Appendix B

Revised Water and Wastewater Feasibility Study

Acorn Environmental – Scotts Valley Rancheria

Water and Wastewater Feasibility Study

Prepared by HydroScience Engineers, Inc.

November 2024

HydroScience

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AF	acre-feet
AFY	acre-feet per year
bgs	below ground surface
BOD	biochemical oxygen demand
City	City of Vallejo
CFR	Code of Federal Regulations
CT	product of chlorine residual and modal contact time measured at the same
District	Vallejo Flood and Wastewater District
DU	dwelling unit
DWR	Department of Water Resources
ET	evapotranspiration rate
Ft	feet
Ft^2	square feet
gal	gallons
gpd	gallons per day
gpm	gallons per minute
LS	lump sum
MBR	membrane bioreactor
MCL	Maximum Contaminant Level
MG	million gallons
mg/L	milligrams per liter
µg/L	micrograms per liter
MGD	million gallons per day
MPN	Most Probable Number
NPDES	National Pollution Discharge Elimination System
NTU	nephelometric turbidity units
PLC	programmable logic controller
RWFP	Recycled Water Facilities Plan
RWQCB	Regional Water Quality Control Board
SWRCB	State Water Resources Control Board
SDS	Safety Data Sheets
sf	square feet
TSS	total suspended solids
UV	Ultraviolet
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
VFWD	Vallejo Flood and Wastewater District
WWTP	Wastewater Treatment Plant

SECTION 1 – INTRODUCTION

HydroScience Engineers, Inc. (HydroScience) was retained by Acorn Environmental to prepare a feasibility study evaluating the regulatory, technical, and engineering issues associated with supplying water and handling wastewater from the Scotts Valley Casino (Project) proposed by the Scotts Valley Band of Pomo Indians (Tribe) of Northern California. The objectives of this water and wastewater feasibility study are to:

- Estimate the proposed Project's water supply and wastewater disposal requirements;
- Describe the facilities that would be necessary to supply the required water, treat the required wastewater, and identify possible connections to existing public infrastructure;
- Develop a strategy for disposing of wastewater generated by the Project; and
- Identify applicable water and wastewater permitting issues for the proposed Project.

This report evaluates these objectives for three Project alternatives located at the project site:

- Alternative A – Proposed Project consists of tribal housing and an administrative building, casino, with event/multipurpose space, restaurants, parking structure, and surface parking lots.
- Alternative B – Reduced Intensity Project plan consists of Alternative A casino without tribal housing and the administrative building.
- Alternative C – Non-Gaming Project is the third development alternative (non-gaming) which consists of hotels, commercial buildings, tribal housing and tribal administrative buildings.

This document describes each alternative's water supply and wastewater requirements, identifies projected flows and demands, and evaluates alternative effluent disposal strategies.

SECTION 4 and **SECTION 5** present a plan summarizing the facilities required to meet the more conservative objectives for Alternative A.

1.1 Proposed Project Site Alternatives

The proposed Project would be constructed within the City of Vallejo (City) boundaries (**Figure 1-1**). The 160-acre (ac) site consists of four parcels located at the intersection of I-80 and Columbus Parkway would be brought into Trust as part of the proposed Project. A map of the location of the site is shown in **Figure 1-2**.

As further described in **Section 2.1**, three separate programs, each comprising of different densities and facilities, will be evaluated as part of this analysis: Alternative A – Proposed Project, Alternative B – Reduced Intensity Project and Alternative C – Non-Gaming Project. See **Appendix A** for a full list of the proposed facilities.

1.2 Report Organization

This report is divided into seven sections as listed below.

- Section 1 – Introduction
- Section 2 – Project Wastewater Flows and Water Demands
- Section 3 – Background and Regulatory Issues
- Section 4 – Water Facility Requirements
- Section 5 – Wastewater Facility Requirements
- Section 6 – Recommendations
- Section 7 – References

Figure 1-1
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Vicinity and Project Location Map

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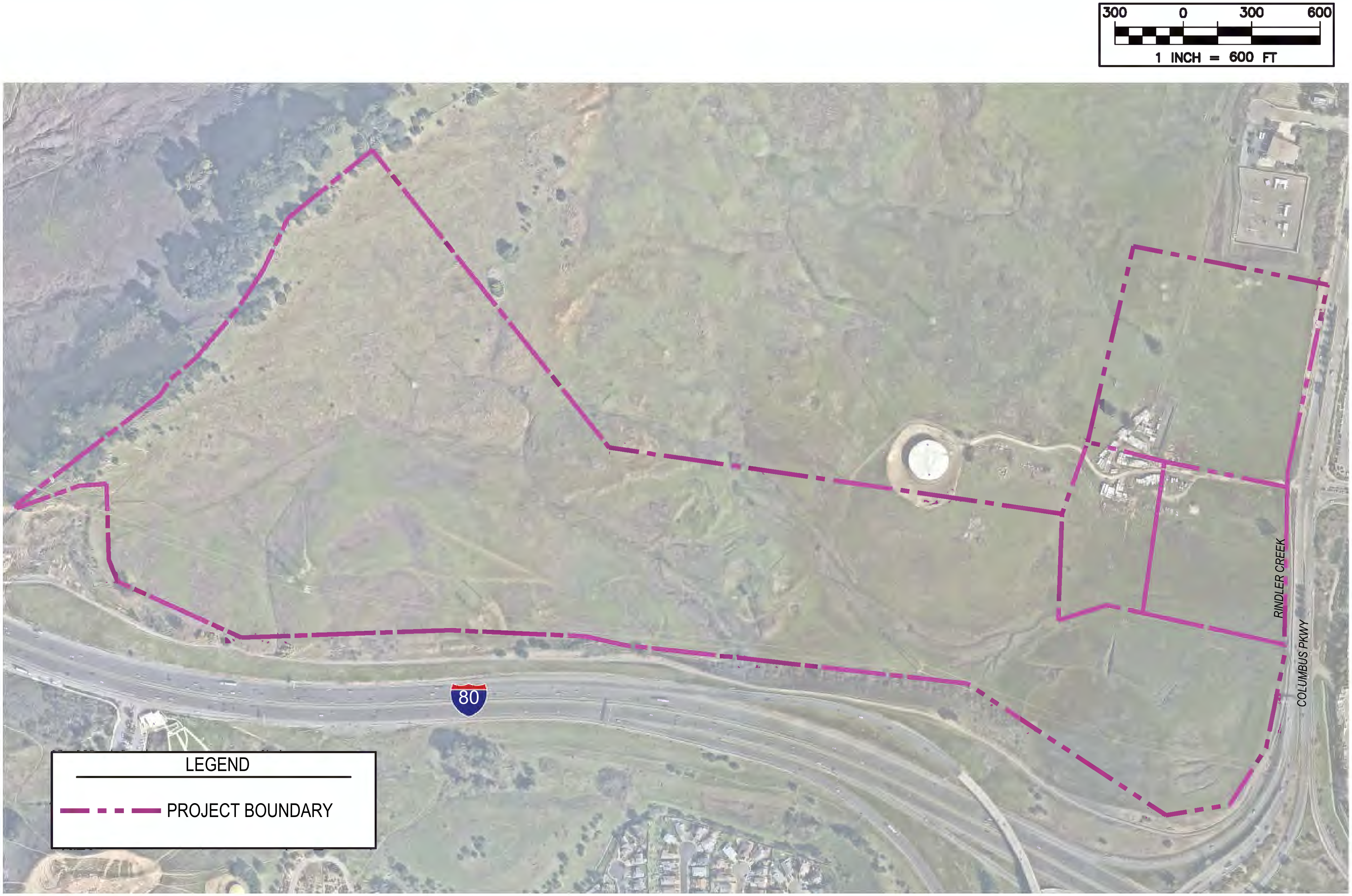

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Figure 1-2
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Aerial Site Plan

SECTION 2 – PROJECT WASTEWATER FLOWS AND WATER DEMANDS

This section provides a summary of each of the three program alternatives and the related water and wastewater facility requirements. For each program alternative, the following information is summarized:

- Wastewater generated, including discussions about wastewater quality;
- Effluent reuse and disposal options; and
- Water supply requirements.

