both digesters have decant lines which are intended to draw off decant from the digesters to increase the solids content of the WAS to approximately 2 percent TS before sending the biosolids to dewatering. The aerobic digestion process was designed with a recommended SRT of 20 days at AADF to minimize odors.

Results of the aerobic digester capacity are shown Table 11. Based on the evaluation operating assumptions (decant to 2 percent TS, 20-day SRT) the capacity of Digester No. 2 is 1.5 mgd AADF, slightly below the expected buildout AADF of 1.59 mgd. However, operating choices, such as running Digester No. 2 at a slightly lower SRT of approximately 19 days, would allow the existing process equipment to meet capacity through the anticipated buildout condition with no expected negative impacts on process performance. Additionally, Digester No. 1 can be brought into operation to increase the process capacity while maintaining a 20-day SRT. Therefore, the existing aerobic digestion equipment was deemed sufficient through buildout capacity by anticipating minor changes to the current operational approach.

Table 11 Summary of Aerobic Digester Capacity

Digester SRT, days	Digester Volume, MG	AADF Capacity, mgd
20	0.33 (Digester No. 2)	1.5
10	0.33 (Digester No. 2)	3.0
20	0.60 (Digester No. 1 and 2)	2.9

There are two existing dewatering centrifuges with one duty and one standby unit. The dewatering centrifuge capacity evaluation was based on the required dewatering schedule to process solids at MMADL and the available digester storage capacity to support dewatering operations. The centrifuges have a capacity to run at 150 gpm but usually do not operate beyond 110 gpm. The dewatering schedule at current and buildout MMADL is presented in Table 12. At current MMADL the centrifuge needs to run approximately 14 hours per week to process solids associated with the MMADL condition. At the expected buildout of MMADL, the dewatering schedule increases to 20 hours per week, which is still feasible for WWRP operations staff.

Table 12 Summary of Dewatering Centrifuge Operations

Parameter	Current MMADL	Buildout MMADL
Influent Flow, mgd	1.3	1.8
Digested Solids, lb/day	3,100	4,200
Digested Solids, %	2	2
Centrifuges in Service	1	1
Centrifuge Capacity, gpm	150	150
Dewatering Schedule, hours per week	14	20

The available storage capacity of the aerobic digesters was also assessed as part of the dewatering capacity evaluation and is summarized in Table 13. The target hydraulic residence time (HRT) is a minimum of three days to provide storage over a three-day weekend if needed. The digester HRT was calculated for one or two digesters in service and with or without decanting to 2 percent TS. Under typical operations, the WWRP decants to a target of 2 percent TS, so the digester HRT will be in the range of nine days with only Digester No. 2 in service. As a worst-case scenario, Digester No. 2 can still provide a three-day HRT, assuming no decanting, although this condition would only be experienced when decanting is stopped due to maintenance or repairs to the associated equipment. Based on the centrifuge throughput capacity, associated dewatering schedule, and the available digester storage capacity, the dewatering process has sufficient capacity to process solids through the buildout MMADL condition.

Table 13 Summary of Aerobic Digester Storage Capacity

Parameter	No Decanting		Decant to 2%	
	1	2	1	2
Digesters in Service				
WAS Load at Buildout MMADL, lb/day	5,900		5,900	
WAS TSS, mg/L	6,900		20,000	
WAS Flow, gpd	103,000		35,000	
HRT, days	3.2	5.9	9.3	17

3.4 Hydraulic Capacity Evaluation

3.4.1 Hydraulic Flows

The hydraulic capacity evaluation was completed by developing a model and using the model to identify possible hydraulic bottlenecks at the WWRP at current and buildout flow conditions. The WWRP influent flows are summarized in Table 5 for the current and buildout conditions. The internal recycle flows used in hydraulic modeling are summarized in Table 14, based on the existing equipment capacities.

Table 14 Hydraulic Modeling Internal Recycle Flows

Process	Flow	Notes
RAS	2.16 mgd	Maximum flow (total pumping capacity is 2.88 mgd)
IMLR	2.66 mgd	Maximum flow per aeration basin

3.4.2 Hydraulic Modeling Analysis

A Hydraulix® model was developed and evaluated for the current and buildout flow conditions to identify possible hydraulic bottlenecks at the WWRP. Two model scenarios were developed, one assuming the existing Trojan UV system and one assuming the new Enaqua UV system. To develop the hydraulic model, Carollo reviewed historical record drawings for the initial WWRP construction and subsequent expansion or upgrade projects as relevant for each process area, as well as operations and maintenance (O&M) manuals and design reports as available on the WWRP equipment O&M SharePoint database.

For completing the hydraulic modeling analysis for the new Enaqua UV system, Carollo reviewed the City of Sedona WWRP UV Replacement Project 100 percent contract documents (January 2025) and revised Design Report (April 2025) prepared by Sunrise Engineering. For the development of this design, Sunrise Engineering established the design criteria for the project by applying Arizona Code R18-9-E301 guidelines as advised by ADEQ during the permit review of the project. As such, Carollo reviewed this UV system hydraulics with the 2.8 mgd buildout peak design flow rate criteria outlined in Table 15. For all other systems throughout the WWRP, the peaking factors previously described in Section 3.1.1 were applied.

Table 15 New Enaqua UV System Design Criteria

Condition	Value	Information Source
Population	10,000-12,000	Sunrise Engineering Design Report – April 2025
Average Flow Rate	1.39 mgd	Sunrise Engineering Design Report April 2025
Peak Design Flow Rate	2.4 mgd	Sunrise Engineering Design Report April 2025
Current Peak Design Flow Rate	2.0 mgd	Sunrise Engineering 100% Design Specification January 2025
Buildout Peak Design Flow Rate	2.8 mgd	Sunrise Engineering Design Report April 2025 (Enaqua)

The model was developed using the following assumptions:

- The current and anticipated buildout flows and peaking factors are summarized in Table 5.
- The RAS flow was set to 2.16 mgd for all conditions reflecting the RAS pump capacity with three pumps in service and one standby.
- The IMLR flow was set to 2.66 mgd for each aeration basin in service, reflecting the total IMLR pump capacity.
- Aeration basins and SCs were modeled for several scenarios, including a condition where both aeration basins and SCs have one unit out of service (three aeration basins and two SCs in service).
- The filters were modeled with one unit in service at MMADF and PDF conditions and two in service at the PHF condition.
- The existing Trojan UV system was modeled with one of two parallel channels in operation at MMADF, PDF, and PHF conditions.
- The new Enaqua UV system was modeled with one of two parallel trains in service at the MMADF, PDF, and PHF of 2.8 mgd with two additional rows of UV lamps installed in each train for buildout conditions.

3.4.3 Hydraulic Recommendations

A possible hydraulic bottleneck was identified in the existing UV system at the filter effluent piping where full plant flow passes through a segment of 18-inch diameter pipe. This 18-inch diameter pipe results in high head loss at plant flow over about 2.5 mgd and results in the filter effluent weir freeboard decreasing well below the typical target value of 0.5 feet at PHF. To remedy this bottleneck, this pipe segment could be upsized to 30 inches to match the rest of the UV-influent header. However, since this pipeline is planned to be abandoned with the construction of the new UV system, no upgrade is recommended at this time.

For the new Enaqua UV system, the hydraulic model was developed based on the 100 percent drawings. The drawings show a new 30-inch pipe to convey filter effluent to an 18-inch header to the new UV system and a new 18-inch header to convey from the UV trains and reducing to tie into an existing 12-inch pipe that connects to the effluent pump station. The pipe diameters as designed showed the filter effluent weirs surcharging in the hydraulic model at buildout PHF, and upsizing is recommended to increase the small section of 12-inch pipe to 18-inch pipe for peak flows at buildout conditions to avoid the surcharge and to increase the complete 18-inch and 12-inch span between the UV system and effluent pump station to maintain a hydraulic free fall. Note that the capability will remain to utilize the emergency basin should the City desire to reduce peaking through the UV system in an extreme case for a limited amount of time.

3.5 Equipment Capacity Evaluation

A graphical summary of the existing major liquids process equipment capacities is presented in Figures Figure 20 and Figure 21. Several process equipment capacities are based on PHFs including preliminary treatment, tertiary filters, UV disinfection, and effluent pumping.

- **Preliminary Treatment**

The preliminary treatment facilities consist of mechanical bar screens and a vortex grit chamber. There are two mechanical screens in parallel with one duty and one standby unit. Each unit can service a peak flow of 6.91 mgd, exceeding the current and buildout PHF conditions with full redundancy. The vortex grit chamber is designed for a peak flow of 7.65 mgd, again providing more than enough capacity to service PHF at the current and buildout conditions.

- **Tertiary Treatment**

The tertiary treatment facilities consist of three cloth media disk filters and UV disinfection. The disk filters were designed with a firm capacity of 4.6 mgd, operating all three units at PHF with no redundancy, meeting the current and buildout PHF conditions.

The existing Trojan UV system consists of two parallel channels, each designed for treatment of up to 3.6 mgd, meeting the current and buildout PHF conditions with full redundancy. The new Enaqua UV system was modeled with one of two parallel trains in service at the MMADF, PDF, and PHF of 2.8 mgd, with two additional rows of UV lamps installed in each train for buildout conditions.

- **Effluent Pumping**

The Sedona WWRP has three sets of effluent pumps to deliver treated effluent to irrigation uses or injection wells. The combined firm capacity of these three pump systems is 5.4 mgd, as summarized below, providing enough capacity to handle the buildout PHF capacity.

- » The area 1 to 3 pump station has four duty and one standby pump with a firm capacity of 2.9 mgd.
- » The area 4 pump station has two duty and one standby pump with a firm capacity of 2.0 mgd.
- » The injection well pump station has one duty and one standby pump with a firm capacity of 0.5 mgd.

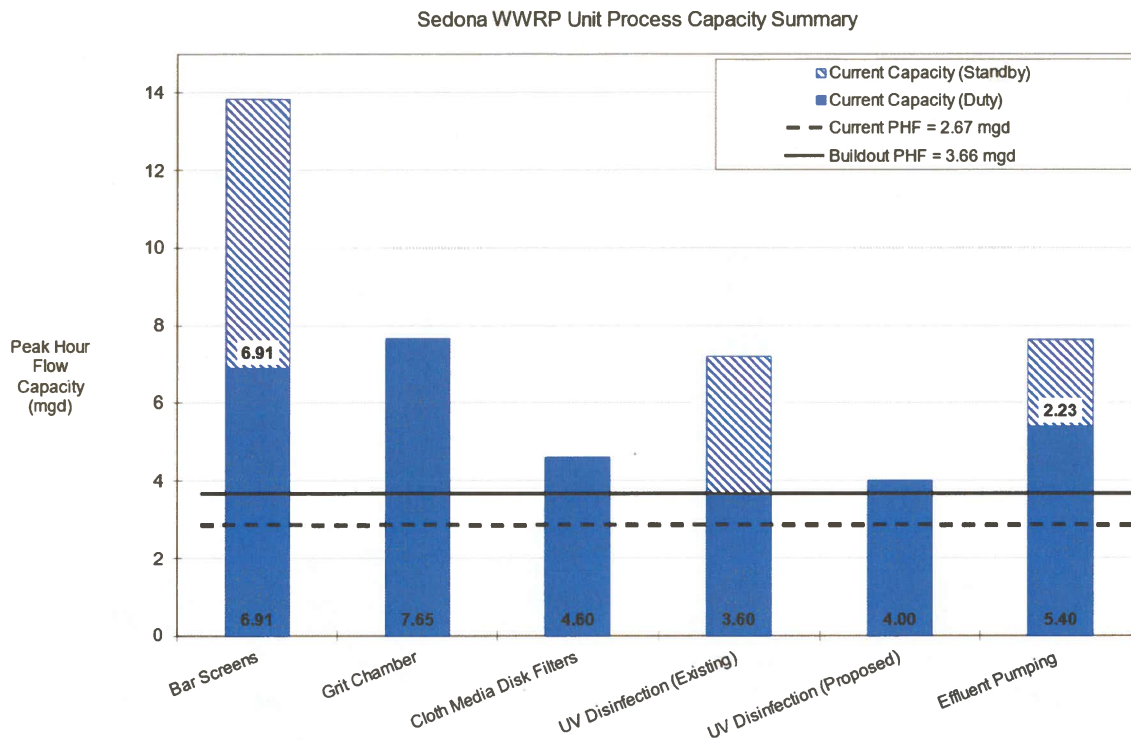


Figure 20 Summary of Equipment Capacities for PHF

■ Secondary Treatment

The SCs, RAS pumps, and effluent management uses were evaluated against MMADF capacity. For the purpose of this evaluation, a RAS ratio of 70 percent influent flow was assumed, noting plants can typically operate RAS in the range of 50 percent to 150 percent depending on many factors. The existing RAS pumps have a capacity of up to 2.16 mgd with one pump out of service, which equates to 160 percent of the current MMADF or 120 percent of the buildout MMADF. Based on the secondary treatment process capacity evaluation included in Section 3.2.3, the clarifier capacity is 2.18 mgd at MMADL with one unit out of service, based on maintaining a minimum CSF of 1.2 at peak day flow to properly support biological process efficiency and clarifier settleability.

■ Effluent Management

Finally, effluent management capacity was evaluated based on flow data provided for the injection wells, irrigation areas, and wetlands. Based on this data, the existing effluent management areas have an AADF capacity of 1.6 mgd, which could be increased by 0.6 mgd with the addition of two more injection wells.

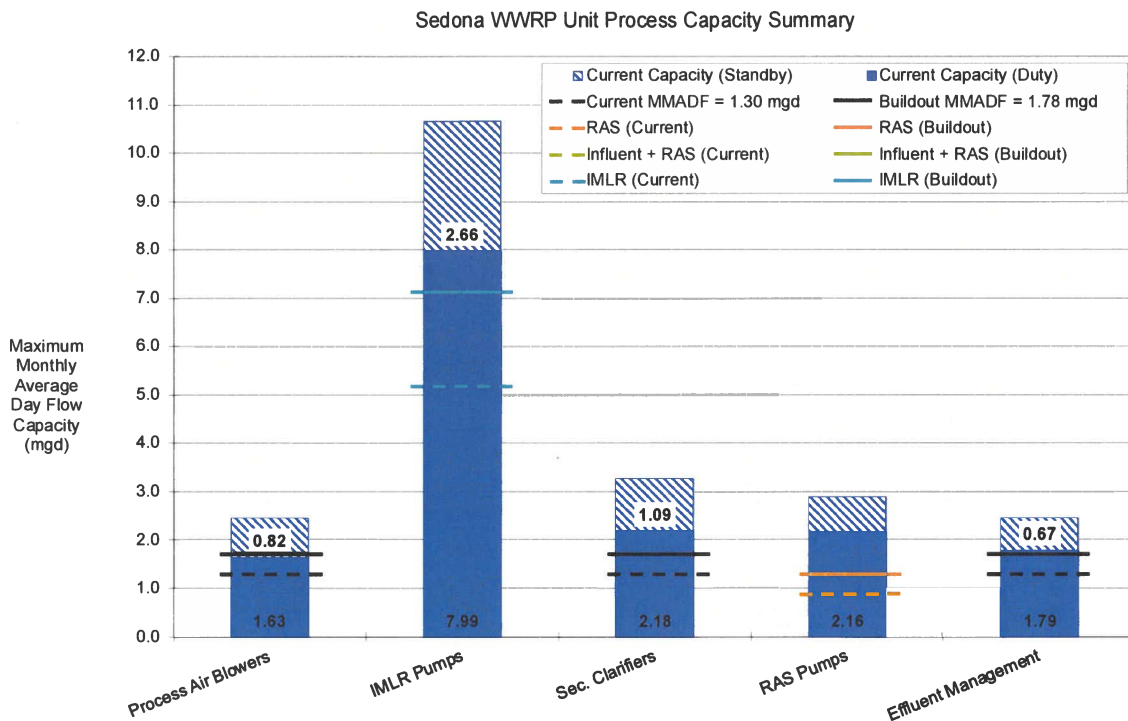


Figure 21 Summary of Equipment Capacity for MMADF

3.6 Summary of Capacity Evaluation Results

The results of the capacity evaluation indicated that existing process equipment and facilities will provide sufficient hydraulic and treatment capacity through the anticipated buildout condition. The results of the new UV system evaluation indicated that equipment and facilities will provide sufficient hydraulic and treatment capacity through the anticipated buildout condition with the addition of piping modifications as part of the new UV system construction at a minimum, as it is expanded to buildout capacity. Additionally, new injection wells could be installed to increase the capacity and operational flexibility of the effluent management system.

CHAPTER 4 PFAS TREATMENT ALTERNATIVES

As previously discussed, PFAS treatment requirements are currently anticipated as part of future AWQS rulemaking by ADEQ. This section provides the conceptual design criteria for PFAS treatment of wastewater effluent.

4.1 Flow Analysis and PFAS Design Capacity

Two PFAS treatment capacity scenarios were considered for this evaluation. The Alternative 1 treatment scenario was sized for 1.30 mgd (903 gpm), corresponding to the current MMADF. The Alternative 2 - Buildout treatment scenario was sized for 1.78 mgd (1,250 gpm), corresponding to the anticipated

buildout MMADF per the 2017 Master Plan. Section 3.1.1 includes additional discussion of WWRP flows used for the basis of this evaluation.

For either flow scenario, it was assumed that the full WWRP effluent flow would be treated for PFAS, rather than a partial treatment approach. This was assumed due to the possibility that future ADEQ PFAS regulations for wastewater effluent will be directly tied to AWQS requiring treatment for PFAS for any of the current or planned methods of effluent management strategies. The PFAS-treated water would then go to existing effluent management uses via the existing effluent pump station or to a combination of existing effluent management and park irrigation or AWP (discussed in Section 3.1.1.1.).