Each program alternative is individually described below.

2.1 Program Alternatives

The three program alternatives that are considered in this feasibility study to understand the range of water and wastewater facility needs are each summarized below:

- **Alternative A:** This program includes a total approximate footprint of 615,000 square feet (ft^2), including a casino, multiple restaurants and bars, and a ballroom. Approximately 1,600,000 ft^2 of on-site parking spaces (guest/employee), valet, bus depot, and a loading dock will be located on the site. This program also includes a tribal community which includes 24 single-family homes and a 12,600 ft^2 administrative building. A map of the Alternative A program site plan is included as **Figure 2-1**.
- **Alternative B:** This program includes Alternative A as described above, without the tribal community. A map of the Alternative B program site plan is included as **Figure 2-2**.
- **Alternative C:** This program includes a total approximate footprint of 141,000 ft^2 of hotels and approximately 130,000 ft^2 of commercial space. This program also includes a tribal community of 50 single-family homes and three separate administrative buildings with a total approximate footprint of 23,000 ft^2. A map of the Alternative C program site plan is included as **Figure 2-3**.

Figure 2-1
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Proposed Site Plan - Alternative A

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Figure 2-2
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Proposed Site Plan - Alternative B

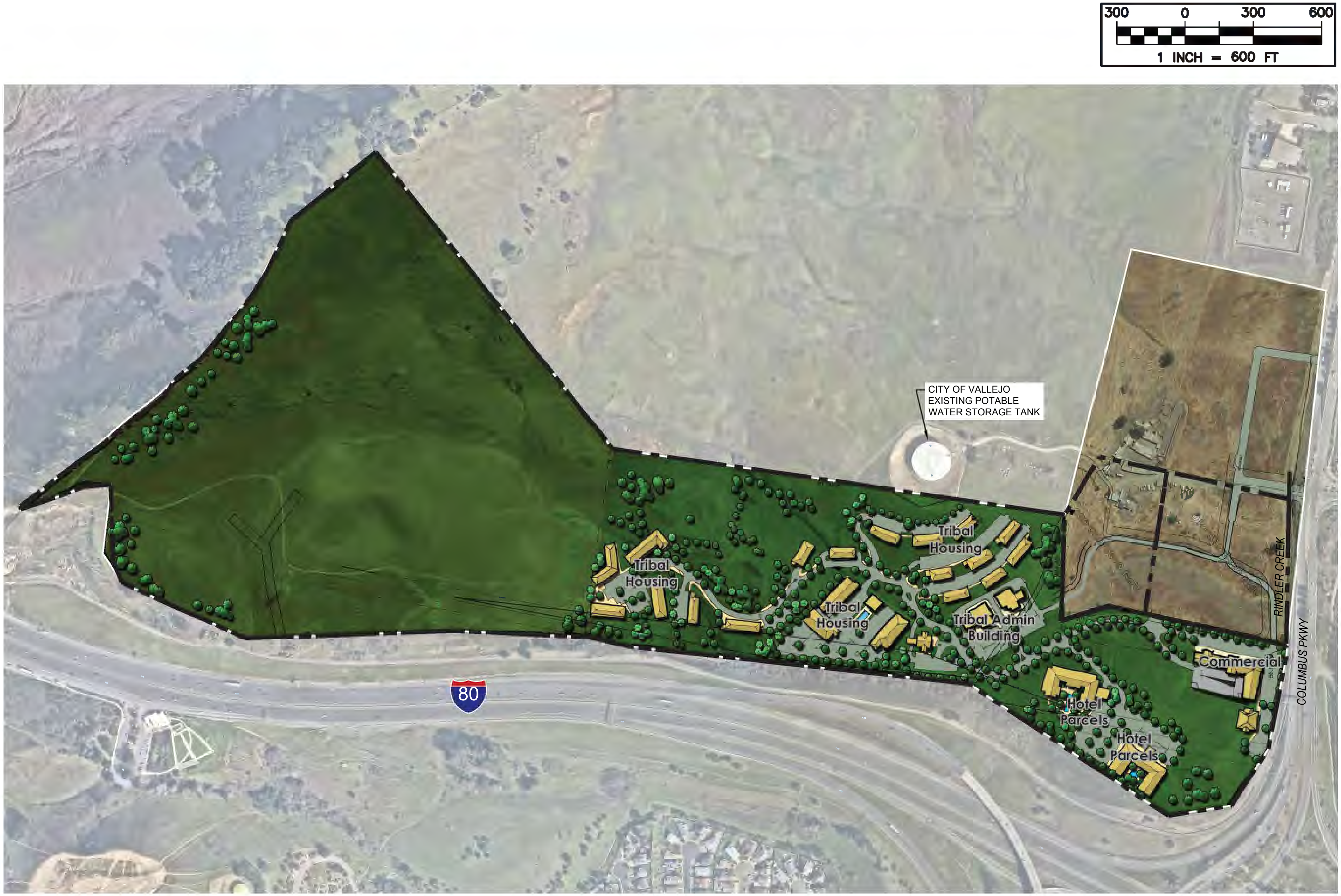

Figure 2-3
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Proposed Site Plan - Alternative C

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2.2 Wastewater

This section identifies the expected strength of wastewater and projected flows for each program alternative.

2.2.1 Wastewater Quality

The quality of influent water for gaming facilities differs from the quality of domestic sewage; typical gaming facility wastes have higher biochemical oxygen demand (BOD) and total suspended solids (TSS) values compared to domestic wastewater, as identified in **Table 2-1**. Shock loadings are also typical of gaming facility wastewater. Wastewater shock loading occurs when a WWTP receives a high flow or high strength discharge outside of its normal loading ranges. Weekend flows are much higher than weekday flows, and evening flows are higher than daytime flows. This occurs due to the higher utilization of casino facilities outside of normal business hours.

Table 2-1: Typical WWTP Influent Water Quality

Parameter	Units	Alternative A	Typical Domestic Sewage
BOD	mg/L	450-600	200-300
TSS	mg/L	450-600	200-300

2.2.2 Wastewater Flows

Average weekday and peak weekend flows for Alternative A, B, and C were developed based on analysis of similar facilities. Real-time data and previous experience developing wastewater flow projections from similar facilities were compared and the most conservative was used to estimate the unit flows for the proposed Project. An occupancy level factor was used to estimate flows for a typical weekday and weekend. The average day flow was estimated using the weighted average of the weekday and weekend estimated flow projections. For non-gaming facilities such as Tribal housing and community buildings, the same weekday and weekend factor was applied. These projections are based on the three Alternative programs provided by Acorn.

Table 2-2 through **Table 2-4** summarize the projections of wastewater volumes generated by Alternative A, B, and C, respectively.

For the full flow projection table see **Appendix A**.

Table 2-2: Projected Wastewater Flows for Alternative A

Area Description	Estimated Occupancy			Wastewater Flow (gpd)	
	Number	Units	gpd/Unit	Wt. Average	Weekend
Casino Gaming and Support Areas	481,988	SF	0.6	102,000	143,000
Employees	3,600	employees	12	35,000	35,000
Restaurants	811	Seats	70	36,000	45,000
Bars & Brew Pub	602	Seats	40	11,000	19,000
Coffee Shop	74	Seats	40	1,000	2,000
Food Hall	182	Seats	60	7,000	9,000
Ballroom / Pre-Function Area	52,794	SF	0.75	14,000	32,000
Cooling Tower Makeup	1	SF	26,737	3,000	3,000
Single-Family Homes	24	EDU	290	7,000	7,000
Administrative Building	30	employees	12	1,000	1,000
Total Wastewater Generated				**217,000**	**296,000**

Notes:
1. Support facilities are lump sum values for back-of-house for casino, lobby, cashier and club.
2. All flows are rounded to the nearest 1,000 gpd.
3. Total wastewater generated sum may be off due to rounding of individual facility wastewater generated.
4. Weighted average is the sum of the weekday flows over four days plus the sum of the weekend flows over three days divided by seven days.

Based on the wastewater generation rates identified in **Table 2-2**, the Project must have the capability to convey or treat the maximum weekend demand of approximately 300,000 gpd for Alternative A.

Table 2-3: Projected Wastewater Flows for Alternative B

Area Description	Estimated Occupancy			Wastewater Flow (gpd)	
	Number	Units	gpd/Unit	Wt. Average	Weekend
Casino Gaming and Support Areas	481,988	SF	0.6	102,000	143,000
Employees	3,600	employees	12	35,000	35,000
Restaurants	811	Seats	70	36,000	45,000
Bars & Brew Pub	602	Seats	40	11,000	19,000
Coffee Shop	74	Seats	40	1,000	2,000
Food Hall	182	Seats	60	7,000	9,000
Ballroom / Pre-Function Area	52,794	SF	0.75	14,000	32,000
Cooling Tower Makeup	1	SF	26,737	3,000	3,000
Total Wastewater Generated				**209,000**	**288,000**

Notes:
1. Support facilities are lump sum values for back-of-house for casino, lobby, cashier and club.
2. All flows are rounded to the nearest 1,000 gpd.
3. Total wastewater generated sum may be off due to rounding of individual facility wastewater generated.
4. Weighted average is the sum of the weekday flows over four days plus the sum of the weekend flows over three days divided by seven days.

Based on the wastewater generation rates identified in **Table 2-3**, the Project must have the capability to convey or treat the maximum weekend demand of approximately 300,000 gpd for Alternative B.

Table 2-4: Projected Wastewater Flows for Alternative C

Area Description	Estimated Occupancy			Wastewater Flow (gpd)	
	Number	Units	gpd/Unit	Wt. Average	Weekend
Hotel 1	132	rooms	250	20,000	33,000
Hotel 2	132	rooms	250	20,000	33,000
Cooling Tower Makeup	1	SF	6,131	1,000	1,000
Commercial (2)	129,702	SF	0.1	6,000	10,000
Single-Family Homes	50	EDU	290	15,000	15,000
Administrative Building (3)	90	employees	12	1,000	1,000
Total Wastewater Generated				**63,000**	**93,000**

Notes:
1. All flows are rounded to the nearest 1,000 gpd.
2. Total wastewater generated sum may be off due to rounding of individual facility wastewater generated.
3. Weighted average is the sum of the weekday flows over four days plus the sum of the weekend flows over three days divided by seven days.

Based on the wastewater generation rates identified in **Table 2-4**, the Project must have the capability to convey or treat the maximum weekend demand of approximately 100,000 gpd for Alternative C.

Summary of Projected Design Flows for each Alternative

Table 2-5 summarizes the proposed design flows for Alternative A, B, and C based on the weekend capacity. The design flows are at least 20% higher than the projected flows in order to provide a factor of safety for planning and design to account for the typical diurnal variation. Additional storage will also be provided for equalization of the peak daily flows.

Table 2-5: Summary of Design Flows for Alternative A, B & C

Program Alternative	Parameter	Projected Wastewater Flow (gpd)	Design Flow (gpd)
Alternative A	Average Daily Flow	217,000	300,000
	Average Weekend Flow	323,000	400,000
Alternative B	Average Daily Flow	209,000	300,000
	Average Weekend Flow	312,000	400,000
Alternative C	Average Daily Flow	63,000	100,000
	Average Weekend Flow	93,000	100,000

2.2.3 Effluent Reuse and Disposal

For any alternative considering an on-site WWTP, the WWTP will treat wastewater to a tertiary level and allow the Project to consider a wide range of effluent disposal options. Tertiary treatment is typically defined as a process that has undergone primary treatment consisting of a gravity settling process, secondary treatment consisting of a biological process, and tertiary treatment consisting of both a filtration and a disinfection process. These treatment processes can be combined into one process spanning the different types of treatment.

If available, recycled water meeting Title 22 criteria will be used in the casino restrooms for toilet and urinal flushing. Although the use of recycled water in the restrooms is on Trust lands, the recycled water quality will be designed to produce the equivalent water quality to disinfected tertiary recycled water as defined by Title 22. In general, this quality of recycled water is available for all approved non-potable uses in the State of California.

Recycled water will also be used for cooling tower makeup. This will help reduce storage requirements through cooling tower drift, evaporation system leakage losses, and blowdown. The brine generated as a byproduct of the recycled water treatment will be hauled off-site. Common disposal alternatives include evaporative ponds, disposal to ocean, deep well injection, incineration, additional treatment to concentrate waste, etc. Given the limited area for additional treatment or evaporative ponds, it is anticipated that the brine will be disposed of off-site. Estimation for brine volume, concentration, and disposal will be determined based on source water quality, generated wastewater volume and quality, and specific treatment components.

In order to evaluate other wastewater disposal strategies, the following assumptions were made:

- Recycled water use on-site will be maximized.
- The Project must identify a reliable wet season disposal method.
- The Project must comply with all applicable regulatory requirements.

Landscape Irrigation

The primary criteria used to determine the required landscape irrigation demands are evapotranspiration (ET) rates and precipitation information. Water demands per acre of irrigated area are calculated for each month based on ET rates and precipitation records with an additional factor to account for a very wet year. This monthly demand is then used to calculate an annual disposal capacity per acre in such a wet year.

ET Rates: ET is a measure of water usage by a particular plant or crop, and is a function of the net solar radiation, air temperature, wind speed, and vapor pressure in a particular location. ET rates for a specific crop in a specific location are calculated on a monthly basis by the following equation:

$$ET = ET_o * k_c$$

where:

ET_0 = Normal year reference crop ET rate for a given geographic location (California Department of Water Resources [DWR], California Irrigation Management Information System [CIMIS] database)

k_c = Crop coefficient for a given crop (DWR Leaflets)

For this Project, ET_0 for the CIMIS station closest to the Project site were obtained from the DWR CIMIS database. Crop coefficients for pasture / shrub crops were obtained from a previous project landscape architecture consultant. Calculated ET rates and irrigation demands are shown in **Table 2-6**.

Precipitation: Precipitation data was obtained from the National Oceanic and Atmospheric Administration's (NOAA) online database using the closest station to the Project site. Monthly rainfall values from 1991 through 2020 were averaged to obtain typical monthly rainfall data.

Estimated Unit Irrigation Demands: Typical monthly unit irrigation demands for pasture are summarized in **Table 2-6** and were calculated using the following formula:

$$ID = \frac{(ET - Pe_p)l_r}{e_i}$$

where:

ID = Irrigation demand in inches

ET = Evapotranspiration for turf grasses

P = Average precipitation, NOAA

e_p = Precipitation irrigation efficiency, 0.95. This assumes that approximately 0.5% of rainfall during growing season is lost to evaporation, runoff, etc.

l_r = Loss rate, 1.05. This assumes that approximately 5% of the applied water passes through the grass root zone and is lost.

e_i = Irrigation efficiency, varies throughout the year between 0.60 in the summer and 0.95 in the winter. This assumes that 5-40% of the applied irrigation water is lost to the environment. For planning purposes an irrigation efficiency of 0.80 was used.

Table 2-6: Typical Irrigation Demands for Regional Pasture

Month	ET (inches)	P (inches)	ID (inches)	ID (feet)
January	0.87	5.27	0.00	0.00
February	1.46	4.88	0.00	0.00
March	2.18	3.20	0.00	0.00
April	2.69	1.17	1.53	0.17
May	3.85	0.74	3.11	0.34
June	4.65	0.21	4.44	0.49
July	4.60	0.00	4.60	0.50
August	4.30	0.06	4.24	0.46
September	3.35	0.12	3.23	0.35
October	2.46	0.96	1.50	0.16
November	1.18	2.32	0.00	0.00
December	0.62	5.39	0.00	0.00
Total	**32.20**	**24.32**	**22.64**	**2.48**

Notes:
1. The irrigation demand shown is for average rainfall. A lower irrigation demand was used in the 100-year annual precipitation event.