4.2 PFAS Data

The WWRP has performed multiple rounds of PFAS sampling for the biosolids, WWRP influent, WWRP effluent, and POC wells starting in September 2023. PFAS data available at the time of this evaluation for the six compounds with MCLs are shown in Table 16 for water samples and Table 17 for biosolids samples. At this time of this evaluation, biosolids PFAS limits have not been set.

Table 16 Sedona WWRP PFAS Data – Influent, Effluent, and POC Wells

Compound	Units	Proposed MCL	Influent			Effluent			Potable Well			POC-1			POC-3			POC-4	
			9/6/2023	8/14/2024	9/11/2024	9/6/2023	8/14/2024	9/11/2024	9/20/2023	8/7/2024	9/7/2024	9/6/2023	8/14/2024	9/11/2024	9/6/2023	8/7/2024	9/4/2024	9/13/2023	8/21/2024
PFOA	ng/L	<4.0 ng/L	4.3	<7.5	<4.6	13	10.4	11.7	3.4	1.5	1.6	3.3	<0.99	<0.98	46	29.9	31.1	5.5	1.9
PFOS	ng/L	<4.0 ng/L	<4.0	<7.5	<4.6	<4.0	<4.5	1.2	<4.0	<1.0	<0.95	<4.0	<0.99	<0.98	7.9	6	7.5	<4.0	<0.99
PFBS	ng/L	<1.0 *HI	4.9	<7.5	<4.6	6.5	6	3.7	<1.5	1.4	1.4	<1.5	1.3	1.3	36	25	23.2	3.8	4.3
PFHxS	ng/L	<10.0 & <1.0 *HI	<1.5	<7.5	<4.6	<1.5	<4.5	1.2	<1.5	<1.0	<0.95	<1.5	<0.99	<0.98	---	3.9	4.2	<1.5	<0.99
PFNA	ng/L	<10.0 & <1.0 *HI	<3.0	<7.5	<4.6	<3.0	<4.5	<0.96	<3.0	<1.0	<0.95	<3.0	<0.99	<0.98	<3.0	1.5	1.7	<3.0	<0.99
Gen X	ng/L	<10.0 & <1.0 *HI	<5.0	<30.0	<18.5	<5.0	<18.0	<3.8	<5.0	<4.0	<3.8	<5.0	<4.0	<3.9	<5.0	<3.9	<3.9	<5.0	<4.0

Notes:

ng/L - nanogram per liter

(1) HI = (GenX (ng/L)/10) + (PFBS (ng/L)/2000) + (PFNA (ng/L)/10) + (PFHxS *(ng/L)/9).

(2) < indicates the result is below the detection limit.

(3) EPA 537 sampling method used for 2023 PFAS concentrations and EPA 1633 sampling method was used for 2024 PFAS concentrations.

Table 17 Sedona WWRP PFAS Data - Biosolids

Compound	Units	Biosolids 4/24/24 Reporting Limit	Biosolids 6/26/24 Reporting Limit	Biosolids 7/17/24 Reporting Limit
PFOA	µg/kg	1.2	6.6	10.4
PFOS	µg/kg	1.6	11.5	16.4
PFBS	µg/kg	<0.37	<3.2	<2.0
PFHxS	µg/kg	<0.37	<3.2	<2.0
PFNA	µg/kg	<0.37	<3.2	<2.0
Gen X	µg/kg	<1.5	<12.9	<8.1

Notes:

µg/kg - micrograms per kilogram

(1) HI = (GenX (ng/L)/10) + (PFBS (ng/L)/2000) + (PFNA (ng/L)/10) + (PFHxS *(ng/L)/9).

(2) < indicates the result is below the detection limit.

4.3 Water Quality Treatment Limits

The water quality treatment limits established for the PFAS treatment alternatives include maintaining Class A+ quality per the existing APP permit and meeting PFAS drinking water MCLs. There are currently no PFAS regulations in Arizona addressing wastewater effluent, but for this evaluation, it was assumed that future wastewater regulations will be tied to the established drinking water MCLs for PFAS. The water quality goals for the PFAS treatment alternatives are summarized in Table 18.

Table 18 Water Quality Limits for PFAS Treatment

Parameter	Units	Value
A + Reclaimed Water		
Turbidity ⁽¹⁾	NTU	≤2
Turbidity ⁽¹⁾	NTU	≤5
Fecal Coliform ⁽²⁾	organisms/100 mL	0
Fecal Coliform ⁽²⁾	organisms/100 mL	≤23
TN ⁽³⁾⁽⁴⁾	mg/L	≤10
PFAS MCLs		
PFOA	ppt	<4
PFOS	ppt	<4
PFHxS	ppt	<10
PFNA	ppt	<10
GenX	ppt	<10
HI ⁽⁵⁾	--	1

Notes:

(1) Turbidity must be ≤ 2 NTU for a 24-hour average sample and ≤ 5 NTU for a 120-second average sample.

(2) Fecal coliform in four of the last seven samples must be non-detect and ≤ 23 for a single sample maximum.

(3) The TN limit is based on a five-sample rolling geometric mean. There is an alert level of 8 mg/L.

(4) TN = NO₃-N + NO₂-N + TKN.

(5) PFBS is included within the HI but does not have an individual MCL.

4.4 Alternatives Development

The following sections present PFAS treatment alternatives chosen for this evaluation. This includes descriptions of TOC pretreatment requirements and PFAS treatment options and corresponding schematic process flow diagrams (PFD) and design criteria.

4.4.1 Summary of Alternatives

Granular activated carbon (GAC) and ion exchange (IX) were chosen as PFAS treatment technologies for the alternatives included in this study. GAC and IX are industry-standard treatment methods due to lower cost and high efficacy compared to other available technologies. Additionally, the EPA identifies GAC, IX, reverse osmosis (RO), and nanofiltration (NF) as the "best available technologies" for meeting PFAS MCLs. RO and NF were not considered for PFAS treatment alternatives due to challenges with brine management, high cost, on-site chemical handling/operator safety concerns, and high operating pressures.

4.4.2 PFAS Treatment Considerations

An important factor to consider for PFAS treatment in this evaluation is TOC pretreatment, as the feed water is treated effluent from the WWRP and high in organics. Wastewater effluent has a higher TOC concentration than groundwater, causing frequent breakthroughs for GAC and IX, and leading to more media changeouts compared with surface water or groundwater treatment applications. TOC pretreatment improves feedwater quality to the GAC or IX vessels and reduces media changeout frequency. Granular media can also become fouled by inorganic precipitation (e.g., iron) or sediment retention, negatively impacting performance because adsorption sites are mainly located within media pores. Therefore, pretreatment to remove solids is also essential for all water treatment systems to protect downstream adsorptive media.

There are multiple differences between IX and GAC related to O&M. Differences that would impact the ultimate selection of a treatment process are as follows:

- **Backwashing** - GAC requires backwashing, whereas IX does not. For this evaluation, it was assumed that the existing equalization (EQ) basin in the WWRP would be used for GAC backwash. The WWRP's EQ basin can be used to return flow to the WWRP headworks or directly to the filters. Backwashing is not utilized in the IX process, which decreases costs and minimizes operation and maintenance.
- **Changeout Frequency** - GAC breakthrough is generally faster compared to IX, causing more frequent media changeouts, although breakthrough for both media types is water quality dependent. At a minimum, bench scale testing of media options should be performed as part of the next phase of design.
- **Pretreatment** - For pretreatment, IX requires robust pretreatment for sediment removal to protect media, whereas GAC requires a less robust pretreatment due to the availability of backwashing to remove sediment from the surface of the bed. To minimize the impact of the mass transfer zone, a robust pretreatment should be used for both GAC and IX.

- **Spent media management** - GAC media can be regenerated and reused before final disposal for non-potable applications. Even when GAC media is reused, some virgin media will need to be purchased during changeouts due to attrition. IX resin media cannot be regenerated and either requires incineration or disposal in a landfill. At this time, there are no requirements to dispose of PFAS-containing wastes in a hazardous waste landfill, although that could change in the future. Carollo typically recommends incineration and disposal of spent media at a hazardous waste landfill to avoid complications should a typical disposal location become a Superfund site in the future.

Table 19 provides a comparison of considerations for GAC and IX for PFAS treatment.

Table 19 Comparison of GAC and IX for PFAS Treatment Considerations

Criteria	GAC	IX
Maturity of Technology	Well-established, multiple media and equipment sources.	Well-established, multiple media and equipment sources.
Required Equipment	GAC contactors and backwash system.	IX vessels. No backwash system required.
Changeout Frequency	Potentially more frequent media changeouts compared to IX.	Potentially less frequent media changeouts compared to GAC.
Footprint	Larger footprint due to more equipment needed.	Smaller footprint due to less equipment needed.
TOC Concentration	Can negatively impact media lifespan.	Can negatively impact media lifespan
Flexibility/Adaptability	Greater flexibility for expanding treatment processes for contaminants beyond PFAS.	Less flexibility for expanding treatment processes for contaminants beyond PFAS.
Forward Flush	Virgin media could leach arsenic depending on GAC source.	Initial forward flushing of media is required. Corrosive water can be found in the forward flush rinse.
Backwash Waste	System requires occasional backwashing and maintenance.	Regular operation requires no backwashing.
Pretreatment Requirements	Backwashing removes the pressure differential created by sediment buildup, allowing for less robust pretreatment.	Robust pretreatment for sediment removal is required to protect integrity of media.
Spent Media Management	Spent media can potentially be regenerated at appropriately permitted facilities.	Spent media generally requires incineration or landfill disposal at appropriately permitted facilities.
Power Requirements	Requires relatively lower head to get through media bed.	Requires relatively higher head to get through media bed.

Color Code:

	Critical Consideration
	Moderately Desirable
	Most Desirable

4.4.3 Proposed Treatment Train

For each alternative, treatment with GAC or IX pressure vessels was considered. The schematic PFD for the PFAS treatment train is shown in Figure 22. From the new UV system, wastewater effluent will enter the wet well and, from there, will be pumped to the TOC pretreatment system. TOC pretreatment would consist of ozonation and biologically active carbon filtration (BAF) where sediment removal occurs. The backwash system from BAF would go to the existing EQ basin. After TOC pretreatment, would water flow

to GAC or IX contactors in a lead-lag configuration? For GAC, there would be a backwash recycle system to the existing EQ basin with single-element bag filters, and backwash recycle pumps, whereas, for IX, backwash would not be required. Bag filters were assumed at this stage of evaluation but may be excluded during detailed design. Fines from backwashing the GAC pressure vessels would settle in the EQ basin and be removed periodically. After the GAC or IX contactors, treated effluent would proceed to irrigation, wetlands, and/or injection wells via the existing effluent pump station, park irrigation, or AWP, as described in Section 5.1. Further information about each process and the design criteria is included in Section 4.5.

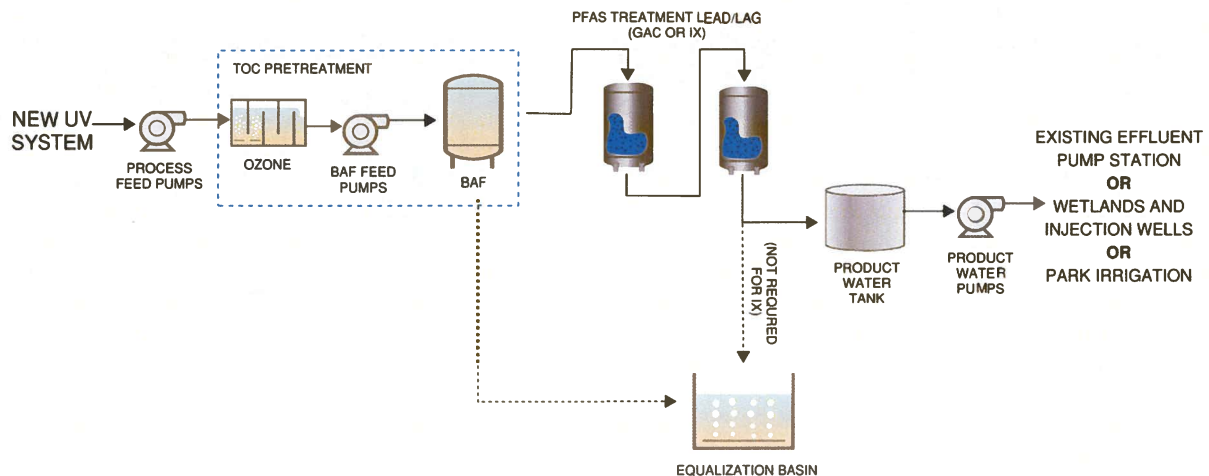


Figure 22 PFAS Treatment PFD

4.5 PFAS Treatment Facility Design Criteria

Alternative 1 and Alternative 2-Buildout design flows for each PFAS treatment alternative are shown in Table 20. Detailed PFAS treatment facility design criteria are included in Appendix B.

Table 20 PFAS Treatment Facility Flow Capacity Assumptions

PFAS Treatment	Alternative	MMADF Capacity (mgd)
GAC	Alternative 1	1.3
	Alternative 2-Buildout	1.8
IX	Alternative 1	1.3
	Alternative 2-Buildout	1.8

4.5.1 Ozone

Ozonation is a chemical oxidation process that breaks down organic molecules to increase their bioavailability. Following ozonation, the BAF process removes organic matter, including trace constituents and their ozonation byproducts, using microbial communities that grow on the surface of the media. By reducing TOC and trace organics and making the BAF filtrate more biostable, ozone/BAF improves downstream GAC/IX performance and minimizes fouling on GAC/IX media. The ozone process will include

liquid oxygen storage, ozone gas generation, ozone injection into an ozone contactor, and destruction of off-gassed ozone.

Future evaluation should be performed in preliminary design to determine the ozone transfer efficiency and number of ozone injection points required. On-site oxygen generation is assumed for this evaluation due to Sedona's more remote location making it more difficult for liquid oxygen deliveries. Ozone design criteria are shown in Table 21.

Table 21 Ozone Design Criteria

Parameter	Units	Alternative 1	Alternative 2 - Buildout
Design Capacity	mgd	1.3	1.8
Number of Generators	-	2	2
Measured TOC ⁽¹⁾	mg/L	7.7	7.7
Ozone/TOC Ratio Assumed	--	1	1

Notes:

gpm - gallons per minute

(1) Data from one sample taken on November 13, 2024.

4.5.2 Biologically Active Filtration

The BAF process can be provided in the form of a gravity filter or pressurized filter. To develop footprint and cost opinions for this study, pressure vessels were assumed. The type of filter that will be employed should be further evaluated and refined during the next phase of design. Empty bed contact time (EBCT) is a key design criterion for BAF. Higher EBCTs result in better biological degradation and TOC removal but increase capital and operational costs. While the optimal time must be selected through piloting, EBCTs of between 10 and 30 minutes are typical for wastewater effluents. For this study, BAF systems were sized to maintain a minimum EBCT of 10 minutes at the design flow rates with one filter in backwash.

As the filtration run time increases over a period of days, solids and biomass will build up on the filter media until a backwash is needed. Backwashing should be minimized for BAF to promote biological growth within the media bed. The entire backwash process typically lasts between 30 and 60 minutes, but the source water quality will play an important role in determining the actual frequency and duration. BAF design criteria are shown in Table 22.

Table 22 BAF Design Criteria

Parameter	Units	Alternative 1	Alternative 2 - Buildout
Design Capacity	gpm	903	1,250
No. of Filters	-	4	5
Filter Diameter	feet	12	12
Flow per filter ⁽¹⁾	gpm/sq ft	2.0	2.2
EBCT per vessel	minutes	19	17

Notes:

gpm/sq ft - gallons per minute per square foot

(1) Flow if all filters are in operation.

4.5.3 GAC

GAC is a porous material with a high volumetric surface area that adsorbs organic impurities and is effective in PFAS removal. GAC hydraulic operation is similar to that of BAF, but unlike BAF, the predominant contaminant-removal mechanism is adsorption. GAC media must be replaced with new or regenerated media after the adsorption sites are exhausted and targeted contaminants begin breaking through. The spent carbon can be sent for off-site disposal or regeneration. The GAC media life for PFAS removal can vary widely depending on the carbon type, target contaminant concentrations, and background water quality. Bench-scale studies such as rapid small-scale column testing (RSSCT) or piloting are recommended when selecting GAC media and estimating changeout frequency.

GAC systems require a backwash supply and provisions for EQ and settling. Carbon fines and any sediment must be periodically backwashed to reduce pressure loss through the contactors. Backwashing is also required to reverse carbon bed compression, which can occur during normal operations. For this study, pressure vessel contactors were assumed.

Conceptual design criteria for GAC systems are provided in Table 23. GAC treatment systems will be configured in lead-lag pairings to extend the life of the media. Design criteria are shown for both Alternatives.

Table 23 GAC Design Criteria

Parameter	Units	Alternative 1	Alternative 2 -Buildout
Configuration	-	Lead/Lag	Lead/Lag
Design Capacity	gpm	903	1,250
Number of Vessels per Train	-	2	3
Flow per Train	gpm	451	417
Contactor Diameter	feet	10	10
Volume of Media per Contactor	pounds	20,000	20,000
EBCT per Train	minutes	19.7	21.3
Liquid Loading Rate	gpm/sq ft	5.7	5.3
Backwash Flow Rate per Vessel	gpm	864	864
No. of Backwash per Week	No.	1	1
Backwash Hydraulic Loading Rate	gpm/sf	11	11
Bed Expansion during Backwashing	%	30-50	30-50
Backwash Tank Volume	gallons	60,000	60,000

4.5.4 IX

Synthetic IX resin systems use pressure vessels operated in downflow mode through an IX resin bed. IX treatment systems will be in a lead-lag configuration to extend the life of the media. Single-use IX resin provides PFAS removal from feed water through IX. The removal capacity diminishes over time with the volume of water treated, and the resin must be replaced and sent off-site for disposal. IX regeneration technology for resins used in PFAS treatment is not yet commercially available. Unlike GAC contactors, IX vessels should not be backwashed. Backwashing of IX resin beds should be avoided, as the spherical resin

beads are structurally stable, and disturbance of the mass transfer zone in the media bed can reduce the media life and increase changeout frequency and cost.

The IX media life for PFAS removal can vary widely depending on the media type, target contaminant concentrations, and background water quality. Bench-scale studies such as RSSCT or piloting are recommended when selecting IX media and estimating changeout frequency and potential for fouling. Conceptual design criteria for IX systems are provided in Table 24.

Table 24 IX Design Criteria

Parameter	Units	Alternative 1	Alternative 2 - Buildout
Configuration	-	Lead/Lag	Lead/Lag
Design Capacity	gpm	903	1,250
Number of Trains	-	1	2
Flow per Train	gpm	903	1,250
Contact Diameter	feet	10	10
EBCT per Train ⁽¹⁾	minutes	5.6	6.0
Liquid Loading Rate	gpm/sq ft	11.5	8.0

Notes:

(1) The calculated EBCT is for maximum flow conditions.

4.5.5 Media Changeout Projections

IX changeout estimates are based on data from other projects and based on projections provided by an IX vendor for a PFAS-selective IX resin. Water quality data was provided to the IX vendor to support their projections. For each alternative, the changeout-controlling compound is PFOA. GAC media life was estimated using GAC changeout data from other projects. Further testing, such as RSSCT's, will be required for more accurate comparison between IX and GAC media life in future phases of design.

Media changeout projections for GAC and IX are summarized in Table 25. The minimum media changeout frequencies for GAC and IX are based on a potable reuse pilot study (Tajdini et al., 2023) including Ozone/BAF. The maximum projected bed volumes for GAC are based on past projects, while the maximum bed volumes for IX are based on vendor projections. These projections aim to compare the relative cost performance between IX and GAC media. However, actual media changeout frequencies depend on site-specific water quality and can vary significantly. It is recommended to test multiple media at bench- and/or pilot-scale, including GAC, IX, and possibly surface-modified clay adsorbents, in the next phase of the design to evaluate site-specific media effectiveness and optimize the treatment train.

Table 25 GAC and IX Bed Volumes and Changeout Projections

Media	Changeout Controlling PFAS Compound	Bed Volume to Breakthrough		Source
GAC	PFOA	Min	2,000	Tajdini et al., 2023
		Max	25,000	Recent Carollo Projects
IX	PFOA	Min	40,500	Tajdini et al., 2023
		Max	106,000	IX Vendor Projections, 2024 ⁽¹⁾

Notes:

- (1) Bed volumes to breakthrough provided by an IX vendor for a PFAS selective IX resin. The estimate is for a 2 ppt breakthrough of the changeout-controlling PFAS compound for the lag vessel.
- (2) Bed volume projections used water quality data from Sedona's effluent.

4.6 Conceptual Cost Opinions

Conceptual-level capital and O&M cost opinions and life-cycle costs were developed for each of the alternatives. This section will discuss these cost opinions. Cost estimates were prepared in accordance with the guidelines of the Association for the Advancement of Cost Engineering (AACE) International for a Class 5 estimate. Table 26 summarizes the AACE International cost estimate classification system, the level of project definition (percent of design), uses, cost estimating methodologies, and expected accuracy of Class 1 through 5 estimates.

Table 26 Conceptual Cost Assumptions

Estimate Class	Maturity Level of Project Definition Deliverables - (Level of Engineering Design)	End Use	Typical Cost Estimating Methodology Used	Expected Accuracy Range (Low/High)
Class 5	0% to 2%	Conceptual screening	Capacity factored, parametric models, judgment or analogy	L: -20% to -50% H: +30% to +100%
Class 4	1% to 15%	Study or feasibility	Equipment factored or parametric models	L: -15% to -30% H: +20% to +50%
Class 3	10% to 40%	Budget authorization or control	Semi-detailed unit costs with assembly-level line items	L: -10% to -20% H: +10% to +30%
Class 2	30% to 75%	Control or bid/tender	Detailed unit cost with forced detailed take-off	L: -5% to -10% H: +5% to +20%
Class 1	65% to 100%	Check estimate or bid/tender	Detailed unit cost with detailed take-off	L: -3% to -10% H: +3% to +15%

4.6.1 Cost Opinions and Assumptions

Several factors and contingencies are applied to the direct costs or total construction costs to arrive at the total project cost.

Direct cost factors applied to equipment costs to arrive at the total direct costs include:

1. **Site Work:** 10 percent applied to equipment costs without installation.
2. **Installation:** 30 percent applied to equipment costs.

3. **Yard Piping and Valves:** 10 percent applied to equipment costs with installation.
4. **Electrical:** 15 percent applied to equipment costs with installation.
5. **Instrumentation and Controls:** 15 percent applied to equipment costs with installation.
6. **Conceptual Level Design Contingency:** 30 percent contingency applied to total direct costs.

Indirect cost factors applied to the direct costs to arrive at the total costs include:

1. **Contractor Overhead and Profit and General Risk:** 27 percent applied to total direct costs with contingency.
2. **Bonds and Insurance:** 3 percent applied to total direct costs with contingency.
3. **Transaction Privilege Tax:** 65 percent of the combined tax rate (10.4 percent), calculated to be 9.9 percent and applied to total direct costs with contingency.
4. **Engineering and Administration:** 25 percent applied to total construction cost.

4.6.2 Cost Escalation

The conceptual cost estimates were developed based on construction data from previous projects, vendor budgetary quote information, and RS Means published unit costs. The Handy-Whitman average Water Infrastructure Cost Index for the Plateau region (includes Arizona) was used to escalate costs as applicable. Whitman, Requardt & Associates, LLP (WRA), based in Baltimore, Maryland, has published the Handy-Whitman Index® of Public Utility Construction Costs™ continuously since 1924. The Handy-Whitman Index is based on basic materials, equipment, wage rates, and other prices specific to common types of construction in each market. The water utility construction index is based on the following types of construction and equipment:

- Water source facilities.
- Pumping plants.
- Treatment plants.
- Transmission facilities (reservoirs, tanks, mains).
- Distribution facilities (pipelines, meters, hydrants).
- Miscellaneous items (specifically flocculation, clarification, and filter gallery equipment).

WRA develops a separate index for six geographical regions with similar characteristics throughout the United States to account for differing regional cost trends. Arizona is part of the Plateau region, which includes Arizona, New Mexico, Nevada, Utah, Colorado, Idaho, Wyoming, and Montana.

Escalation of unit costs, construction data from bid tab summaries, and vendor costing information was done using the calculated average Handy-Whitman Index so that all costs represent 2024 dollars (Handy-Whitman Average Water Utility Construction Index for the Plateau Region Average Value of 1311, July 2024).

4.6.3 Cost Summary

Table 27 provides PFAS treatment capital costs for Alternative 1 and Alternative 2 - Buildout. PFAS treatment costs include applicable reclaimed water conveyance costs and PFAS treatment costs. Differences in total capital costs are due to conveyance requirements and differences in PFAS treatment. Table 28 provides O&M costs for both alternatives and media options.

Table 29 summarizes the conceptual cost opinions for each alternative. Present-worth analyses were prepared for the final treatment alternatives. These values represent the sum of the capital cost opinions and the present worth of annual O&M cost projections, assuming a discount rate of 4 percent and a term of 20 years. The detailed cost opinions for each alternative are included in Appendix C.

Table 27 PFAS Treatment Alternatives Capital Cost Summary (\$ million)

	GAC		IX	
	Alternative 1	Alternative 2 - Buildout	Alternative 1	Alternative 2 - Buildout
Total Direct Cost	\$13.4	\$15.1	\$12.4	\$14.2
Total Direct Cost with Contingency	\$17.4	\$19.7	\$16.2	\$18.4
Total Indirect Cost	\$6.9	\$7.8	\$6.4	\$7.3
Total Construction Cost	\$24.3	\$27.5	\$22.6	\$25.8
Total Capital Cost	\$30.4	\$34.4	\$28.3	\$32.2

Notes:

- (1) Construction costs include total direct cost with contingency and total indirect cost.
- (2) Total capital costs include total construction costs and Engineering and Administration of 0.25.

Table 28 PFAS Treatment Alternatives Annual O&M Cost Summary (\$ million)

	GAC		IX	
	Alternative 1	Alternative 2 - Buildout	Alternative 1	Alternative 2 - Buildout
Media and Filter Replacements (High Estimate)	\$2.97	\$4.11	\$0.70	\$0.97
Media and Filter Replacements (Average Estimate)	\$0.44	\$0.61	\$0.39	\$0.54
Media and Filter Replacements (Low Estimate)	\$0.24	\$0.33	\$0.27	\$0.37
Power	\$0.02	\$0.03	\$0.03	\$0.04
Staffing	\$0.15	\$0.15	\$0.15	\$0.15
General Equipment Maintenance	\$0.09	\$0.10	\$0.08	\$0.09
Total Annual O&M Costs (\$ million) High	\$3.22	\$4.38	\$0.96	\$1.24
Total Annual O&M Costs (\$ million) Average	\$0.69	\$0.88	\$0.64	\$0.81
Total Annual O&M Costs (\$ million) Low	\$0.49	\$0.60	\$0.52	\$0.64

Notes:

- (1) Media changeouts estimate depends on a range breakthrough bed volumes.

Table 29 PFAS Treatment Alternatives Conceptual Cost Opinions Summary (\$ million)

	GAC		IX	
	Alternative 1	Alternative 2 - Buildout	Alternative 1	Alternative 2 - Buildout
Total Capital Cost	\$30.4	\$34.4	\$28.3	\$32.2
Annual O&M Cost	\$0.7	\$0.9	\$0.6	\$0.8
Present Worth of Annual O&M	\$9.4	\$12.0	\$8.7	\$11.0
Total Present Worth	\$39.8	\$46.4	\$37.0	\$43.2
Equivalent Annual Cost	\$2.9	\$3.4	\$2.7	\$3.2
\$/1,000 gallons	\$13.0	\$17.8	\$13.0	\$17.8

Notes:

- (1) Cost opinions shown in July 2024 dollars (Handy-Whitman Average Water Infrastructure Index equals 1311).
- (2) Net present value calculations based on a 20-year term at a 4 percent discount rate.
- (3) Total present worth represents the sum of capital costs and present worth of annual O&M.
- (4) Assumes discount rate of 4 percent per year and term of 20 years and assumes design flow of each facility within each alternative operating all day, all year.

4.7 Considerations for PFAS Treatment Implementation

As discussed in Section 4.4.2., non-cost related considerations between GAC and IX need to be considered as part of PFAS treatment implementation. IX offers some operational simplicity due to not requiring backwashing and has a smaller footprint and generally lower costs compared to GAC. However, IX resin cannot be regenerated. GAC requires backwashing and more frequent media changeouts, but GAC media can be regenerated. A range of estimated breakthrough bed volumes was estimated for GAC and IX media changeouts using similar projects and studies. Further testing, such as RSSCTs, will be required for a more accurate comparison between IX and GAC media life in future phases of design.

CHAPTER 5 **ALTERNATIVE EFFLUENT MANAGEMENT**

The Facility Plan evaluation includes alternative effluent management approaches that the City may pursue in combination with, or to replace, existing effluent management uses (injection wells, irrigation areas, wetlands). The two alternative effluent management approaches discussed in this chapter include delivery of PFAS-treated reclaimed water to the City for use in park irrigation applications or AWP for treatment to potable water standards and delivery to an Arizona Water Company property for introduction to the potable distribution system.

5.1 Reclaimed Water Delivery Evaluation

There is an existing ductile iron pipeline built in the early 1990s that was intended to deliver reclaimed water into West Sedona, but the pipeline was never brought into operation. The pipeline starts with a 12-inch section that goes from the north side of the WWRP along Route 89A to the intersection with Foothills Drive South. A 10-inch section then extends east on Foothills Drive South from the intersection of Route 89A and terminates near the El Camino Lift Station at 700 El Camino Road. For the purpose of this evaluation, it was assumed that this existing pipeline would need to be relined due to the age of the pipeline and likely impacts to gaskets and joint integrity. The condition of the pipeline should be further evaluated as a part of the next phase of design. Following rehabilitation, the pipeline would be brought into service for either the reclaimed or potable delivery alternatives that are covered in this chapter. New pipelines would be constructed to deliver water from the existing pipeline to final delivery locations as applicable to each alternative considered.

The City provided monthly totals for irrigation water used by City-owned parks and properties between the months of January 2020 to March 2024. Parks outside of West Sedona were not considered due to proximity to the WWRP and the expense associated with installing pipelines in complex construction conditions to meet very small demands. Of the parks in West Sedona, those with the highest irrigation use and closest proximity to the existing pipeline were considered in order to maximize the benefit in terms of project cost per water supply benefit. Based on these assumptions three alternatives were considered for this evaluation:

1. Alternative 1 includes a booster pump station at the WWRP to deliver reclaimed water to the high school on Route 89A for irrigation of fields on the high school property.
2. Alternative 2 includes a booster pump station at the WWRP to deliver reclaimed water to the high school, plus a second booster pump station located at the high school to deliver reclaimed water to Sunset Park for irrigation use.
3. Alternative 3 includes a booster pump station at the WWRP to deliver reclaimed water to the high school, plus a second booster pump station located at the high school to deliver reclaimed water to Sunset Park, Jamison Park, Posse Grounds Park, and West Sedona School for irrigation use.

Figures showing these alternatives are included in Appendix D.

5.1.1 Reclaimed Delivery Design Criteria

Design flows were based on data for monthly total irrigation water provided for City parks between the months of January 2020 to March 2024 and shown in Table 30. The maximum monthly irrigation demand was used as the basis for the delivery volume, with a 6-hour irrigation window assumed to develop the design flow for each alternative in gallons per minute. Design criteria for pumping requirements for each alternative are summarized in Table 31.

Table 30 Summary of Park Irrigation Demand and Design Flows

Park	Maximum Monthly Irrigation Demand, gpd	Design Flow, gpm ⁽¹⁾
High School	52,886	147
Sunset Park	33,784	94
Jamison Park	356	1.0
Posse Grounds Park	24,582	68
West Sedona School	25,935	72
Total - Alternative 1 ⁽²⁾	52,886	147
Total - Alternative 2 ⁽³⁾	86,670	241
Total - Alternative 3 ⁽⁴⁾	137,543	382

Notes:

- (1) The design flows were calculated assuming a 6-hour irrigation window.
- (2) Alternative 1 includes irrigation at the high school.
- (3) Alternative 2 includes irrigation at the high school and Sunset Park.
- (4) Alternative 3 includes irrigation at the high school, Sunset Park, Jamison Park, Posse Grounds Park, and West Sedona School.

Table 31 Reclaimed Delivery Alternatives Design Criteria

Parameter	Alternative 1	Alternative 2	Alternative 3
Design flow, gpm ⁽¹⁾	147	241	382
Pipeline ID, inches	6.7	6.7	6.7
V, fps	1.1	1.8	2.8
Primary Booster Pump Station (WWRP to High School)			
Pipeline length, feet	22,600	22,600	22,600
Static head, feet	400	400	400
TDH, feet	422	454	527
Combined efficiency	75%	75%	75%
Pump hp required	21	37	68
Secondary Booster Pump Station (High School to Final Irrigation Location)			
Pipeline length, feet	100	14,100	22,900
Static head, feet	---	20	120
TDH, feet	69	123	318
Combined efficiency	75%	75%	75%

Parameter	Alternative 1	Alternative 2	Alternative 3
Pump hp required	5	10	27
Total pump hp required	26	47	95
Total kWh per year	169,000	306,000	618,000

Notes:

ID - identification; fps - feet per second; TDH - total dynamic head

(1) Design flows assume an irrigation window of 6 hours per day.

For the purpose of this evaluation, it was assumed that Primus liner would be used for rehabilitation of the reclaimed pipeline. This liner is made of Kevlar between two layers of polyethylene and is a structural liner that does not depend on the structural integrity of the host pipe. Primus liner is suitable for potable use. The need for lining and alternatives to pipeline rehabilitation may be investigated in a later stage of project development, including cured-in-place pipe lining, pipe bursting, slip lining, or constructing a new parallel pipeline. A detailed condition assessment of the host pipe is recommended prior to any additional evaluation of the approach.

5.1.2 Conceptual Cost Opinions

AACE Class 5 cost estimates as described in Section 4.6 were developed for each reclaimed delivery alternative. Present-worth analyses were prepared for the reclaimed delivery alternatives. These values represent the sum of the capital cost opinions and the present worth of annual O&M cost projections, assuming a discount rate of 4 percent and a term of 20 years. A summary of costs is presented in Table 32, with additional detailed cost information available in Appendix C.