As shown, above, in **Table 2-6**, the typical annual unit irrigation demand for pasture is estimated at 22.64 inches or 2.48 feet.

The irrigated areas are limited by the proposed Project site plans, topography, and site infiltration capacity. These conditions can contribute to run-off which must be carefully managed when using recycled water. An infiltration study was performed for the Project site in April 2024 which found very low infiltration soil capacities at the site; those results are included in **Appendix B**.

2.3 Water Supply Requirements

There are no existing water demands for the proposed project site. **Table 2-7** compares the projected average annual demands for Alternatives A, B, and C.

Table 2-7: Comparison of Alternative Water Demands

Program Alternative	Average Annual Demand (AFY)	Average Daily Demand (gpd)[1]
Alternative A	289	258,000
Alternative B	280	250,000
Alternative C	83	74,000

Notes:
1. This demand represents indoor water use.

The experience of other similarly sized gaming and entertainment facilities has shown that water demands can be significantly reduced when recycled water is introduced as an alternative water supply source. Although the availability of recycled water has not yet been determined, water supply requirements including the use of recycled water were calculated considering recycled water for toilet flushing, landscape irrigation, cooling tower make-up and other approved non-potable uses under Title 22 regulations. Although it doesn't apply to uses on Trust lands, the recycled water quality would be designed to produce the equivalent water quality to disinfected tertiary recycled water as defined by Title 22.

The average water demand for Alternatives A, B, and C is shown in **Table 2-8**. These projections are based on estimated average wastewater flows (see **Table 2-2** through **Table 2-4**) and include a 20% allowance for system losses as well as a safety factor to ensure adequate supply. Also provided in this table is the projected water demand assuming that recycled water is produced on-site and available to the project. The average water demand is expected to be representative of typical daily water use. Peak water demands, which would typically occur on the weekends, were calculated assuming a peaking factor of 1.5.

Table 2-8: Projected Water Demands for Alternative A, B & C

Program Alternative	Parameter	Projected Water Demands (gpd)[1]	Projected Water Demands with Recycled Water (gpd)[1]
Alternative A	Average Daily Demand	258,000	205,000
	Peak Day Demand	387,000	334,000
Alternative B	Average Daily Demand	250,000	197,000
	Peak Day Demand	374,000	321,000
Alternative C	Average Daily Demand	74,000	62,000
	Peak Day Demand	111,000	99,000

Notes:
1. Assumes augmenting indoor potable use with recycled water use for dual plumbed and cooling purposes.

Preliminary projections of the water supply needed to reliably meet water demand for the programs are summarized in **Table 2-9**. These are preliminary and for planning purposes only.

Table 2-9: Projected Water Supply Design Flows

Program Alternative	Water Supply Requirement without Recycled Water (gpm)	Water Supply Requirement with Recycled Water (gpm)	Minimum Recommended Firm Water Supply (gpm)
Alternative A	300	250	300
Alternative B	300	250	300
Alternative C	100	100	100

Notes:
1. Units of gpm = gallons per minute. All flows rounded to the nearest increment of 50 gpm.
2. Water supply required for Alternative A versus Alternative B is similar due to negligible demands from housing community compared to anticipated Casino demands.

A "firm" water source is considered that which can be supplied by the system with the single largest source out of service in a redundant system. The "firm" water supply is required 24 hours a day, 365-day a year, and must be able to meet the maximum day demand for the Project.

Water system redundancy may be achieved in a variety of ways – in a groundwater system, multiple wells or another redundant source is typically required. Diurnal peaks, fire flow, and other peak demands may be met with storage tanks.

In addition to the use of recycled water, the project alternatives are also expected to be designed and managed to minimize potable water usage. Recommended water conservation measures include low flow fixtures, voluntary towel re-use, central plant optimization, recirculating fountains or water features, if applicable, high efficiency/water conserving appliances, etc. For restaurants, potable water can also be conserved, if only served to patrons who request it. To facilitate this, sub-metering of water for each of the uses within the Project will discourage waste and help identify areas where consumption can be reduced. Employee training and participation, regular maintenance, and customer education are all expected to help reduce water use.

Fire flow requirements (or guidelines) are set by the local fire authorities based on the building's use and classification. Storage requirements for casinos are generally controlled by fire protection requirements and not by domestic peaking requirements. Storage needs will be determined upon issuance of the fire flow and duration requirements from the local fire authority. Referencing the City's Water Master Plan from 2015, the expected fire flow requirements for a large facility such as this will be 4,000 gpm for four hours.

2.3.1 Water Supply

The Project will require a potable water supply for use within the site. Currently, there are no groundwater wells identified on the site or within a half mile radius. A hydrogeological assessment – included as **Appendix C** – was conducted in May 2024 to identify the existing sources of groundwater for the site. The results of the assessment determined that the potential yield of a new well on site is uncertain, seasonal fluctuation affect output of on-site springs, colluvium and alluvium is present on site and variable and may affect yield conditions negatively, and historical mercury mining operations were present near the site which may contaminate any groundwater through the site. Irrigation water could be provided either by reuse of effluent from the proposed on-site wastewater treatment plant (WWTP) as recycled water or by potable water.

For any on-site groundwater well, it is likely that treatment will be required to remove heavy metals based on historical mining activities in the region. A well pump test would also need to be conducted to determine the available pumping capacity and safe pumping yield of the groundwater basin. The number of wells required would be dependent on the capacity of each new groundwater well. At a minimum, sufficient capacity would be required to meet the maximum day demand with the largest source out of service. If a groundwater supply is pursued, the anticipated well capacity, location and operating strategy would be developed further during the testing and design phase.

Due to the uncertainty of the groundwater yield and possible contamination of the groundwater supply, the number of wells and type of treatment are not known. Assumptions have been made for planning purposes and are further discussed in **Section 4.1**.

SECTION 3 – BACKGROUND AND REGULATORY ISSUES

This section identifies the typical regulatory requirements applicable to the Project with respect to the proposed water supply, wastewater treatment, and wastewater discharge methods identified in this report.

3.1 Water Supply

Two options are considered for water supply: on-site groundwater wells or a municipal connection to the City's water system.

3.1.1 Local Hydrogeologic Assessment

As discussed in **Section 2.3.1**, a hydrogeologic assessment (**Appendix C**) was prepared by Engeo to assess the existing sources of groundwater at the Project site. In general, the following conclusions were presented:

- Groundwater supply wells were not located on the Project site or nearby. Previous well pump tests were not conducted on the Project site. The potential yield of the site's soil materials is uncertain.
- The output from the springs is not known although seasonal fluctuation and drought periods will result in reduced spring flow.
- Depths of colluvium and alluvium at the site were variable. Colluvium contains high concentrations of clay which may result in low yield conditions.
- Historical mercury mining operations were present at multiple locations near the site, including St. John's Mine located less than 1 mile northeast of the site. Groundwater contamination with heavy metals is probable due to these operations or from flow through rocks containing heavy metals.

Any groundwater supply used to serve the project must meet all USEPA water quality standards.

3.1.2 City of Vallejo Municipal Connection

In this case, regulatory requirements for water supply for the Project would be met by the City and it is anticipated that the on-site water storage, supply, and distribution facilities would be constructed by the Tribe and adhere to City standards and requirements, a copy of which is included as **Appendix D.**

Initial review of the City's water distribution system according to the 2015 Water Master Plan indicates that there is adequate system capacity both during maximum day demand, maximum day demand plus fire flow, and peak hour demand conditions. The Project site is located within the City's 292 Zone which has up to 12 MG of storage capacity with the Skyview Tank (currently inactive) and is identified as "Planned Development Commercial." This zone is served by the elevation head in the Columbus Parkway Tank.

Water Supply Reliability

The Project site is identified as a combination of “Business/Limited Residential” and “Parks, Recreation and Open Space” in the General Plan, which is cited as the basis for projecting future water demands in the City’s UWMP. It is assumed that the basis for the water demand does not capture the level of development proposed for this project. Thus, in reviewing the water supply reliability, it is assumed that the Project water demands would be in addition to the City’s projected demands, to be conservative.

According to the City’s 2020 Urban Water Management Plan (UWMP), there is adequate supply during all years including, normal, single-dry, and multiple consecutive dry years. There are no shortfalls.

Table 3-1 and **Table 3-2** shows the normal year and single dry year supplies and demands in 5-year timesteps from 2025 through 2045.

Table 3-1 Normal Year Water Supply and Demand through 2045 (AFY, UWMP Table 5-2)

Normal Year	2025	2030	2035	2040	2045
Supply	35,820	35,823	35,825	38,778	38,780
Demand	28,111	29,153	30,331	31,888	31,892
Difference	7,709	6,670	5,494	6,890	6,888

Table 3-2 Single-Dry Year Water Supply and Demand through 2045 (AFY, UWMP Table 5-2)

Single Dry Year	2025	2030	2035	2040	2045
Supply	31,585	31,588	31,590	33,093	33,095
Demand	29,113	30,207	31,443	33,079	33,083
Difference	2,472	1,381	147	14	12

During a normal year the City projects a minimum excess supply of 5,494 AFY in 2035, the lowest net difference under normal conditions. In a single dry year, the net supply is reduced to 12 AFY by 2045.

Table 3-3 shows the anticipated annual water supply and demand conditions for the City’s service area in five consecutive dry years from 2025 through 2045. Under this analysis, no water conservation has been assumed and mild increases have been incorporated from year to year out of an abundance of caution, per the UWMP. Under all five-year drought conditions through 2045 the City is projecting adequate supply to meet demand with no shortfalls. During the most conservative scenario by year 2039 the difference between supply and demand decreases to 21 AFY.

Table 3-3 Multi-Dry Year Water Supply and Demand through 2045 (AFY, UWMP Table 5-3)

		2025	2030	2035	2040	2045
Year 1	Supply	33,526	33,529	33,531	35,034	35,036
	Demand	29,113	30,207	31,443	33,079	33,083
	Difference	4,413	3,322	2,088	1,955	1,953
Year 2	Supply	32,592	32,595	32,597	34,100	34,102
	Demand	29,263	30,357	31,543	33,080	33,083
	Difference	3,329	2,238	1,054	1,020	1,019
Year 3	Supply	31,667	31,670	31,672	33,175	33,177
	Demand	29,413	30,507	31,643	33,081	33,083
	Difference	2,254	1,163	29	94	94
Year 4	Supply	31,769	31,772	31,774	33,277	33,279
	Demand	29,563	30,657	31,743	33,082	33,083
	Difference	2,206	1,115	31	195	196
Year 5	Supply	31,859	31,862	31,864	33,367	33,369
	Demand	29,713	30,807	31,843	33,083	33,083
	Difference	2,146	1,055	21	284	286

The highest Project demand alternative (Alternative A) is 322 AFY, which would result in a deficit of 310 AFY in a single dry year and a deficit of 301 AFY by 2039 in a multi-dry year condition.

Under drought conditions, the City would implement actions in accordance with the City's Water Shortage Contingency Plan. Stage 1 (Water Alert) is intended to induce 10% conservation to match a 10% shortage condition. This stage would be voluntary and would precede any mandated reductions. Stage 2 (Moderate Shortage) would be implemented if Stage 1 restrictions are deemed insufficient or if State mandated reductions require further conservation. This would be implemented under supply shortage conditions from 10-20%. A Stage 1 action would induce potentially over 3,100 AFY of water conservation in either dry year condition. The estimated deficits of up to 309 AFY are well within the 10% margin and is equivalent to less than 1% of total demand.

3.2 Wastewater Handling

Two options are considered for wastewater handling: a connection to the District's collection system or onsite wastewater treatment.

3.2.1 Vallejo Flood and Wastewater District Connection

For this option, the Project collection system would adhere to the District's Engineering Standards (included as **Appendix E**). The District is responsible for meeting all State Water Resources Control Board (SWRCB) standards and requirements related to sewer system management, wastewater treatment, and disposal/discharge.

The District recently finalized their Sanitary Sewer Collection System Master Plan (Sewer Master Plan) in August 2023. There are many existing identified deficiencies throughout the collection system including areas within the system where the District experiences sanitary sewer overflows (SSOs) during wet weather events. The most notable issues identified in the Sewer Master Plan are related to the Sears Point Pump Station and Tank; this facility is the largest pump station conveying peak flows of 23 MGD with 3.2 MG of storage capacity. The Sewer Master Plan explores increasing pipeline capacity to accommodate peak wet weather flows and/or rehabilitation and replacement in subbasins where inflow and infiltration (I&I) are excessive. The District has invested, and continues to invest, millions of dollars to recapture collection system capacity. The Sewer Master Plan acknowledges that while future flows do not create the need for additional improvements the District is working with developments to contribute to mitigation funding.

The Sewer Master Plan does not evaluate WWTP capacity and deficiencies. The Vallejo WWTP is located at 450 Ryder Street, adjacent to the Mare Island Strait. There is no noted deficiency at the District's WWTP to treat average dry weather flow (ADWF). The WWTP's dry weather capacity is 15.5 MGD and it treats an ADWF of approximately 8 MGD using primary sedimentation, trickling filters, short-term aeration, and sodium hypochlorite for disinfection. Solids undergo lime stabilization, gravity thickening, and dewatering by belt filter press prior to land application. Screenings and grit are transported to a landfill for disposal. Its peak wet weather capacity for secondary treatment is 35 MGD. The WWTP has an additional 25 MGD primary treatment capacity. Thus the peak wet weather discharge capacity of the blended primary and secondary effluent is 60 MGD. According to the District's National Pollution Discharge Elimination System (NPDES) permit, the daily wet weather flow from November 2017 through April 2021 ranged from 3.9 to 60 MGD, up to the wet weather capacity limit.

The permit specifically requires the implementation of actions that will reduce blending at the WWTP. The actions outlined include projects to reduce I&I and peak wet weather flow. These actions to be implemented within the timeframe of the permit (thru March 31, 2028) include:

- Report Annually on Implementation of 10-Year Sanitary Sewer Capital Improvement Plan
- Report Annually on Implementation of Storm Drain Master Plan
- Report Annually on Reducing Inflow and Infiltration Due To Flooding
- Continue Collection System Rehabilitation and Replacement
- Continue Implementation of Asset Management Program
- Continue Updating 10-Year CIP
- Complete Treatment Plant Master Plan
- Implement and Revise Ryder Street Storage Basin Standard Operating Procedures
- Continue and Expand Upper Lateral Program
- Complete Mare Island Pump Station Replacement and Rehabilitation
- Complete North Secondary Clarifier Rehabilitation
- Develop Private Sewer Lateral Inspection Ordinance

3.2.2 On-Site Wastewater Treatment

Any new on-site WWTP will be located on Trust lands, constructed by the Tribe, and subject to federal standards and regulation. The USEPA does not require or restrict type of onsite reuse of treated wastewater. The tribe will ensure the protection of any onsite drinking water sources and prevent runoff of recycled water into waterbodies.

The WWTP will be designed to comply with the effluent quality requirements for tertiary level recycled water for unrestricted reuse to allow for use both onsite and offsite. The MBR process, which is discussed later in **Section 5.2** is capable of meeting these requirements with minimal modifications.

Nitrogen removal will be achieved in the anoxic basin of the MBR process as discussed in **Section 5.2.4**. If phosphorus removal is required, the MBR process is well suited to provide for phosphorous removal to very low concentrations. Phosphorus removal is enhanced in MBR treatment plants by employing one or multiple of the following operational methods: 1) addition of a coagulant to the aeration basin, 2) a higher solids retention time in the MBR basins, 3) ensuring there is an ample carbon source for the microorganisms, and 4) utilization of a membrane which virtually eliminates any particulate phosphorus in the effluent. The method(s) the Tribe will employ for phosphorus removal will be determined during the WWTP design phase depending upon targeted end use.