Table 32 Reclaimed Delivery Alternatives Cost Summary

Cost	Alternative 1	Alternative 2	Alternative 3
Delivery Locations	High School	High School, Sunset Park	High School, Sunset Park, Jamison Park, Posse Ground Park, West Sedona School
Total Capital Cost ⁽¹⁾ (\$ million)	\$14.5	\$21.0	\$28.6
Annual O&M Cost (\$ million)	\$0.2	\$0.3	\$0.4
Present Worth of Annual O&M ⁽²⁾ (\$ million)	\$2.3	\$3.5	\$5.6
Total Present Worth ⁽³⁾ (\$ million)	\$16.8	\$24.6	\$34.2
Equivalent Annual Cost (\$ million)	\$1.2	\$1.8	\$2.5
\$/1,000 gallons	\$16.1	\$14.8	\$13.0

Notes:

(1) Cost opinions shown in July 2024 dollars (Handy-Whitman Average Water Infrastructure Index equals 1311).

(2) Net present value calculations based on a 20-year term at a 4 percent discount rate.

(3) Total present worth represents the sum of capital costs and present worth of annual O&M.

5.2 AWP Treatment Alternatives

5.2.1 Flow Analysis and AWP Design Capacity

The AWP treatment design capacity was determined using the current MMADF as discussed in Section 3.1.1. For Alternative 1, a partial flow scenario was considered where a portion of PFAS treated water would be used for injection wells and wetlands uses and the remaining PFAS treated effluent not used in those effluent management locations would proceed to the AWP treatment facility. For Alternative 2, the current MMADF of 1.30 mgd was assumed and the Alternative 3 - Buildout scenario reflects the projected future MMADF flow of 1.78 mgd. The AWP treatment facility design capacity is summarized in Table 33.

Table 33 AWP Treatment Facility Flow Capacity Assumptions

Alternative	MMADF Capacity (mgd)
Alternative 1	0.9
Alternative 2	1.3
Alternative 3 - Buildout	1.8

5.2.2 AWP Water Quality Goals

AWP water quality goals were developed from federal drinking water standards and more stringent targets based on industry standards for AWP projects in Arizona. These water quality goals were considered in the design criteria development discussed in Section 5.2.3 and are shown in Table 34.

Table 34 AWP Water Quality Goals

Parameter	Units	Value	Notes
Primary drinking water standards		-	USEPA MCLs with exceptions for more stringent targets noted below.
Finished water stability			
pH		7.5 - 8.0	
Langelier Saturation Index	Unitless	>0	
CCPP	mg/L as CaCO ₃	4-10	
Buffer capacity	moles/L/unit pH	-	Similar or greater than current water distributed.
Aggressive Index	Unitless	>12	Noncorrosive.
CSMR	Unitless	-	Similar or less than current water distributed.
TDS	mg/L	<700	Consistent with supplies to local service area.
TOC	mg/L	<2	
Ammonia	mg/L as N	<0.25	To ensure full nitrification.
Total trihalomethanes	µg/L	64	80 percent of MCL.
Haloacetic acids	µg/L	48	80 percent of MCL.

Parameter	Units	Value	Notes
Nitrate	mg/L as N	8	80 percent of MCL.
Nitrite	mg/L as N	0.8	80 percent of MCL.

CaCO₃ - calcium carbonate; CCPP - calcium carbonate precipitation potential; TDS - total dissolved solids; CSMR - chloride-to-sulfate mass ratio; moles/L/unit pH - moles per liter of acid or base to change pH of water by one unit; µg/L - micrograms per liter

5.2.3 Development of AWP Treatment Alternatives

For this evaluation, reverse osmosis-based advanced treatment (RBAT) was the main alternative considered for the development of site layouts and cost estimates. For the purpose of this evaluation RBAT was chosen as the treatment train for full evaluation as a conservative approach in terms of footprint and cost, and to follow industry trends by other municipalities in the state. Carbon-based advanced treatment (CBAT) treatment with and without a 50 percent side stream RO system was also considered for cost savings potential, but a full conceptual design evaluation was not conducted for these two alternative treatment trains. Due to the current proposed ADEQ AWP rule, CBAT may be infeasible, especially for smaller and more rural facilities to implement due to more stringent monitoring requirements compared with RBAT. The following sections present the AWP treatment alternatives chosen for this evaluation.

5.2.3.1 Summary of Alternatives

For this evaluation, it was assumed that AWP treatment via RBAT would be implemented downstream of a PFAS treatment system described in Chapter 4. Even though the membrane filter systems proposed for AWP are capable of removing PFAS, the separate process approach acknowledges that PFAS treatment is likely to be installed prior to AWP treatment being implemented, and importantly, this approach would eliminate the issues associated with addressing PFAS in the RO concentrate, which significantly complicates brine management and disposal. The concepts for both RBAT and CBAT are discussed in the following sections.

RBAT Treatment Train

The RBAT treatment train includes the following elements:

- Treatment processes: ultrafiltration (UF), RO (full stream), stabilization, ultraviolet advanced oxidation process (UV AOP), and chlorination.
- Concentration management facility (CMF) is required with this treatment train, including lime softening (LS), UF/microfiltration (MF), RO, mechanical vapor recompression evaporator (MVRE), and a crystallizer or evaporation pond.

For this evaluation, calcite contactors were chosen for stabilization. Pretreatment, including ozonation and BAF, is often added upstream of UF, depending on water quality considerations. However, since ozone and BAF would already be included in the PFAS treatment train, they were not considered for the RBAT treatment train in this evaluation.

Due to Sedona's remote location, brine would need to be handled on-site, requiring a CMF as part of this treatment train. The two options for brine management are discharging brine directly to evaporation ponds or implementing a CMF treatment train to further concentrate RO brine and discharging brine to smaller evaporation ponds. The second option was chosen for this evaluation due to land availability and the large area required for unconcentrated brine volumes.

CBAT Treatment Train

The CBAT treatment train with 50 percent side stream RO includes the following elements:

- Treatment processes: UF, side stream RO treating 50 percent of flows, UV AOP, GAC, and chlorination.
- CMF and solids handling facilities would still be required with this treatment train, but facility sizes would be smaller than those required for an RBAT train.

The CBAT train without RO includes the following elements:

- Treatment processes: UF, UV AOP, GAC, and chlorination.
- A new blending source is required under this treatment train for management of TDS in final treated water.
- No CMF or solids handling facilities would be required with this treatment train.

The CBAT treatment train does not require brine management unless a side-stream RO system is included. CBAT treatment offers a cost-effective option compared to RBAT. However, ADEQ's proposed AWP rule includes regulatory implications that may deem this infeasible for smaller and more rural facilities like Sedona. For this evaluation, CBAT and 50 percent side stream RO treatment trains were considered at a high level for cost comparison, but a full conceptual design evaluation was not conducted.

Proposed Treatment Train

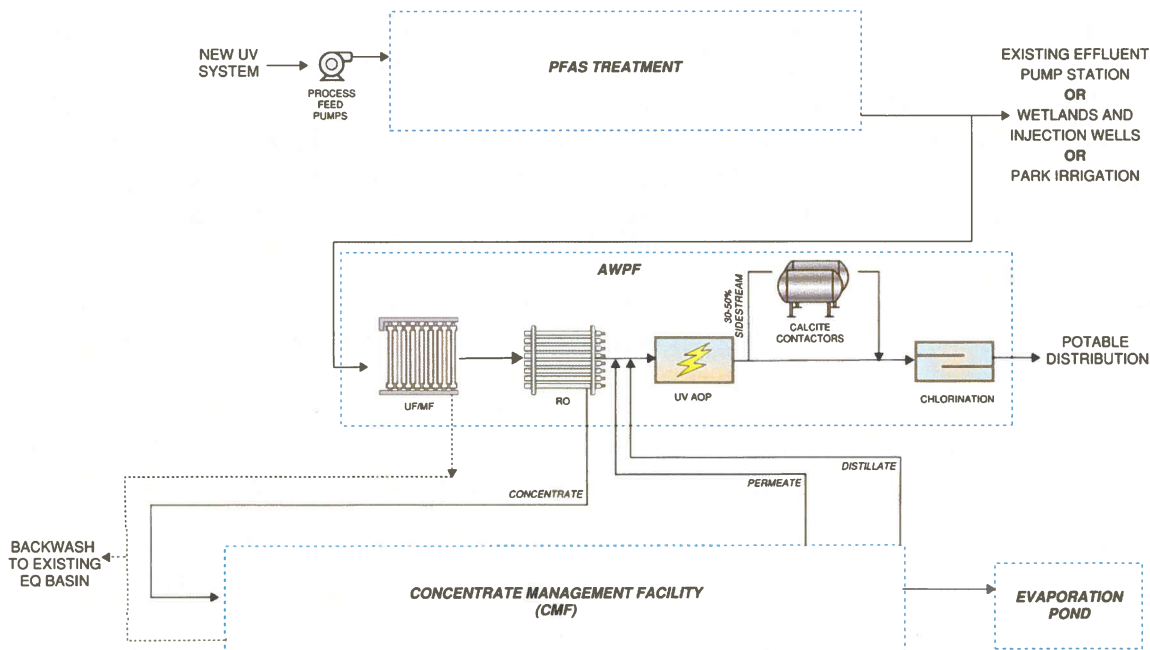


Figure 23 AWP Treatment PFD

5.2.3.2 AWP Facility Design Criteria

The following section includes process descriptions for the proposed RBAT treatment train. Design criteria for the AWP facility are included in Appendix E.

Primary Ultrafiltration

PFAS-treated water will feed the primary UF system, which is a low-pressure membrane filtration system used to remove pathogens and particulate matter from the feed water to enhance the performance of the downstream RO membrane. The RO process requires consistent feed water quality with turbidity levels of less than 0.1 NTU and silt density index values of less than 3. UF provides consistent, high-quality water within a compact footprint.

A UF feed tank is assumed ahead of the system and the UF feed pumps will pressurize flow from the UF feed tank through the UF system. It is assumed chloramines will be added ahead of the UF system to prevent membrane biofouling. A system recovery of 95 percent is assumed after the UF filters.

The primary UF system generates backwash flows. To optimize water use, the treated backwash water will be recycled upstream to the existing EQ basin at the WWRP, minimizing water loss and maintaining system efficiency.

The secondary UF system that will serve the CMF is discussed later in this section.

Primary Reverse Osmosis

RO is a well-established purification process that uses semi-permeable membranes and a driving force of inlet water pressure to remove dissolved salts. This physical separation process can reject constituents as small as 0.0001 micrometers (μm), meaning it can remove dissolved salts (i.e., TDS), hardness, dissolved organic carbon, synthetic organic chemicals, disinfection by-product precursors, and PFAS.

An RO system is made up of spiral-wound RO elements, which consist of several layers of RO membranes wound around a central permeate collection tube and enclosed in a cylindrical housing. The membranes separate the feed flow into treated water (i.e., permeate) and a waste stream (i.e., concentrate). As feed water flows along the length of the element, permeate passes through the membrane, leaving behind most dissolved constituents. This ultimately results in a progressively decreasing concentrate flow that carries the same mass of dissolved salts. The RO system recovery is the permeate production expressed as a percentage of the feed flow.

Permeate is composed of low-salinity, high-quality water. While some salts, neutrally charged chemicals, and gasses will pass through the RO membrane into the permeate, the concentrate stream will contain all remaining constituents that were trapped on the feed side. Since the ions being removed will be further concentrated as water passes through the system, scaling and foulants may form on the membrane surface, which can decrease the system's efficiency. Scaling can be prevented by adding sulfuric acid and a chemical scale inhibitor upstream of the RO process, which keeps scalants in solution.

RO trains are typically designed in stages with the number of stages depending on the supply water's quality and design recovery rate. For this study, a two-stage RO system with a recovery of 80 percent is assumed. In two-stage systems, concentrate from the first-stage pressure vessels is combined and fed to a smaller number of second-stage pressure vessels. To size the primary RO system, the City's target TDS concentration of 700 mg/L was used.

An RO transfer pump located in the RO feed tank supplies UF filtrate to the RO feed pump. Solids, such as fine sands or organic debris, will induce RO membrane fouling, which may cause mechanical damage to the RO membrane elements. Although the UF system will provide exceptionally high-quality water free of suspended solids, cartridge filters are still required to protect against membrane damage from suspended material introduced into the RO feed tank as well as other unexpected solids. For all advanced water purification facilities (AWPF), disposable cartridge filters are assumed as a final barrier of protection after UF and prior to the RO system.

The secondary RO system that will serve the CMF is discussed later in this section.

Ultraviolet Advanced Oxidation Process

The UV AOP process combines high doses of UV light with an oxidant, generating hydroxyl radicals that effectively degrade contaminants. When UV-treated RO permeate is used, certain compounds such as chloramines and the oxidant itself act as UV-absorbing scavengers, reducing ultraviolet light transmission (UVT). Additionally, TOC, nitrite, alkalinity, and bromide can behave as scavengers, consuming hydroxyl radicals and diminishing the treatment efficiency.

Both free chlorine and hydrogen peroxide can be employed as oxidants in the UV AOP process. In the AWPF, free chlorine is the assumed oxidant, as it simplifies operations by eliminating the need for a hydrogen peroxide quenching post-treatment system. The UV light effectively breaks down chloramines, which improves UVT and reduces scavenger demand within the reactor. This maintains optimal UV AOP system performance while keeping the free chlorine residual available (adjustable in the next treatment step to meet potable water quality standards) prior to distribution.

The primary purpose of UV AOP is to reduce emerging contaminants such as N-nitrosodimethylamine (NDMA) and 1,4-dioxane, which may not be fully removed by RO membranes. NDMA log reduction primarily defines the UV dose. Per ADEQ AWP rules, the UV AOP system must achieve at least 0.5-log reduction of 1,4-dioxane. Demonstration testing will refine design criteria, including oxidant dosing, UV dose, and assumed log reduction values.

Calcite Contactors

Post-RO treatment, the permeate undergoes stabilization to ensure non-corrosive and distribution-ready water. Stabilization is achieved by introducing minerals (i.e., calcium and alkalinity) through up-flow calcite contactors in a side-stream treatment configuration. For this study, 12-foot diameter vertical contactors are assumed. In this process, 50 percent of the RO permeate flows through the contactors, while the remaining bypassed flow is blended back to achieve the desired water chemistry before final chlorination. Depending on post-RO treated water quality, carbon dioxide addition upstream of the calcite contactors could be considered, followed by caustic dosing downstream of the contactors for stabilization.

Clear well/Chlorine Disinfection

A clear well will be required for purified WS. The clear well will also serve as a chlorine contact basin before purified water is pumped into the distribution system. Clear wells were sized assuming a 3-hour minimum storage time.

5.2.3.3 Concentrate Management Facility Design Criteria

The following section includes process descriptions for the proposed CMF. Design criteria for the CMF required for the RBAT treatment train are included in Appendix E.

Lime Softening

LS is the first step within the CMF. The primary RO brine contains high concentrations of hardness and silica which can cause scaling in the secondary RO process. LS is an established technology for reliably removing silica, TOC, metals, and both calcium and magnesium hardness to 50 mg/L as CaCO_3 or lower. During the design phase, bench scale studies or projections must be performed to quantify both silica and TOC removal via LS.

Secondary Ultrafiltration

LS will be followed by a secondary UF process to provide consistently low-turbidity water to the secondary RO process. As with the primary UF, a system recovery of 95 percent was assumed. Additionally, the secondary UF system also produces backwash flows. Residuals handling and backwash waste assumptions for the primary UF system are also assumed for the secondary UF system.

Secondary Reverse Osmosis

Secondary UF filtrate will feed the three-stage secondary RO system. An 85 percent recovery rate is assumed for the secondary RO. RO permeate will feed the primary UV AOP system, while the brine will be sent to MVRE for recovery.

RO recovery projections were not performed as part of this study. Within the design phase, RO projections should be performed to select the most appropriate RO system and optimize the RO recovery percentages. A pilot study is recommended for this refinement of the RO system design.

Ion Exchange

Secondary RO brine will go through IX pretreatment before MVRE to remove silica and hardness, preventing scaling and fouling within the equipment. Strong base IX resins are assumed due to their effectiveness in silica removal through both exchange and adsorption mechanisms at optimal pH levels. Further evaluation is recommended to refine the IX system design.

Mechanical Vapor Recompression Evaporator

A MVRE brine concentrator functions by compressing steam (i.e., evaporated brine), raising both the pressure and temperature of the steam. Compressed steam is then returned to the brine concentrator's evaporator vessel, which contains tubes. Brine solution is sprayed onto the tubes containing the hot compressed steam, generating more steam. Finally, condensed, distilled water is collected, the remaining steam is compressed again, and the remaining brine is re-circulated and mixed with fresh RO brine within the evaporator vessel, while a constant volume of concentrated brine is wasted from the process. Distillate is assumed to feed the UV AOP process, while the concentrated brine is sent to evaporation ponds for disposal. For this evaluation, a MVRE system recovery of 90 percent is assumed.

Evaporation Ponds

Concentrated brine from MVRE would be sent to evaporation ponds for final disposal. Evaporation ponds are most feasible for low-volume flows. While evaporation ponds can be easy to construct and require limited maintenance, and contain no mechanical equipment beyond a conveyance pump, construction costs can increase depending on the terrain. Impervious double-liners with a leachate collection system are required to avoid seepage and underlying aquifer contamination.

Evaporation ponds have environmental concerns, including the evaporation loss of a water source, potential groundwater contamination, and the potential health impacts associated with wildlife being attracted to the concentrate. Therefore, the brine volume should be minimized as much as possible before disposal. For all alternatives, land adjacent to the AWP is assumed for the evaporation ponds.

Evaporation pond surge capacity, storage capacity for precipitated salts, and freeboard for precipitation and wave action must be evaluated during the design phase. Additionally, the feasibility of using supplementary technologies that may reduce the required evaporation pond footprint, such as ecoVAP and Wind-Aided Intensified evaporation, may be evaluated during the design phase.