3.2.3 Sludge Disposal

Sludge (biosolids) produced by the WWTP must also be disposed of in accordance with the California Code of Regulations, Water Code, Resource Conservation and Recovery Act, and the RWQCB policy. These regulations are commonly referred to as the 40 CFR Part 503 Biosolids Rule promulgated by the USEPA. It is anticipated that biosolids produced by the Project WWTP will be disposed of to an off-site landfill in accordance with all regulatory requirements. Prior to off-site disposal, biosolids will be dewatered. The dewatered sludge, also known as cake, would be periodically hauled to a Class III landfill for disposal. The frequency and volume of dewatered sludge is typically determined during the design phase of the project as more data is available on the source water quality and treatment process.

3.2.4 Cooling Tower Brine Generation and Disposal

The flow rate and water quality of brine generation from cooling tower processes is unknown. It will ultimately depend on the water chemistry of the makeup water, type/model of the cooling system and operation of the cooling system. Disposal sources for brine generation from cooling processes generally include off-site disposal or discharge to one or more of the following: receiving municipal utility district, surface water bodies, sewer system, ocean outfall, deep well injection, incineration, and/or environmental service providers. If disposal to the WWTP is the preferred option, further evaluation will be required to determine the maximum limits of constituents of concern, expected brine flow rates, expected water quality monitoring parameters, cycles of concentration, etc. Further evaluation will be needed to determine the brine generation volume and most cost-effective disposal alternative. Similarly for the brine generated from the recycled water treatment process (see **Section 2.2.3**).

3.3 Recycled Water

For any alternative involving on-site treatment, it is expected that the WWTP will produce recycled water for on-site reuse, which will add to the water quality requirements of the effluent from the WWTP. In order to reuse recycled water on non-trust land in California, a Title 22 reclamation permit would be required. The RWQCB typically issues this permit in California. However, on trust land, the USEPA does not require or restrict the type of reuse. Indian Health Service would regulate the use of recycled water on trust lands. For the range of uses considered for this project, it is expected that the WWTP would need to produce disinfected tertiary recycled water in accordance with Title 22 requirements. Disinfected tertiary recycled water meets the following water quality requirements, which are specific to the MBR treatment process expected for the Project's wastewater treatment facility:

- Has been passed through a microfiltration, ultrafiltration, nanofiltration, or reverse osmosis membrane so that the turbidity of the filtered wastewater does not exceed any of the following:
    - 0.2 NTU more than 95 percent of the time within a 24-hour period; and
    - 0.5 NTU at any time.
- The filtered wastewater has been disinfected by either:
    - A chlorine disinfection process following filtration that provides a CT (the product of total chlorine residual and modal contact time measured at the same point) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow; or
    - A disinfection process that, when combined with the filtration process, has been demonstrated to inactivate and/or remove 99.999 percent of the plaque forming units of F-specific bacteriophage MS2, or polio virus in the wastewater. A virus that is at least as resistant to disinfection as polio virus may be used for purposes of the demonstration. The median concentration of total coliform bacteria measured in the disinfected effluent does not exceed an MPN of 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed and the number of total coliform bacteria does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30 day period. No sample shall exceed an MPN of 240 total coliform bacteria per 100 milliliters.

In addition to the aforementioned recycled water quality requirements, there are a number of operational, use, and reporting restrictions identified in Title 22. However, it is not expected that any of these requirements will limit the viability of recycled water reuse on-site, and these requirements are typical for any recycled water use application. All uses of recycled water would have to be approved by USEPA. As long as disinfected tertiary recycled water is produced, there would appear to be no issues associated with this intended use.

SECTION 4 – WATER FACILITY REQUIREMENTS

Two water supply options are considered to serve the Project. The first option is via onsite groundwater and the second option is thru a municipal connection. Both are described below.

4.1 Groundwater Supply

The hydrogeologic assessment did not identify any groundwater wells within a half mile vicinity of the project site and no history of pump tests on or near the site were available to speak to the availability of groundwater, thus the potential yield is currently unknown. While the available capacity is not known, potential facilities are described herein. Water supply facilities described in this section are preliminary and should be utilized for planning purposes only.

4.1.1 Water Production Wells

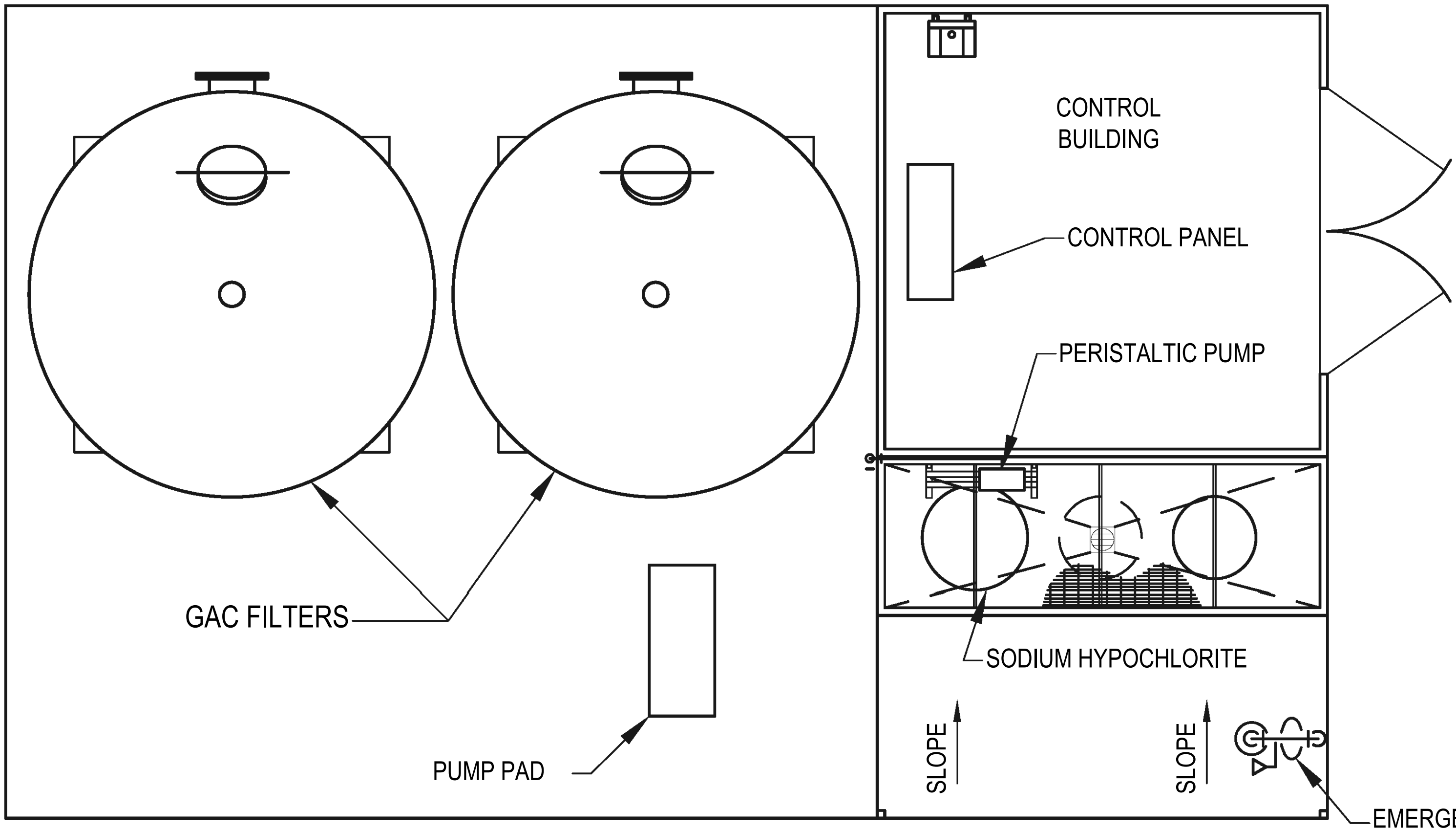
The potable water supply system must have a firm reliable supply based on projected water demands. Firm capacity is the remaining water supply capacity with the largest single source out of service. In a well system, it is generally recommended to have a minimum of two wells available for service, so one can be serviced without interrupting the water supply. It is noted that two or more groundwater wells may be required to serve the development depending on the available capacity of each, which is currently unknown. The actual well capacity, location, and operating strategy would be developed during the design phase.