5.2.4 Development of AWP Delivery Alternatives

Each AWP treatment alternative would require delivery of the potable product water to West Sedona for use by Arizona Water Company, who is the main potable water purveyor in West Sedona.

5.2.4.1 AWP Delivery Design Criteria

Potable delivery alternatives were developed under the same assumptions as the reclaimed delivery alternatives described in Section 5.1 including rehabilitation of the existing effluent return line using Primus liner, which has National Sanitation Foundation-certified product lines. After collaboration with the City and Arizona Water Company it was decided that for the purpose of this study, potable water would be delivered to the Cultural Park site across from the high school on Route 89A. After delivery to this location, the water would be in the jurisdiction of Arizona Water Company for introduction to the potable distribution system. Design criteria for potable conveyance to the Cultural Park are shown in Table 35.

Table 35 Potable Delivery Alternatives Design Criteria

Parameter	Alternative 1	Alternative 2	Alternative 3 - Buildout
Design flow, mgd	0.90	1.30	1.78
Pipeline ID, inches	7.6	7.6	7.6
V, fps	2.75	3.98	5.45
WWRP to Cultural Park			
Pipeline length, feet	30,900	30,900	30,900
Static head, feet	400	400	400
TDH, feet	635	863	1228
Combined efficiency	75%	75%	75%
Pump hp required	134	262	511
Total kWh per year	653,000	1,306,000	1,960,000

5.2.5 Conceptual Cost Opinions and Assumptions

Conceptual-level capital and O&M cost opinions and life-cycle costs were developed for each of the alternatives. AWP cost estimates were prepared in accordance with the same AACE and Handy Whitman resources as described in Section 4.6 Conceptual Cost Opinions. Several factors and contingencies are applied to the direct costs or total construction costs to arrive at the total project cost. Direct and indirect cost factors are applied similarly to the description in Section 4.6, with the exception that site work was assumed at 5 percent applied to equipment costs without installation.

5.2.6 Cost Summary

Table 36 provides RBAT treatment train capital costs for Alternative 1, Alternative 2 and Alternative 3 - Buildout. Table 37 provides O&M costs for all alternatives. Table 38 presents the conceptual cost opinions for the RBAT treatment train for the RBAT alternatives. Present worth analyses were prepared for all treatment alternatives. These values represent the sum of the capital cost opinions and the present worth of annual O&M cost projections assuming a discount rate of 4 percent and a term of 20 years. The total unit cost is the total present worth divided by annual yield in acre-feet per year. Table 39 provides the potential cost savings that may be possible if the 50 percent RO (resulting in higher TDS product water) or CBAT treatment train were feasible alternatives. The detailed conceptual cost opinions for each alternative are included in Appendix C.

Table 36 AWP Treatment and Delivery Capital Cost Summary (\$ million)

Parameter	AWP Alternatives		
	Alternative 1	Alternative 2	Alternative 3 - Buildout
Total Direct Cost	62.1	67.3	75.8
Total Direct Cost with Contingency	80.8	87.5	98.6
Total Indirect Cost	24.2	26.2	29.5
Total Construction Cost	104.9	113.7	128.1
Total Capital Costs (\$ million)	132.2	143.1	161.1

Notes:

- (1) Construction cost includes total direct cost with 30 percent contingency and total indirect costs.
- (2) Total capital cost includes total construction costs and Engineering and Administration of 25 percent.

Table 37 AWP Treatment and Delivery Annual O&M Cost Summary (\$ million)

Parameter	AWP Alternatives		
	Alternative 1	Alternative 2	Alternative 3 - Buildout
Media and Filter Replacements	0.96	1.39	1.92
Power	0.45	0.65	0.89
Staffing	0.87	0.87	0.87
General Equipment Maintenance	0.40	0.44	0.49
Potable Delivery	0.15	0.24	0.32
Total Annual O&M Costs (\$ million)	2.83	3.58	4.50

Table 38 AWP Treatment and Delivery Conceptual Cost Opinions Summary (\$ million)

Parameter	AWP Alternatives		
	Alternative 1	Alternative 2	Alternative 3 - Buildout
Total Capital Cost ⁽¹⁾	132	143	161
Annual O&M Cost	2.8	3.6	4.5
Present Worth of Annual O&M ⁽²⁾	38.4	48.6	61.1
Total Present Worth ⁽³⁾	171	192	222
Equivalent Annual Cost	12.6	14.1	16.4
\$/1,000 gallons	38.2	29.7	25.2

Notes:

- (1) Cost opinions shown in July 2024 dollars (Handy-Whitman Average Water Infrastructure Index equals 1311).
(2) Net present value calculations assumes a 20-year term at 4 percent discount rate.
(3) Total present worth represents the sum of capital costs and present worth of annual O&M.

Table 39 Potential Cost Savings for 50 Percent RO or CBAT Treatment Trains (\$ million)

Treatment Train	Category	Alternative 1	Alternative 2	Alternative 3 - Buildout
50 Percent RO Train	Capital cost (\$ million)	95.7	101.4	113.7
	% Reduction compared to RBAT	28%	29%	29%
	Annual O&M costs (\$ million)	2.1	2.7	3.4
	% Reduction compared to RBAT	24%	24%	24%
	Total unit cost, \$/acre-foot	106,800	78,100	63,800
	% Reduction compared to RBAT	44%	47%	49%
CBAT Train	Capital cost (\$ million)	57.7	58.3	63.3
	% Reduction compared to RBAT	56%	59%	61%
	Annual O&M costs (\$ million)	0.5	0.6	0.6
	% Reduction compared to RBAT	81%	84%	86%
	Total unit cost, \$/acre-foot	64,400	44,900	35,500
	% Reduction compared to RBAT	66%	70%	72%

CHAPTER 6 ALTERNATIVES RANKING AND SCORING

A decision matrix approach was developed to evaluate the effluent alternatives investigated during this project and to compare against the alternatives that the City has previously considered. The effluent management alternatives evaluated as part of this effort include the following:

- Alternative 1: Keep existing irrigation (approximately 265 acres), wetlands, and injection wells and add two injection wells for needed future effluent management capacity.
- Alternative 2: Abandon 200 acres of irrigation and add two new injection wells in the short term; plan for two additional wells in the long term for needed capacity. This alternative was identified in the 2014 Effluent Management Optimization Plan.
- Alternative 3: Deliver reclaimed water to parks and keep current irrigation, wetlands, and injection wells.
- Alternative 4: Deliver reclaimed water to parks, keep wetlands and existing injection wells, and abandon irrigation except Area 3 irrigation by adding two new injection wells.
- Alternative 5: Implement AWP and potable delivery.

As part of the decision matrix approach, a set of evaluation criteria were established and defined as shown in Table 40. Weighting factors for these criteria were then developed using a pairwise comparison approach whereby each criterion is weighed against each of the others to determine which is considered more important or less important for the purpose of this evaluation. The weighting factors in Table 40 were developed using the averages of pairwise rankings created independently by five members of the joint Carollo/City project team.

Table 40 Definitions of Evaluation Criteria

Evaluation Criterion	Description	Weighting Factor
Capital Cost	Capital cost	7%
Operational Cost	Operational cost	6%
Life Cycle Cost	20-year life cycle cost	9%
Environmental	Beneficial use of effluent, water quality	9%
Economic	Land use, tourism, potential revenue	4%
Water Resource	Aquifer sustainability, water resiliency	11%
Effluent Management Dependability	Consistent effluent disposal, operational complexity	10%
Health and Safety	For the public and plant staff	10%
Public Perception	How the general public views and understands	7%
Infrastructure	Conveyance infrastructure requirements	6%
Land Required	Treatment facility footprint	1%
Implementation Time	Length of time to construct /implement each scenario	2%
Ease of Future Tie-ins	Ability of this scenario to tie into future upgrades	4%
O&M Considerations	Operational complexity, ease of maintenance/repairs for each scenario	9%
Personnel Requirements	Staffing needs, operator certifications, and training	5%

After establishing the evaluation criteria and corresponding weighting factors, each alternative was scored on each criterion using a scale of 1 to 10, with 1 reflecting the lowest benefit and/or highest cost and 10 reflecting the highest benefit and/or lowest cost. These scores were developed jointly by the Carollo/City project team. The resulting scores for each effluent management alternative are summarized in Table 41.

Based on this evaluation, effluent management Alternative 2 (abandon 200 acres of irrigation, add two new injection wells) was the preferred alternative based on aquifer sustainability, O&M considerations, and cost-effectiveness. Alternative 4 (deliver reclaimed water to parks and abandon irrigation by adding injection wells) was also a higher-ranked alternative mainly due to the environmental and water resources benefits, but it is not cost-effective. Alternative 1 (keep existing irrigation and add two new injection wells) and Alternative 3 (deliver reclaimed water to parks and keep current irrigation) were the least preferred, with a large impact on the ranking related to the effluent management consistency challenge associated with maintaining existing irrigation areas, which requires the WWRP to adjust the effluent management strategy based on seasonality and weather. Alternative 5 (implement AWP and potable delivery) was in the middle of the rankings due to the high benefits related to water resources and the environment, but the disadvantages of high costs and challenges associated with implementation, O&M, and staffing. Based on the findings of this evaluation, it is recommended that the WWRP pursue Alternative 2, which presents the most preferred alternative based on the criteria, weighting factors, and scoring method used for this evaluation.

Table 41 Effluent Management Alternatives Decision Matrix

Evaluation Criterion	Description	Weighting Factor	Alternative Ranking (1-10) 1 = Lowest Benefit and/or Highest Cost 10 = Highest Benefit and/or Lowest Cost				
			Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5
Capital Cost	Capital Cost	7%	10	9	8	7	1
Operational Cost	Operational Cost	6%	8	9	7	8	7
Life Cycle Cost	20 Year Life Cycle Cost	9%	9	8	8	7	5
Environmental	Beneficial use of effluent, water quality	9%	1	9	5	10	10
Economic	Land use, tourism, potential revenue	4%	1	6	4	8	7
Water Resource	Aquifer sustainability, water resiliency	11%	1	9	5	10	10
Effluent Management Dependability	Consistent effluent disposal	10%	3	6	3	6	5
Health and Safety	For the public and plant staff	10%	10	8	9	9	8
Public Perception	How the general public views and understands	7%	5	4	6	6	4
Infrastructure	Conveyance infrastructure requirements	6%	5	7	4	6	6
Land Required	Treatment facility footprint	1%	1	7	1	7	7
Implementation Time	Length of time to construct /implement each scenario	2%	8	4	6	3	2
Ease of Future Tie-ins	Ability of this scenario to tie into future upgrades	4%	2	5	3	6	7
O&M Considerations	Operational complexity, ease of maintenance/repairs for each scenario	9%	9	9	8	8	5
Personnel Requirements	Staffing needs, operator certifications, and training	5%	10	9	5	4	2
Score			5.70	7.66	5.92	7.51	6.14
Capital Cost (\$, million; injection well budgets provided by City)			\$11	\$22	\$28.6	\$39.6	\$161

Notes:

- (1) Alternative 1: Keep existing irrigation and add injection wells.
- (2) Alternative 2: Abandon 200 acres of irrigation and add new injection wells.
- (3) Alternative 3: Deliver reclaimed water to parks and keep current irrigation.
- (4) Alternative 4: Deliver reclaimed water to parks and abandon irrigation by adding injection wells.
- (5) Alternative 5: Implement AWP and potable delivery.

CHAPTER 7 SUMMARY AND CONSIDERATIONS

Table 42 provides a summary of treatment and delivery alternatives reviewed in this study including cost per 1,000 gallons, allowing comparison to other water supplies. The capacity evaluation indicated that other than pipe size changes in the new UV system, the WWRP currently has sufficient treatment capacity through the projected buildout of the facility.

The PFAS regulations that are currently expected to be promulgated through the ADEQ rule-making process are expected to impact the operation and required treatment at the WWRP. The PFAS regulatory process is expected to begin with 1 to 2 years of PFAS monitoring to develop baseline data for use in permitting. At this time, it is recommended that the City plan include the cost of PFAS treatment in the 10-year Capital Improvement Plan, with timing to be evaluated based on the approach to rulemaking as ADEQ begins the process later in 2025.

Delivery of the PFAS-treated effluent for irrigation use must be weighed against other disposal options using economic and non-economic factors. Reclaimed water delivery presents a large capital investment with a low return on investment. Consideration would need to be given to a rate structure for delivery of reclaimed water for residential or commercial use.

Implementing AWP is not a requirement but an option that may be beneficial based on the specific needs of the municipality and the water provider(s). Policy decisions regarding the approach to AWP must consider a variety of criteria, such as cost-effectiveness in terms of capital, operational and life cycle costs, environmental benefits, economic benefits, water resource portfolio impacts, and operational challenges and reliability.

Table 42 Summary of Treatment and Delivery Alternatives

Alternative		Alternative	Flow Capacity (MMADF), mgd	Total Capital Cost (\$ million)	Annual O&M Cost (\$ million)	Total Present Worth ⁽²⁾ (\$ million)	Equivalent Annual Cost (\$ million)	\$/1,000 gallons
PFAS Treatment	GAC	Alternative 1	1.30	\$30.4	\$0.7	\$39.8	\$2.9	\$6.9
		Alternative 2 - Buildout	1.78	\$34.4	\$0.9	\$46.4	\$3.4	\$8.1
	IX	Alternative 1	1.30	\$28.3	\$0.6	\$37.0	\$2.7	\$6.4
		Alternative 2 - Buildout	1.78	\$32.2	\$0.8	\$43.2	\$3.2	\$7.5
Reclaimed Delivery	To high school	---	147 gpm	\$14.5	\$0.2	\$16.8	\$1.2	\$16.1
	To high school and Sunset Park	---	241 gpm	\$21.0	\$0.3	\$24.6	\$1.8	\$14.8
	To high school and All Parks ⁽¹⁾	---	382 gpm	\$28.6	\$0.4	\$34.2	\$2.5	\$13.0
AWP Treatment and Potable Delivery	RBAT with delivery to Cultural Park	Alternative 1	0.9	\$132	\$2.83	\$171	\$12.6	\$38.2
		Alternative 2	1.3	\$143	\$3.58	\$192	\$14.1	\$29.7
		Alternative 3 - Buildout	1.78	\$161	\$4.50	\$222	\$16.4	\$25.2

Notes:

(1) All Parks include Sunset Park, Jamison Park, Posse Ground Park, and West Sedona School.

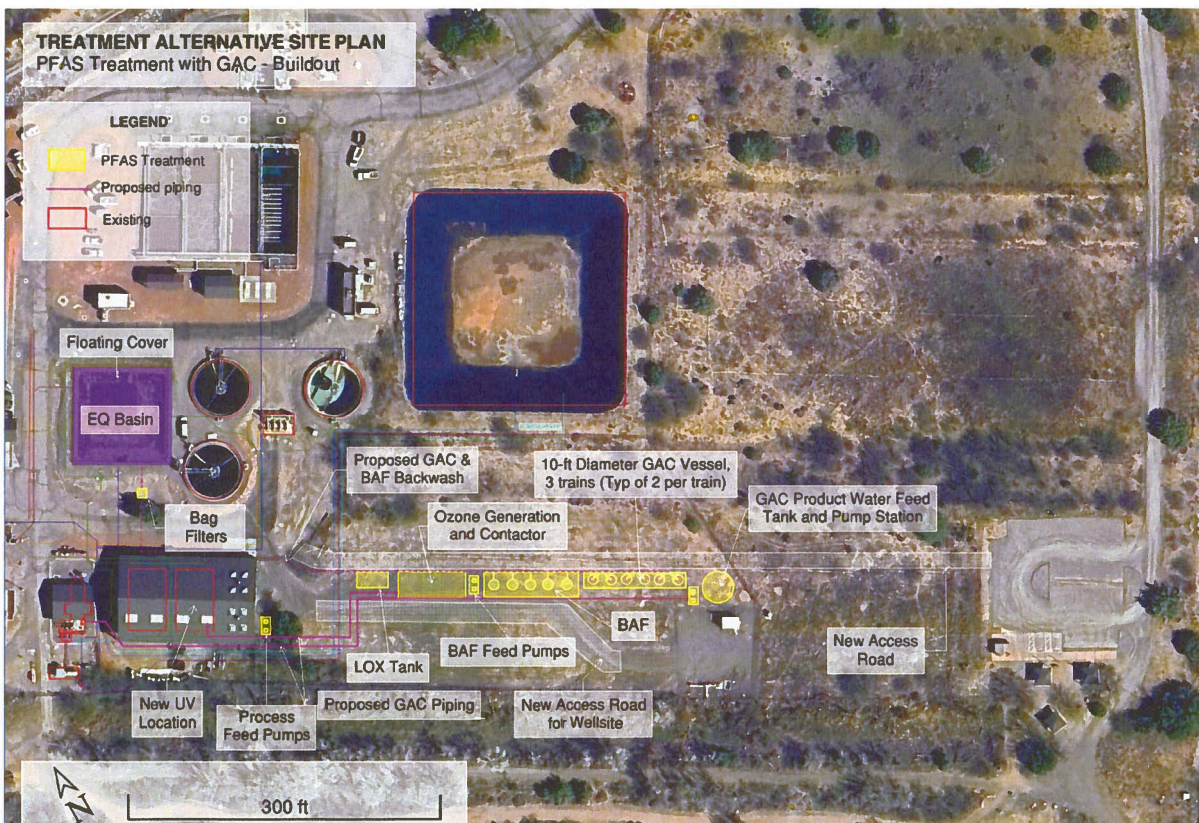
(2) Present worth was calculated based on a discount rate of 4 percent per year and a term of 20 years.

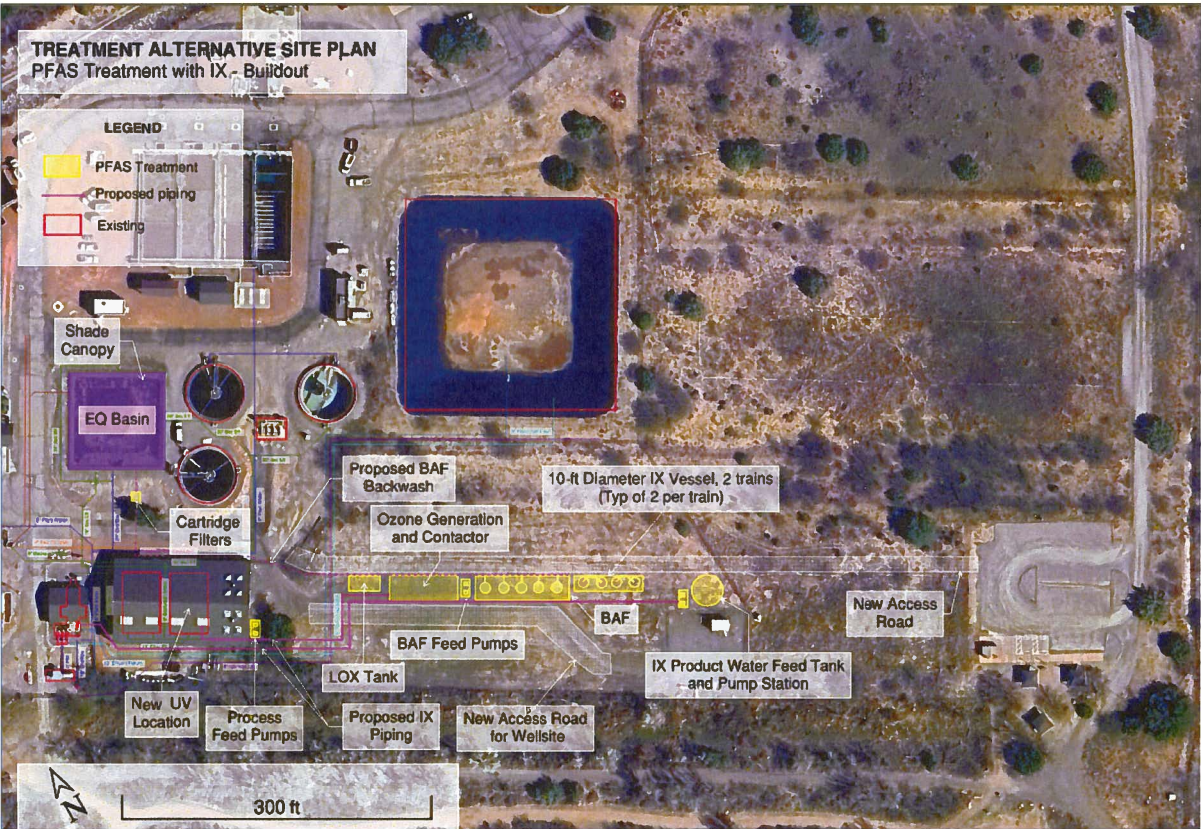
CHAPTER 8 REFERENCES

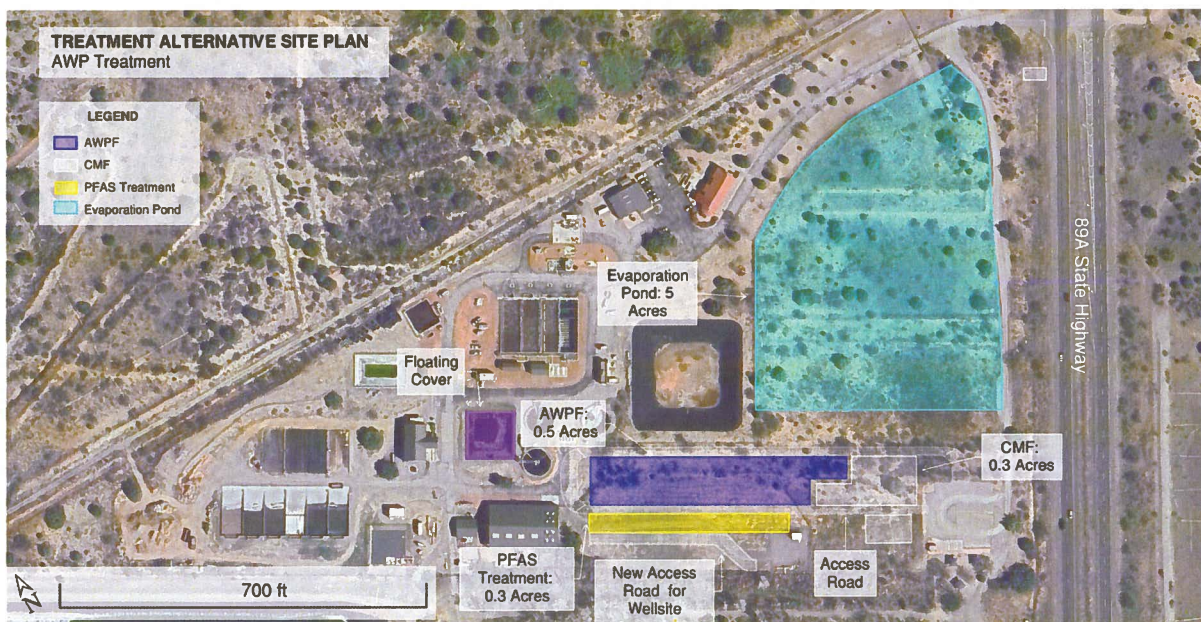
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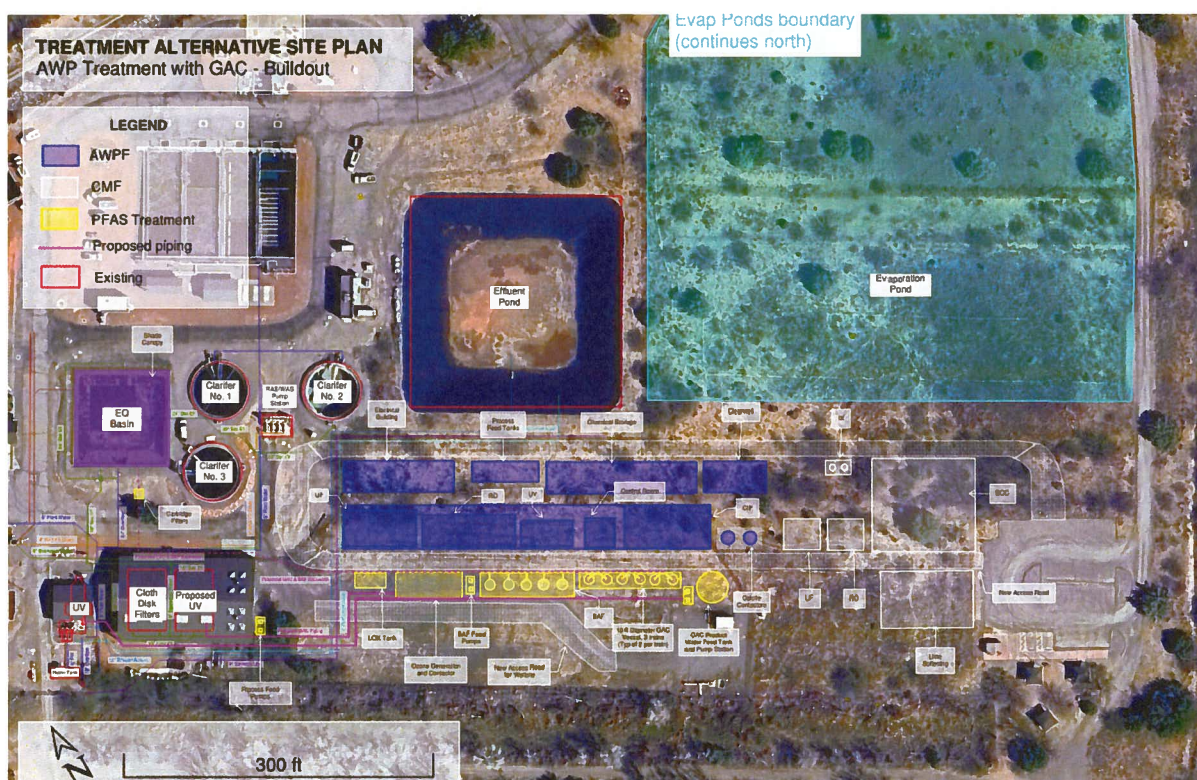
APPENDIX A

TREATMENT ALTERNATIVE SITE LAYOUTS











APPENDIX B

PFAS TREATMENT DESIGN CRITERIA

Table A.1 Ozone Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	0.9	1.3	1.8
Number of Ozone Generators				
In-Service	No.	1	0	1
Reliability	No.	1	0	1
Total	No.	2	0	2
Ozone Production				
Measured TOC (Max)	mg/L	7.7	7.7	7.7
Ozone/TOC Ratio	--	1	1	1
MTZ Efficiency	%	90	90	90
Maximum Ozone Dose	mg/L	8.6	8.6	8.6
Average Ozone Dose	mg/L	8.6	8.6	8.6
Ozone Weight Percent	%	12	12	12
Ozone Contact Time	min	0	10	10
Maximum Oxygen Production	ppd	64	93	128
Average Oxygen Production	ppd	64.2	92.8	128.4
Number of Liquid Oxygen Tanks	No.	1	1	1
Liquid Oxygen Tank Size	gal	1000	1000	1000
Abbreviations: gpm = gallons per minute; ft = feet (foot); mg-min/L = milligrams – minute per liter; ppd = pounds per day; kW = kilowatts;				
Notes:				
(1) Ozone CT required to remove log Cryptosporidium at degrees Celsius, according to the equation $\text{Cryptosporidium LRV} = \text{CT} \cdot 10^{\frac{1}{\text{Temperature (EPA)}}}$. The ability to achieve this CT is dependent on the dose-response curve, and must be confirmed through jar testing.				
(2) On-site oxygen generation was assumed.				

Table A.2 BAC Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
No. of Filters	No.	3	4	5
Filter Area	sq ft	113	113	113
Filter Depth	ft	5.0	5.0	5.0
Flow per Filter				
All Filters Operating	mgd	0.3	0.3	0.4
One Filter in Backwash	mgd	0.5	0.4	0.5
Hydraulic Loading				
All Filters Operating	gpm/sf	2	2	2
One Filter in Backwash	gpm/sf	2.8	2.7	2.8
EBCT				
All Filters Operating	min	20.3	18.7	16.9
One Filter in Backwash	min	13.5	14.0	13.5
Abbreviations: sq ft = square foot (feet); gpm/sq ft = gallons per minute per square foot; mgd = million gallons per day				
Notes:				
(1) During detailed design, the number of filters would likely decrease, and the size of the filters would increase.				

Table A.3 GAC Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	625	903	1250
No. of Trains (n)	No.	2	2	3
No. of Contactors/Train	No.	2.0	2.0	2.0
No. of Contactors Total	No.	4	4	6
Design Flow/Train	gpm	312	451	417
Contactore Diameter	ft	10.0	10.0	10.0
Liquid Loading Rate				
Liquid Loading Rate	gpm/sf	4	6	5
Empty Bed Contact Time				
Empty Bed Contact Time Total	min	28	20	21
Backwash System				
Backwash Hydraulic Loading Rate	gpm/sf	11	11	11
Backwash Duration	min	60	60	60
Backwash Tank Volume	gal	60000	60000	60000
Abbreviations: sq ft = square foot (feet); gpm/sq ft = gallons per minute per square foot; mgd = million gallons per day				

Table A.4 IX Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	625	903	1250
No. of Trains (n)	No.	1	1	2
No. of Contactors/Train	No.	2.0	2.0	2.0
No. of Contactors Total	No.	2	2	4
Design Flow/Train	gpm	625	903	625
Contactor Diameter	ft	10.0	10.0	10.0
Liquid Loading Rate				
Liquid Loading Rate	gpm/sf	4	6	5
Empty Bed Contact Time				
Empty Bed Contact Time Total	min	6	6	6
Abbreviations: sq ft = square foot (feet); gpm/sq ft = gallons per minute per square foot; mgd = million gallons per day				
Notes:				
(1) EBCT for maximum flow conditions.				

APPENDIX C

CONCEPTUAL COST OPINIONS

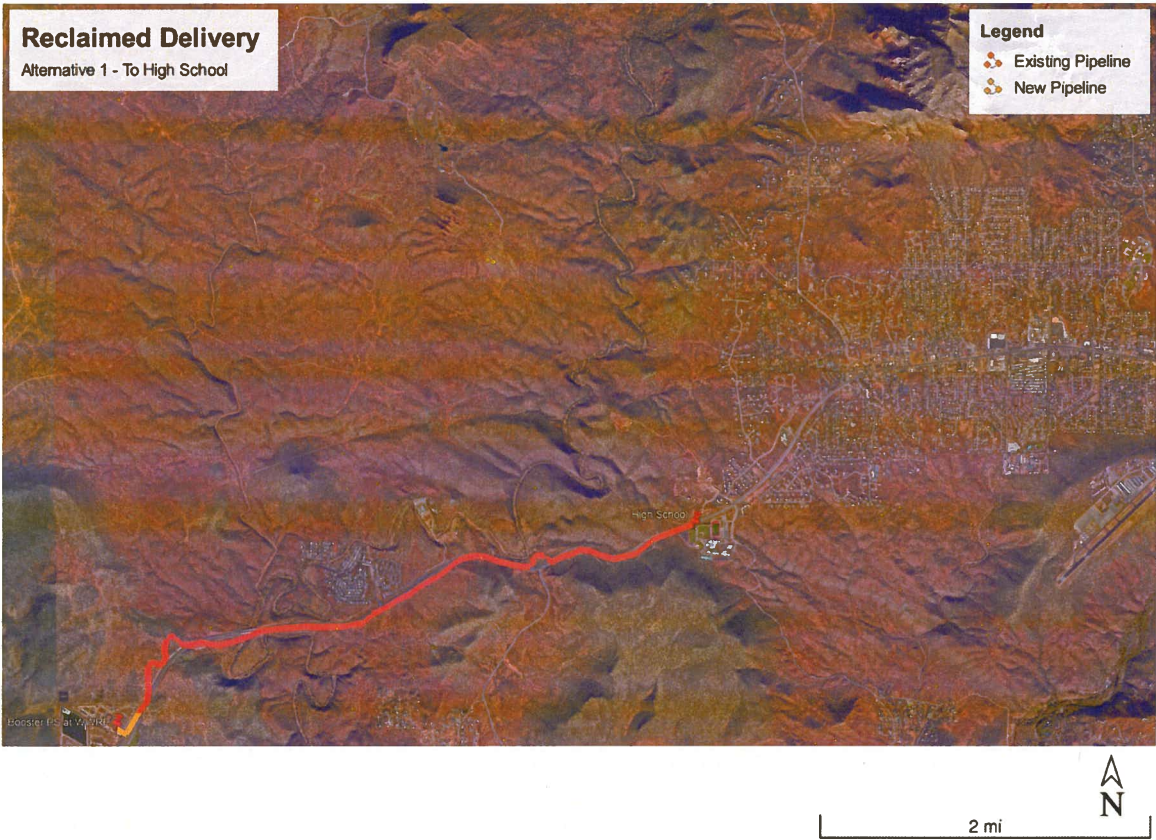
AACE International Class 5 Estimate (expected Accuracy Range of +100/-50 percent)	Factor	Full Flow PFAS Treatment			
		GAC		IX	
		Alternative 1 1.3 mgd	Alternative 2- Buildout 1.8 mgd	Alternative 1 1.3 mgd	Alternative 2 - Buildout 1.8 mgd
CAPITAL COST¹					
DIRECT COST					
Site Work ²	10%	\$580,000	\$580,000	\$511,000	\$609,000
Yard Piping & Valves ²	10%	\$580,000	\$580,000	\$511,000	\$609,000
Main Process Pipelines					
Backwash Supply		\$54,000	\$54,000	\$54,000	\$54,000
Backwash Recycle		\$231,000	\$231,000	\$231,000	\$215,000
Process Feed		\$198,000	\$198,000	\$198,000	\$131,000
Product Water		\$520,000	\$520,000	\$520,000	\$343,000
Foundation ³		\$283,000	\$365,000	\$243,000	\$327,000
Ozone Building		\$1,251,000	\$1,251,000	\$1,251,000	\$1,251,000
EQ Basin Cover		\$24,144	\$24,144	\$24,144	\$24,144
Process Equipment					
Ozone/BAF					
Ozone Feed Pumps		\$56,000	\$68,447	\$56,000	\$68,447
BAF Feed Pumps		\$56,000	\$68,447	\$56,000	\$68,447
On-site Oxygen Generator		\$260,000	\$260,000	\$260,000	\$260,000
Ozone System		\$1,915,000	\$1,915,000	\$1,915,000	\$1,915,000
BAF Pressure Vessels		\$1,180,000	\$1,475,000	\$1,180,000	\$1,475,000
BAF Media		\$381,000	\$476,154	\$381,000	\$476,000
IX					
IX Contactor Vessels	--	--	--	\$480,000	\$960,000
IX Resin	--	--	--	\$235,000	\$353,000
GAC					
GAC Contactors		\$960,000	\$1,440,000	--	--
GAC Media		\$200,000	\$300,000	--	--
Backwash Supply (BAF and GAC)					
Pumps		\$56,000	\$56,000	\$56,000	\$56,000
Backwash Recycle (BAF and GAC)					
Pumps		\$29,000	\$29,000	\$29,000	\$29,000
Bag Filter Elements		\$100	\$100	\$100	\$100
Bag Filter Housings		\$8,000	\$8,000	\$8,000	\$8,000
Product (GAC or IX Treated) Water Tank		\$562,000	\$562,000	\$562,000	\$562,000
Product Water Pumps		\$56,000	\$68,447	\$56,000	\$68,447
Installation ⁴	30%	\$1,699,000	\$1,997,000	\$1,565,000	\$1,869,000
Electrical ²	15%	\$1,113,000	\$1,309,000	\$1,026,000	\$1,225,000
I&C ²	15%	\$1,113,000	\$1,309,000	\$1,026,000	\$1,225,000
TOTAL DIRECT COST		\$13,370,000	\$15,140,000	\$12,430,000	\$14,180,000
Conceptual Level Design Detail Contingency ⁴	30%	\$4,011,000	\$4,542,000	\$3,729,000	\$4,254,000
TOTAL DIRECT COST WITH CONTINGENCY		\$17,380,000	\$19,680,000	\$16,160,000	\$18,430,000
INDIRECT COST					
General Conditions, Overhead, Profit & Risk ²	27%	\$4,693,000	\$5,314,000	\$4,363,000	\$4,976,000
Bonds and Insurance ⁵	3%	\$521,000	\$590,000	\$485,000	\$553,000
Sales Tax ³	9.85%	\$1,712,000	\$1,938,000	\$1,592,000	\$1,815,000
TOTAL INDIRECT COST		\$6,930,000	\$7,840,000	\$6,440,000	\$7,340,000
TOTAL CONSTRUCTION COST		\$24,310,000	\$27,520,000	\$22,600,000	\$25,770,000
Engineering and Administration ⁶	25%	\$6,077,500	\$6,880,000	\$5,650,000	\$6,442,500
TOTAL CAPITAL COST		\$30,390,000	\$34,400,000	\$28,250,000	\$32,210,000
ANNUAL OPERATION AND MAINTENANCE COST					
IX Media Changeout (including spent media management), High end estimate (40500 BV)	--	--	--	\$703,000	\$968,000
IX Media Changeout (including spent media management), Average end estimate (73,250 BV) ⁷	--	--	--	\$389,000	\$535,279
IX Media Changeout (including spent media management), Low estimate (106,000 BV)	--	--	--	\$268,000	\$370,000
GAC Media Changeout (including spent media management), High end estimate (2000 BV)		\$2,969,000	\$4,111,000	--	--
GAC Media Changeout (including spent media management), Average end estimate (13,500 BV) ⁷		\$440,000	\$609,000	--	--
GAC Media Changeout (including spent media management), Low end estimate (25,000 BV)		\$238,000	\$329,000	--	--
Pump Power Cost ⁸	\$0.13	\$20,000	\$28,000	\$27,000	\$34,000
Bag Filter Changeout ⁹		\$1,000	\$1,000	\$1,000	\$1,000
General ¹⁰	0.5%	\$87,000	\$98,000	\$81,000	\$92,000
Labor ¹⁰	\$35	\$145,000	\$145,000	\$145,000	\$145,000
TOTAL ANNUAL O&M (HIGH END ESTIMATE)		\$3,220,000	\$4,380,000	\$960,000	\$1,240,000
TOTAL ANNUAL O&M (AVERAGE END ESTIMATE)		\$690,000	\$880,000	\$840,000	\$810,000
TOTAL ANNUAL O&M (LOW END ESTIMATE)		\$490,000	\$600,000	\$520,000	\$640,000
ECONOMIC ANALYSIS					
Present Worth of Annual O&M ¹¹		\$9,377,000	\$11,959,000	\$8,698,000	\$11,008,000
PRESENT WORTH¹²		\$39,770,000	\$46,360,000	\$36,950,000	\$43,220,000
Annualized Capital Cost		\$2,240,000	\$2,530,000	\$2,080,000	\$2,370,000
EQUIVALENT ANNUAL COST¹¹		\$2,930,000	\$3,410,000	\$2,720,000	\$3,180,000
Table Notes					
¹ Cost opinions shown in July 2024 dollars (Handy-Whitman Average Water Infrastructure Index = 1311).					
² Applied to equipment costs.					
³ Includes costs for concrete equipment pads for vessels, pumps, sediment removal area and ozone contact basin.					
⁴ Applied to direct costs.					
⁵ Applied to direct costs with contingency.					
⁶ Applied to total construction cost.					
⁷ Median bed volume was used to calculate O&M cost.					
⁸ Pump power costs based on 50% operational cycle, \$0.13/kWh electricity costs, and maximum headloss across the process equipment.					
⁹ Bag filter costs assume 100% operational cycle with changeout assumed every 14 days.					
¹⁰ Assumes two operators at 40 hours per week each for a total of 80 hours. Hourly wage does not include any benefits, insurance, or retirement.					
¹¹ Assumes discount rate of 4% per year and term of 20 years.					