Based on the hydrogeologic assessment, the local groundwater conditions are characterized as fractured bedrock. A deep test hole would be drilled to determine water bearing capacity within the Great Valley Sequence and silica-carbonate rock. Per DWR, the new well will require a minimum radius of 50-ft control zone around the well, to protect the source from vandalism, tampering, and other possible sources of contamination. As noted previously, the hydrogeologic assessment documented historical mercury mining operations near the Project site, one of which is located within one mile of the Project site. Thus there is a likelihood that groundwater will contain heavy metals. The implementation of water treatment to remove mercury, will likely be required to treat the well water.

The number of wells required is not currently known. Each well is expected to have an approximate footprint of 20 feet by 30 feet, including the pump, well, piping, and miscellaneous equipment. Each well would also be setback from any recycled water use area or impoundment as required by Title 22 criteria.

4.1.2 Water Treatment Plant

The USEPA has identified four technologies for treatment of mercury including precipitation, lime softening, media adsorption processes using granular activated carbon (GAC), and membrane filtration using reverse osmosis. Media adsorption using GAC is an effective method of removing a wide range of constituents and is assumed here for planning purposes. Water quality testing will be required to confirm the appropriate treatment methods. It is assumed that two treatment vessels would be installed in series. A typical layout of the treatment plant is shown in **Figure 4-1**. A process flow diagram showing how water is treated within the treatment plant is shown as **Figure 4-2**.



CONTROL BUILDING SPECIFICATIONS:

10' x 10' x (LOW SIDE) W 8' x 10' CANOPY -SHED STYLE 1:12 SLOPE

FRAME: 2" x 2" x 120 STEEL TUBE ALL WELDED WALL SECTIONS TO BE BOLTED IN FIELD

ROOF/SIDING: McELROY "MULTI-RIB" 28 GA RIBBED METAL - PAINTED DOOR: 6' x 6'-8"

DOUBLE SWING STEELCRAFT F-16 SERIES FRAME W/L18 SERIES DOOR W/POLYSTYRENE CORE

LOCKSET: D SERIES W/ORBIT STYLE KNOB AND ENTRANCE LOCK

VENTS: 2 EACH 12" x 12" (NOMINAL)

SITE PLAN

SCALE: 1" = 50'-0"

Figure 4-1
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Preliminary Water Treatment Plant Layout

HydroScience

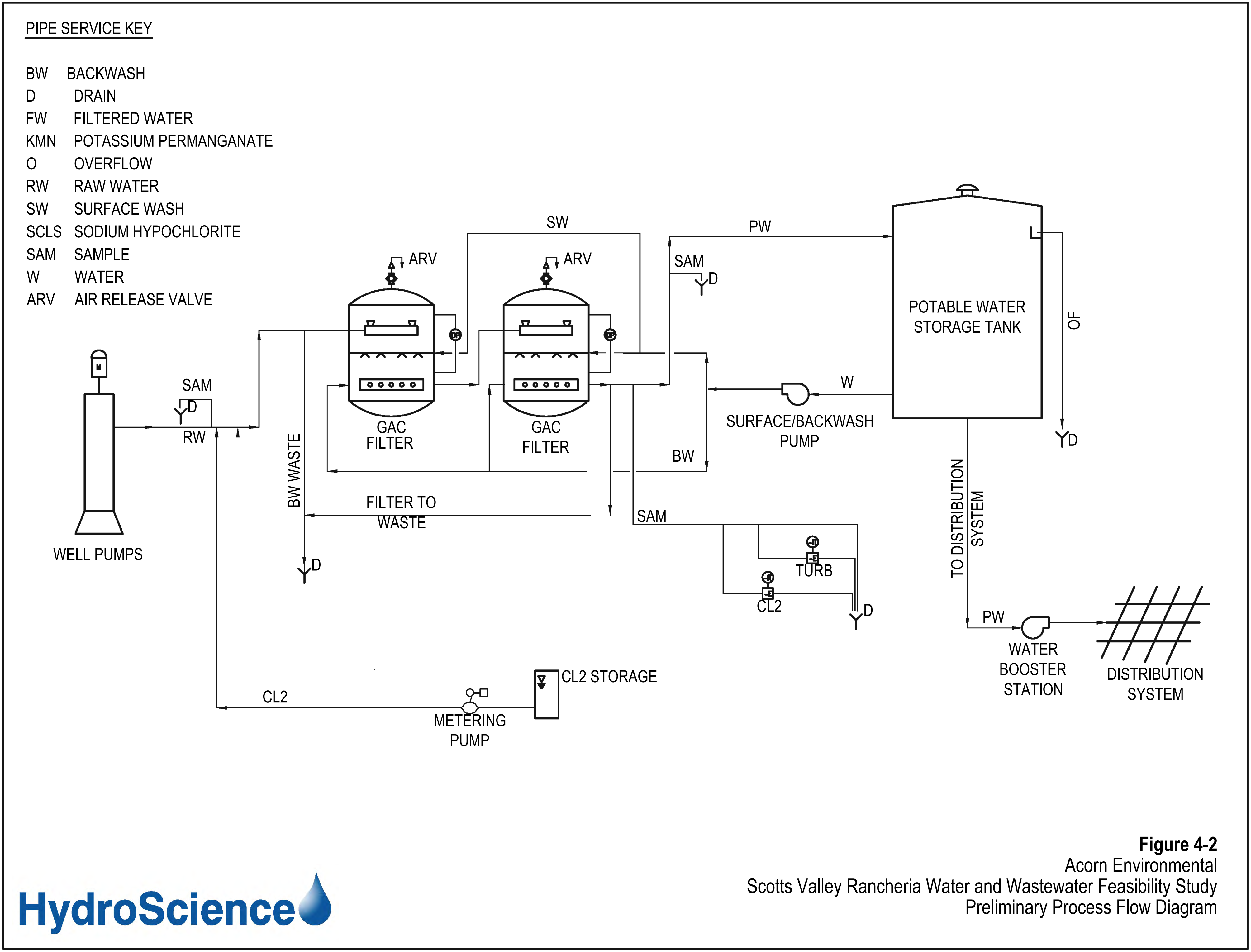


Figure 4-2
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Preliminary Process Flow Diagram

Mercury is removed with simple on/off cycling and infrequent backwashing is required. Gentle breakthrough curve allows for reduced sampling frequency. Pilot testing is required to determine adsorption capacity. Efficiency is subject to competing adsorption by non-target compounds. Sodium hypochlorite would be used to disinfect the water before on-site distribution. A continuous monitoring residual analyzer will monitor chlorine residual at the end of the filters, before entering a water storage tank. Chlorine dosage control would be manual, with options for automatic pacing based on residual. The WTP process facilities would be located within an enclosed building.

Significant features of the plant would include:

- PLC control system interlinked to a common water/wastewater SCADA system.
- Surface wash to reduce the possibility of "mudball" formation on the media surface.
- Fail-safe control valves that would fail in the filter-forward mode of operation.

The recommended WTP design criteria are summarized in **Table 4-1.**

Table 4-1: Recommended Water Treatment Plant Design Criteria

Parameter	Value
Process	Pressure filtration
Media for Adsorption	GAC
Number of filters	2
Filter loading rate	3 gpm/sf
Filter size	10 ft diameter
Disinfection	Sodium Hypochlorite
Process control	PLC/on with service well

Filter media and size may vary based on water quality and Project Alternative water demands. Storage facilities are described in **Section 4.3**.