AACE International Class 5 Estimate (expected Accuracy Range of +100/-50 percent)	Factor	AWP Treatment and Delivery		
		Alternative 1 0.9 mgd	Alternative 2 1.3 mgd	Alternative 3 - Buildout 1.8 mgd
CAPITAL COST ¹				
DIRECT COST				
Site Work ²	5%	\$1,237,000	\$1,378,000	\$1,608,000
Yard Piping & Valves ²	10%	\$2,473,000	\$2,756,000	\$3,217,000
Foundation		\$426,000	\$426,000	\$467,000
Process Building		\$9,108,000	\$9,108,000	\$9,108,000
Electrical Building		\$1,348,000	\$1,348,000	\$1,348,000
Treated Water Conveyance				
New Pipeline		\$532,000	\$532,000	\$532,000
Reline Existing Pipeline		\$3,978,000	\$3,978,000	\$3,978,000
Access Pits		\$225,000	\$225,000	\$225,000
Process Equipment				
Advanced Water Treatment				
Process Feed Tanks ³		\$113,000	\$113,000	\$113,000
UF/MF ⁴		\$2,238,000	\$2,238,000	\$3,358,000
RO ⁵		\$1,567,000	\$1,567,000	\$1,567,000
UV-Chlorine AOP		\$2,143,000	\$2,143,000	\$2,143,000
Chemical Storage and Feed ⁶		\$382,000	\$382,000	\$382,000
Calcite Contactors		\$381,000	\$381,000	\$761,000
Chlorination ⁹		\$382,000	\$382,000	\$382,000
Treated Water Pump Station ⁷		\$302,000	\$453,000	\$604,000
Brine Management				
Process Feed Tanks ³		\$46,000	\$46,000	\$46,000
UF/MF ⁴		\$700,000	\$700,000	\$700,000
RO ⁵		\$676,000	\$676,000	\$676,000
IX		\$201,000	\$201,000	\$556,000
Lime Softening		\$2,795,000	\$3,865,000	\$4,866,000
Brine Concentrator		\$8,000,000	\$8,000,000	\$8,000,000
Evaporation Ponds ⁸		\$4,806,000	\$6,408,000	\$8,011,000
Installation ²	30%	\$7,420,000	\$8,267,000	\$9,650,000
Electrical ²	15%	\$4,823,000	\$5,373,000	\$6,272,000
I&C ²	15%	\$4,823,000	\$5,373,000	\$6,272,000
	TOTAL DIRECT COST	\$62,130,000	\$67,320,000	\$75,840,000
Conceptual Level Design Detail Contingency ⁹	30%	\$18,639,000	\$20,196,000	\$22,752,000
	TOTAL DIRECT COST WITH CONTINGENCY	\$80,770,000	\$87,520,000	\$98,590,000
INDIRECT COST				
General Conditions, Overhead, Profit & Risk ¹⁰	22%	\$17,769,000	\$19,254,000	\$21,690,000
Bonds and Insurance ¹⁰	3%	\$2,423,000	\$2,626,000	\$2,958,000
Sales Tax ¹⁰	9.85%	\$3,978,000	\$4,310,000	\$4,856,000
	TOTAL INDIRECT COST	\$24,170,000	\$26,190,000	\$29,500,000
	TOTAL CONSTRUCTION COST	\$104,940,000	\$113,710,000	\$128,090,000
Engineering and Administration ¹¹	25%	\$26,235,000	\$28,427,500	\$32,022,500
	TOTAL CAPITAL COST	\$132,180,000	\$143,140,000	\$161,110,000
ANNUAL OPERATION AND MAINTENANCE COST				
Chemicals, Membrane Replacements and Solids		\$960,000	\$1,386,000	\$1,918,000
Pump Power Cost ¹²	\$0.13	\$448,000	\$648,000	\$893,000
General ¹⁰	0.5%	\$404,000	\$438,000	\$493,000
Labor ¹²	\$35	\$867,000	\$867,000	\$867,000
Potable Delivery				
Pump Power Cost	\$0.13	\$85,000	\$170,000	\$255,000
General		\$65,000	\$67,000	\$69,000
	TOTAL ANNUAL O&M	\$2,829,000	\$3,576,000	\$4,495,000
ECONOMIC ANALYSIS				
Present Worth of Annual O&M ¹³		\$38,447,000	\$48,599,000	\$61,089,000
	PRESENT WORTH ¹³	\$170,630,000	\$191,740,000	\$222,200,000
Annualized Capital Cost		\$9,730,000	\$10,530,000	\$11,850,000
	EQUIVALENT ANNUAL COST ¹³	\$12,560,000	\$14,110,000	\$16,350,000
Table Notes				
¹ Cost opinions shown in July 2024 dollars (Handy-Whitman Average Water Infrastructure Index = 1311).				
² Applied to equipment costs.				
³ Includes UF and RO feed tanks.				
⁴ Includes UF strainers, feed pumps, and membranes.				
⁵ Includes feed pumps, pressure vessels, and CIP system.				
⁶ Includes chemical storage tank and feed pump skid, and sump pump.				
⁷ Cost assumes 100 HP vertical turbine pumps with 1 duty + 1 standby for Phase 1, 2 duty + 1 standby for Phase 2, 3 duty + 1 standby for Buildout.				
⁸ Cost assumes 3 acres of evaporation ponds for Phase 1, 4 acres of evaporation ponds for Phase 2, and 5 acres of evaporation ponds for Buildout.				
⁹ Applied to direct costs.				
¹⁰ Applied to direct costs with contingency.				
¹¹ Applied to total construction cost.				
¹² Assumes 12 operators of 40 hours per week each for a total of 80 hours. Hourly wage does not include any benefits, insurance, or retirement.				
¹³ Assumes discount rate of 4% per year and term of 20 years.				

AACE International Class 5 Estimate (expected Accuracy Range of +100/-50 percent)		Factor	Reclaimed Delivery		
			Alternative 1 High School	Alternative 2 High School + Sunset Park	Alternative 3 High School + All Parks
CAPITAL COST ¹					
DIRECT COST					
New Pipeline			\$566,000	\$1,647,000	\$6,175,000
Reline Existing Pipeline			\$4,838,000	\$7,538,000	\$7,538,000
Access Pits			\$225,000	\$350,000	\$350,000
Reservoir ⁴			\$477,000	\$477,000	\$477,000
Booster Pumps					
At WWRP ³			\$115,000	\$147,000	\$199,000
At High School ⁴			\$50,000	\$76,000	\$115,000
Installation ²	30%		\$50,000	\$67,000	\$94,000
Electrical ²	15%		\$32,000	\$44,000	\$61,000
I&C ²	15%		\$32,000	\$44,000	\$61,000
TOTAL DIRECT COST			\$6,390,000	\$10,390,000	\$15,070,000
Conceptual Level Design Detail Contingency ⁵	30%		\$1,917,000	\$3,117,000	\$4,521,000
TOTAL DIRECT COST WITH CONTINGENCY			\$8,310,000	\$13,510,000	\$19,590,000
INDIRECT COST					
General Conditions, Overhead, Profit & Risk ⁶	27%		\$2,244,000	\$2,244,000	\$2,244,000
Bonds and Insurance ⁶	3%		\$249,000	\$249,000	\$249,000
Sales Tax ⁶	9.85%		\$819,000	\$819,000	\$819,000
TOTAL INDIRECT COST			\$3,310,000	\$3,310,000	\$3,310,000
TOTAL CONSTRUCTION COST			\$11,620,000	\$16,820,000	\$22,900,000
Engineering and Administration ⁷	25%		\$2,905,000	\$4,205,000	\$5,725,000
TOTAL CAPITAL COST			\$14,530,000	\$21,030,000	\$28,630,000
ANNUAL OPERATION AND MAINTENANCE COST					
Pump Power Cost ⁹	\$0.13		\$24,147	\$49,621	\$120,072
General ⁶	1%		\$145,300	\$210,300	\$286,300
TOTAL ANNUAL O&M			\$170,000	\$260,000	\$410,000
ECONOMIC ANALYSIS					
Present Worth of Annual O&M ¹³			\$2,310,000	\$3,533,000	\$5,572,000
PRESENT WORTH ¹³			\$16,840,000	\$24,560,000	\$34,200,000
Annualized Capital Cost			\$1,070,000	\$1,550,000	\$2,110,000
EQUIVALENT ANNUAL COST ¹³			\$1,240,000	\$1,810,000	\$2,520,000
Table Notes					
¹ Cost opinions shown in July 2024 dollars (Handy-Whitman Average Water Infrastructure Index = 1311).					
² Applied to equipment costs.					
³ Booster pumps located at the WWRP would pump from the PFAS treatment product water tank to the High School. All alternatives assume two pumps with n+1 redundancy.					
⁴ A reservoir and booster pump station at the High School would pump flow to its final irrigation location. All alternatives assume two pumps with N+1 redundancy.					
⁵ Applied to direct costs.					
⁶ Applied to direct costs with contingency.					
⁷ Applied to total construction cost.					

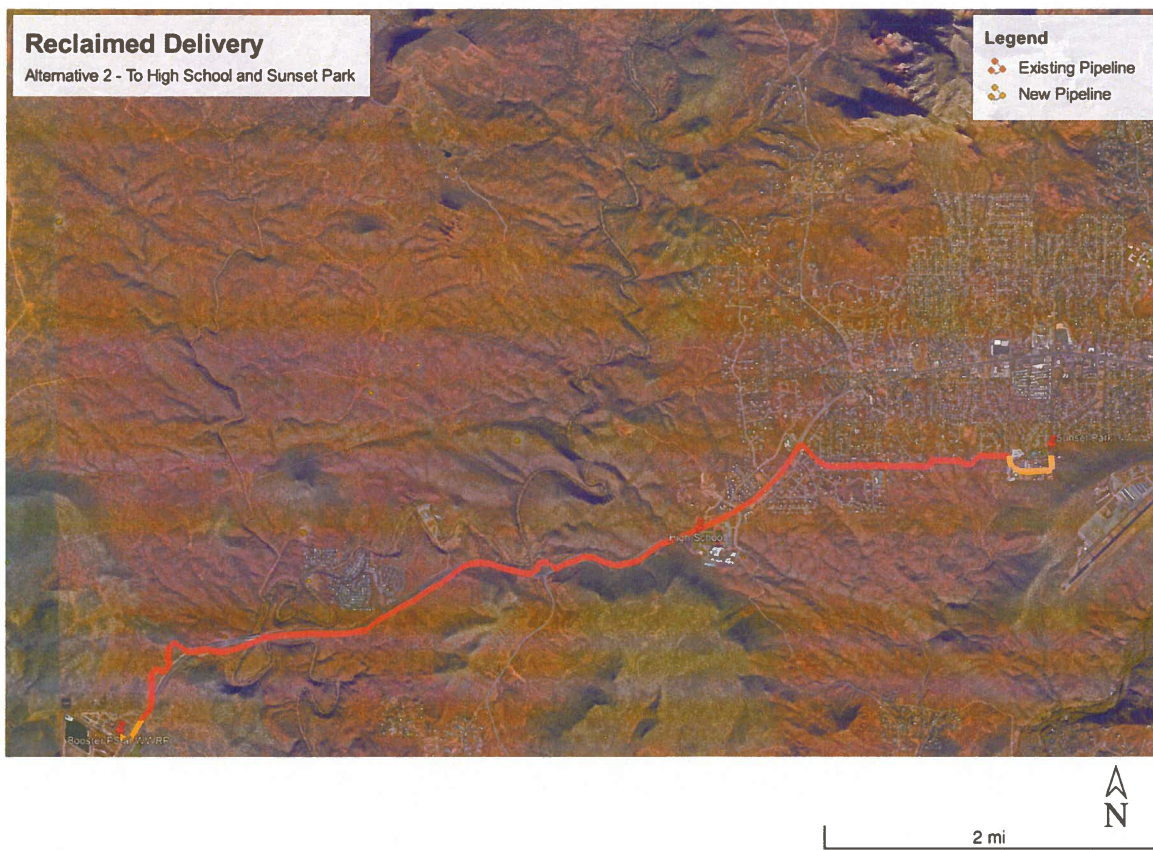
APPENDIX D

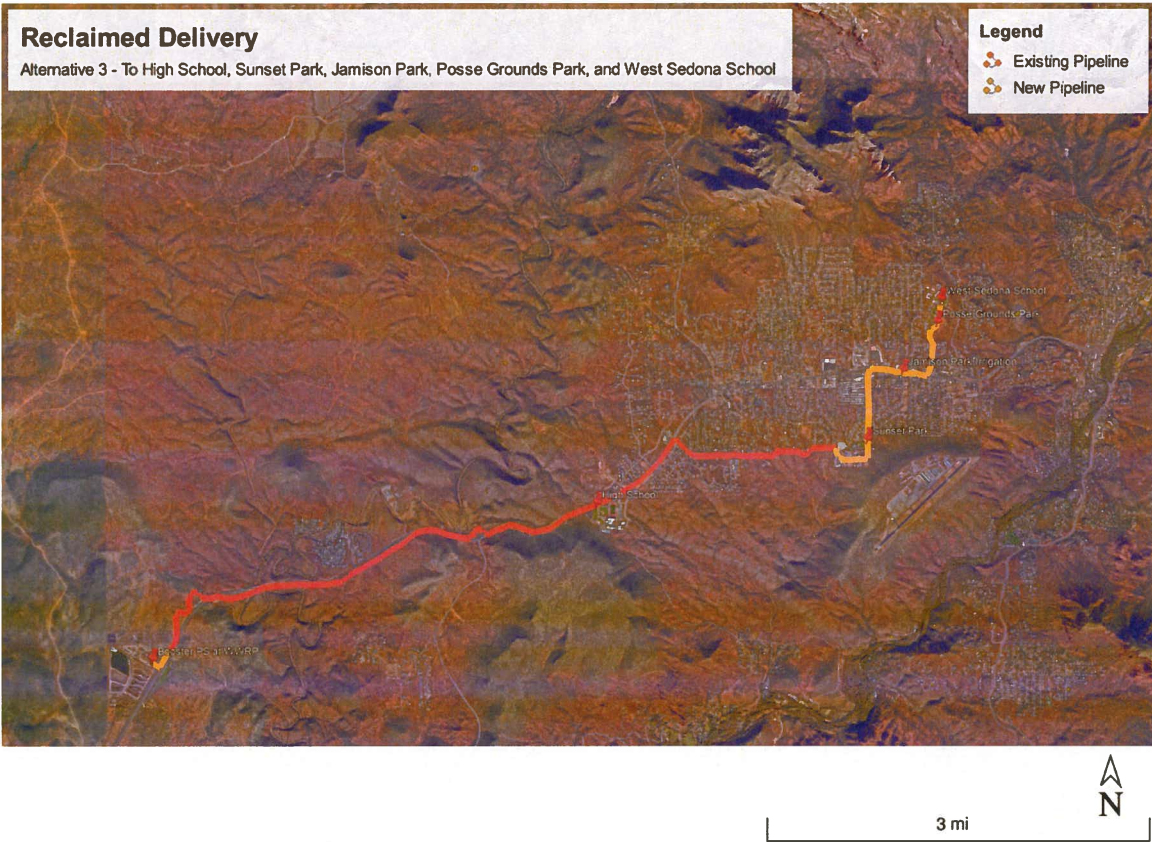
DELIVERY ALTERNATIVE SCHEMATICS

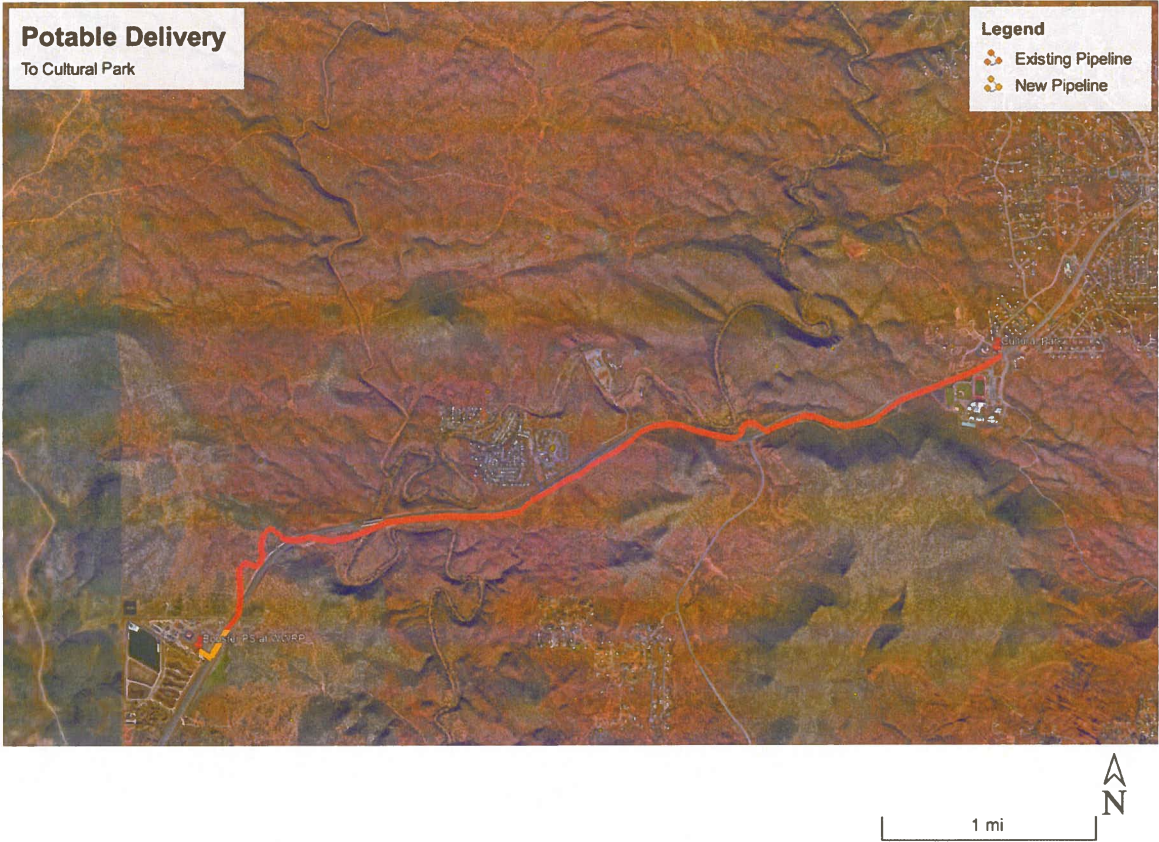


CITY OF SEDONA

D-1







APPENDIX E

AWP TREATMENT DESIGN CRITERIA

Table B.1 UF Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Automatic Backwashing Strainers				
Design Capacity per Screen	gpm	625	903	1250
Number of Strainers				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total	No.	2	2	2
Filter Media Type		Wedgewire		
Filter Material		316 SST		
Filter Rating	microns	200	200	200
Clean Pressure Drop	psi	3	3	3
Maximum Pressure Drop	psi	7.0	7.0	7.0
Maximum Backwash Flow Rate	gpm	220	220	220
Backwash Duration	sec	36	36	36
UF Process				
Type		Pressurized, Polymeric Hollow Fiber Ultrafiltration (UF)		
Overall Recovery	%	95	95	95
Number of UF Trains				
In-Service	No.	1	1	1
Reliability	No.	1	1	2
Total	No.	2	2	3
Installed Modules per Train	No.	34	44	64
Maximum Modules per Train	No.	108	108	108
Flow Criteria				
System Feed Rate	gpm	625	903	1250
Feed Water Loss	%	2.5	2.5	2.5
Gross Filtrate Production	gpm	609	880	1219
Filtrate Losses	%	0.025	0.025	0.025
Overall Recovery	%	95	95	95
System Net Filtrate	gpm	594	858	1187
Online Factor	%	88	88	88

Table B.1 UF Design Criteria

Instantaneous Filtrate Production	gpm	691	999	1383
Module Criteria				
Membrane Area Per Module	sf	775	775	775
Membrane Area Per Train	sf	26350	34100	49600
Membrane Area Total	sf	52700	68200	148800
Gross Flux Rate	gfd	33.3	37.2	35.4
Backwash Criteria				
Type	Reverse Flow Followed By Air Scour and Drain			
Backwash Interval Per Train				
Minimum	min	20	20	20
Maximum	min	30	30	30
Filtration Flow ⁽¹⁾	Ratio	1.1	1.1	1.1
Backwash Supply Flowrate	gpm	761	1099	1521
Backwash Duration	sec	30	30	30
Air Scour Flowrate	ACFM	238	308	448
Air Scour Duration	sec	30-60	30-60	30-60
Forward Flush Flowrate	gpm	612	792	1152
Abbreviations: psi - pounds per square inch; sq ft - square foot; gpm - gallons per minute				
Notes:				
(1) The ratio between instantaneous filtrate production and the backwash supply flow rate.				

Table B.2 RO Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Feed Flowrate	gpm	625	903	1250
Recovery	%	80	80	80
Permeate Flowrate	gpm	500	722	1000
Concentrate Flowrate	gpm	125	181	250
Feed Flowrate Per Train	gpm	625	903	1250
Permeate Flowrate per Train	gpm	500	722	1000
Concentrate Flow per Train	gpm	125	181	250
Number of RO Trains				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total	No.	2	2	2
Staging of RO Trains				
1st Stage				
Pressure Vessels per Train	No.	14	21	29
Elements per Pressure Vessels	No.	7	7	7
2nd Stage				
Second Stage	No.	7	10	14
Elements per Pressure Vessels	No.	7	7	7
Number of Elements				
Per Train	No.	147	217	301
Total (In-service)	No.	294	434	602
Membrane Area				
Per Element	sq ft	400	400	400
Per Train	sq ft	58800	86800	120400
Total (In-service)	sq ft	58800	86800	120400
Average Flux Rate	gfd	12	12	12
Abbreviations: gfd = gallon per square foot per day ; μ m = micrometer; ACFM = actual cubic feet per minute				
Notes:				
(1) Larger trains with more membrane elements may be selected during detailed design.				

Table B.3 UV Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	0.9	1.3	1.8
UV Model		UV Flex 200		
UV Reactors				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total	No.	2	2	2
UVT	percent	96	96	96
Number of Lamps Per Reactor	No.	24	24	24
Total Numbers of Lamps (In-service)	No.	24	24	24
Total Numbers of Lamps (Total)	No.	48	48	48
UV System				
Inlet NDMA	µg/L	2 – 29	2 – 29	2 – 29
Outlet NDMA	µg/L	<10	<10	<10
Design NDMA LRV	Log	0.5	0.5	0.5
Design 1,4-dioxane LRV	Log	0.5	0.5	0.5
NDMA RED	mJ/cm ²	1611	1611	1611
Free Chlorine Dose Injection	mg/L as Cl ₂	7	7	7
Free Chlorine Dose at UV Inlet	mg/L as Cl ₂	3	3	3
Abbreviations: UVT = ultraviolet transmittance ; µg/L = microgram per liter; mJ/cm ² = millijoule per square centimeter				

Table B.4 Calcite Contactors Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Side-Stream Flow %	%	0.5	0.5	0.5
Design Flow	mgd	0.4	0.6	0.9
Number of vessels	No.	1.0	1.0	2.0
Vessels Area	sq ft	113	113	113
Media Depth	ft	6	6	6
Flow per filter				
All filters operating	mgd	0.4	0.6	0.4
One filter in backwash	mgd	--	--	0.9
Flow per filter				
All filters operating	gpm/sf	3	4	3
One filter in backwash	gpm/sf	--	--	5
EBCT				
All filters operating	min	16	11	16
One filter in backwash	min	--	--	8
Backwash System				
Backwash Loading Rate	gpm/sf	11	11	11
Backwash Flowrate	gpm/sf	1243	1243	1243
Backwash Duration	min	30	15	45
Backwash Volume	gal	37290	18645	55935
Calcite Loading				
Dose	mg/L	140	140	140
Calcite Consumption	ppd	518	749	1037
Calcite Storage	lbs.	15555	22468	31110
Number of bags	No.	311	449	622
Abbreviations: mgd - million gallons per day; No. - number; gpm/sf - gallons per minute per square foot; min - minute; gal - gallon; mg/L - milligrams per liter; ppd - pounds per day; lbs - pounds				
Notes:				
(1) Vessel area based on a 12-foot diameter vertical vessel.				

Table B.5 Clearwell Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
AWT Design Flow	mgd	0.9	1.3	1.8
Storage/Detention Time	hrs	5.40	--	2.70
Storage Volume	Mgal	0.2	--	0.2
Baffling Factor	No.	0.5	--	0.5
Effective Time	min	162	--	81
Abbreviations:				
mgd - million gallons per day; hrs - hours; Mgal - million gallons; No. - number; min - minutes				

Table B.6 Lime Softening Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	0.2	0.3	0.4
Recovery	%	100	100	100
Basin Type	--	Concrete		
Number of Trains				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total	No.	2	2	2
Unit Capacity	gpm	1500	1500	1500
Total Capacity	gpm	1500	1500	1500
Abbreviations: mgd - million gallons per day; No. - number; gpm - gallons per minute				

Table B.7 Secondary UF Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Automatic Backwashing Strainers				
Design Capacity per Screen	gpm	125	181	250
Number of Strainers				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total	No.	2	2	2
Filter Media Type		Wedgewire		
Filter Material		316 SST		
Filter Rating	microns	200	200	200
Clean Pressure Drop	psi	3	3	3
Maximum Pressure Drop	psi	7.0	7.0	7.0
Maximum Backwash Flow Rate	gpm	220	220	220
Backwash Duration	sec	36	36	36
UF Process				
Type		Pressurized, Polymeric Hollow Fiber Ultrafiltration (UF)		
Overall Recovery	%	95	95	95
Number of UF Trains				
In-Service	No.	1	0	1
Reliability	No.	1	0	1
Total	No.	2	2	2
Installed Modules per Train	No.	8	4	12
Maximum Modules per Train	No.	108	108	108
Flow Criteria				
System Feed Rate	gpm	125	181	250
Feed Water Loss	%	2.5	2.5	2.5
Gross Filtrate Production	gpm	122	176	244
Filtrate Losses	%	2.5	2.5	2.5
Overall Recovery	%	0.95	0.95	0.95
System Net Filtrate	gpm	119	172	237

Table B.7 Secondary UF Design Criteria

Online Factor	%	1	1	1
Instantaneous Filtrate Production	gpm	149	216	299
Module Criteria				
Membrane Area Per Module	sf	775	775	775
Membrane Area Per Train	sf	6200	3100	9300
Membrane Area Total	sf	12400	--	18600
Gross Flux Rate	gfd	28.3	--	37.7
Backwash Criteria				
Type		Reverse Flow Followed By Air Scour and Drain		
Backwash Interval Per Train				
Minimum	min	20	20	20
Maximum	min	30	30	30
Filtration Flow ⁽¹⁾	Ratio	1.1	1.1	1.1
Backwash Supply Flowrate	gpm	164	237	329
Backwash Duration	sec	30	30	30
Air Scour Flowrate	ACFM	56	28	84
Air Scour Duration	sec	30-60	30-60	30-60
Forward Flush Flowrate	gpm	144	72	216
Abbreviations: psi - pounds per square inch; sq ft - square foot; gpm - gallons per minute				
Notes:				
(1) The ratio between instantaneous filtrate production and the backwash supply flow rate.				

Table B.8 Secondary RO Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	gpm	116	168	233
Recovery	%	85	85	85
Permeate Flowrate	gpm	99	143	198
Concentrate Flowrate	gpm	17	25	35
Feed Flowrate Per Train	gpm	116	168	233
Permeate Flowrate per Train	gpm	99	143	198
Concentrate Flow per Train	gpm	17	25	35
Number of RO Trains				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total	No.	2	2	2
Staging of RO Trains				
1st Stage				
Pressure Vessels per Train	No.	34	34	34
Elements per Pressure Vessels	No.	7	7	7
2nd Stage				
Pressure Vessels per Train	No.	14	14	14
Elements per Pressure Vessels	No.	7	7	7
3rd Stage				
Pressure Vessels per Train	No.	6	6	6
Elements per Pressure Vessels	No.	7	7	7
Number of Elements				
Per Train	No.	378	378	378
Total (In-service)	No.	756	756	756
Membrane Area				
Per Element	sq ft	400	400	400
Per Train	sq ft	151200	151200	151200
Total (In-service)	sq ft	151200	151200	151200
Average Flux Rate	gfd	0.9	1.4	1.9

Abbreviations: sq ft - square foot; gpm - gallons per minute; min - minute; gfd - gallons per square feet per day

Table B.9 CMF IX Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	0	0	0
Recovery	%	0.98	0.98	0.98
Treated Flow	mgd	0	0	0
Vessel Diameter	ft	6	6	6
No. of Vessels				
In-Service	No.	1	1	1
Reliability	No.	1	1	1
Total No. of Vessels	No.	2	2	2
Flow per Vessel	gpm	119	172	237
Loading Rate	gpm/sf	4	6	8
Resin Type	--	Acid/Base Regen IX (DOW/ROHM & HAAS)		
Resin Volume	cf	84	84	84
EBCT	min	5.3	3.7	2.6
Abbreviations: mgd - million gallons per day; No. - number; gpm/sf - gallons per minute per square foot; min - minute; gal - gallon; mg/L - milligrams per liter; ppd - pounds per				

Table B.11 CMF Multi-effect Vapor Recompression Conceptual Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	0.03	0.04	0.06
System Recovery	%	90	90	90
Number of Trains				
In-Service	No.	1	1	1
Reliability	No.	0	0	0
Total No. of Vessels	No.	1	1	1
Unit Capacity	gpm	250	250	250
Abbreviations: mgd - million gallons per day; Mgal - million gallons; No. - number; gpm - gallons per minute				

Table B.12 CMF Evaporation Pond Design Criteria

Process and Criteria	Unit	Alternatives		
		Phase 1	Phase 2	Buildout
Design Capacity	mgd	0.003	0.004	0.006
Evaporation Pond Area	Acres	3	4	5
Loading Rate	gpm/acre	0.63	0.7	0.75