4.2 Municipal Connection

The second option for Project water supply is connecting to the nearest City of Vallejo municipal water system. There is an existing 6 MG capacity tank located adjacent to the Project site, identified as the Columbus Parkway Tank owned by the City, as well as an easement traversing the Project site for the City's 24-inch transmission main. A 24-inch transmission main also extends south from the tank to Columbus Parkway.

Initial communication with the City indicates that there is likely adequate storage and flow capacity to serve the Project; however, adequate pressure is not available and would need to be provided by on-site infrastructure. Further coordination with the City is expected to confirm the needed infrastructure to connect to the City's distribution system and confirm design capacity.

The following section identifies preliminary water storage, and pumping requirements to supply the proposed Project with potable water. The general concept for the water supply facilities is that the Project will include storage and pumping on-site to meet the needs of the Project with water supplied by the City. Having storage onsite will help to mitigate hydraulic impacts to the City's facilities by allowing the tank to be filled during off-peak periods so as not to affect the City's peak hour hydraulic conditions. Sizing onsite storage for maximum day plus four hours of fire flow would also mitigate any impact to the City's system during a fire emergency. All new water storage, supply, and distribution facilities would be designed to comply with City standards (**Appendix D**).

The ultimate location of the water facilities and connection to City infrastructure will be based on coordination with the City and the final design of the Project facilities. All of the recommended water supply facilities described in this section are preliminary and should be utilized for planning purposes only.

4.3 Water Storage Tank and Pump Station

A storage tank would be constructed to store water provided either by the onsite WTP or by the City. For this assessment it is assumed that the storage tank will be designed for maximum day demand plus four hours of fire flow at 4,000 gpm. For the municipal connection option, it is possible that fire flow can be provided with dedicated pumping capacity directly from the City's transmission main allowing fire flow storage to be met by the City's Columbus Parkway Tank.

The storage tank would be of welded steel construction meeting all American Water Works Association (AWWA) specifications for welded steel tanks. A typical section of a tank is shown in **Figure 4-3**. The tank would be a cylindrical shape, and the tank sizing would be based on standard pre-engineered tank dimensions, which are typically in 8-foot increments. It is assumed that the tank would be located at grade. **Table 4-2** provides recommended tank volumes and dimensions for each Alternative.

Table 4-2: Water Storage Tank Capacity and Dimensions

Project Alternative	Max Day Demand[1] (gal)	Fire Flow (gal)	Nominal Tank Volume[2] (MG)	Height (ft)	Diameter (ft)
Alternative A	431,000	960,000	1.5	40	80
Alternative B	417,000	960,000	1.5	40	80
Alternative C	110,000	960,000	1.2	32	80

Notes:
1. See **Table 2-8** for peak day demand. For planning purposes the tanks are sized assuming no recycled water use.
2. Exact volume is to be determined during the design phase of the project.

Proposed siting of a potable water storage tank is provided in **Figure 5-11** and **Figure 5-12**. For a municipal connection, the water storage tank may be filled with the elevation head from the City's tank assuming the top of the Project's potable water storage tank is at, or below, the base elevation of the Columbus Parkway Tank which is 257 ft.

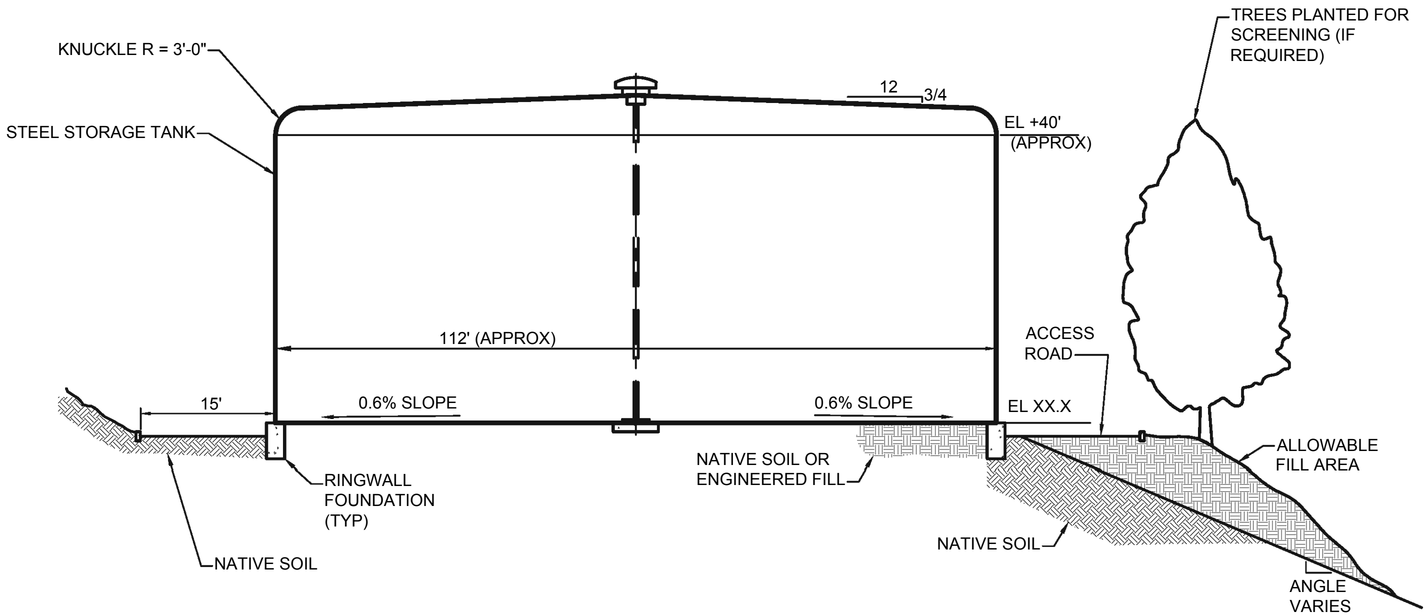
Due to the topography, a pump station would be necessary to pump water from the storage tank to the distribution system. This potable water pump station will be required to convey water from the storage tank to the facilities requiring potable water and would be sized to handle both fire flow and domestic demands. The ultimate pumping capacity will be dependent on fire flow requirements and would be satisfied by two variable-speed high-service pumps that are half the capacity of the projected flow requirement. The pump station would provide enough total dynamic head to serve the highest elevation user at least 40 psi of pressure. High pressures in the lower elevations can be mitigated with pressure reducing valves (PRVs) to create pressure zones with operating ranges between 50 psi and 80 psi. A hydropneumatic tank can sustain pressure and minimize pump starts and stops. **Table 4-3** shows the recommended design criteria for the pump station.

Table 4-3: Pump Station Design Criteria

Parameter	Value
Minimum number of low service pumps	2
Pump type	Variable speed turbine
Minimum number of high service pumps	2
Hydropneumatic tank approximate volume range[1]	1,500 – 2,500 gallons

Notes:
1. Exact volume is to be determined during the design phase of the project. Tank volume is dependent on the desired flowrate and pressure from the hydropneumatic tank.

Proposed locations for the water treatment and storage facilities for each alternative are shown at the end of **SECTION 5** in **Figure 5-11** and **Figure 5-12**.



HydroScience

Figure 4-3
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Typical Water Storage Tank

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SECTION 5 – WASTEWATER FACILITY REQUIREMENTS

This section identifies feasible preliminary options for wastewater collection, treatment, effluent discharge, and recycled water facilities required to manage wastewater generated by the proposed Project.

The general concepts for the wastewater facilities are to develop an on-site collection system and connection to the VFWD collection system or provide on-site treatment with a combination of on-site and off-site recycled water use. The intent is to comply with all applicable permitting requirements discussed in **Section 3.2** and ensure that any wastewater or recycled water facilities are designed in a manner that does not limit existing uses or future expansion. This section describes the following facilities:

- VFWD Connection
- On-Site WWTP
    - Discharge Facilities
    - Operations and Maintenance
- Recycled Water

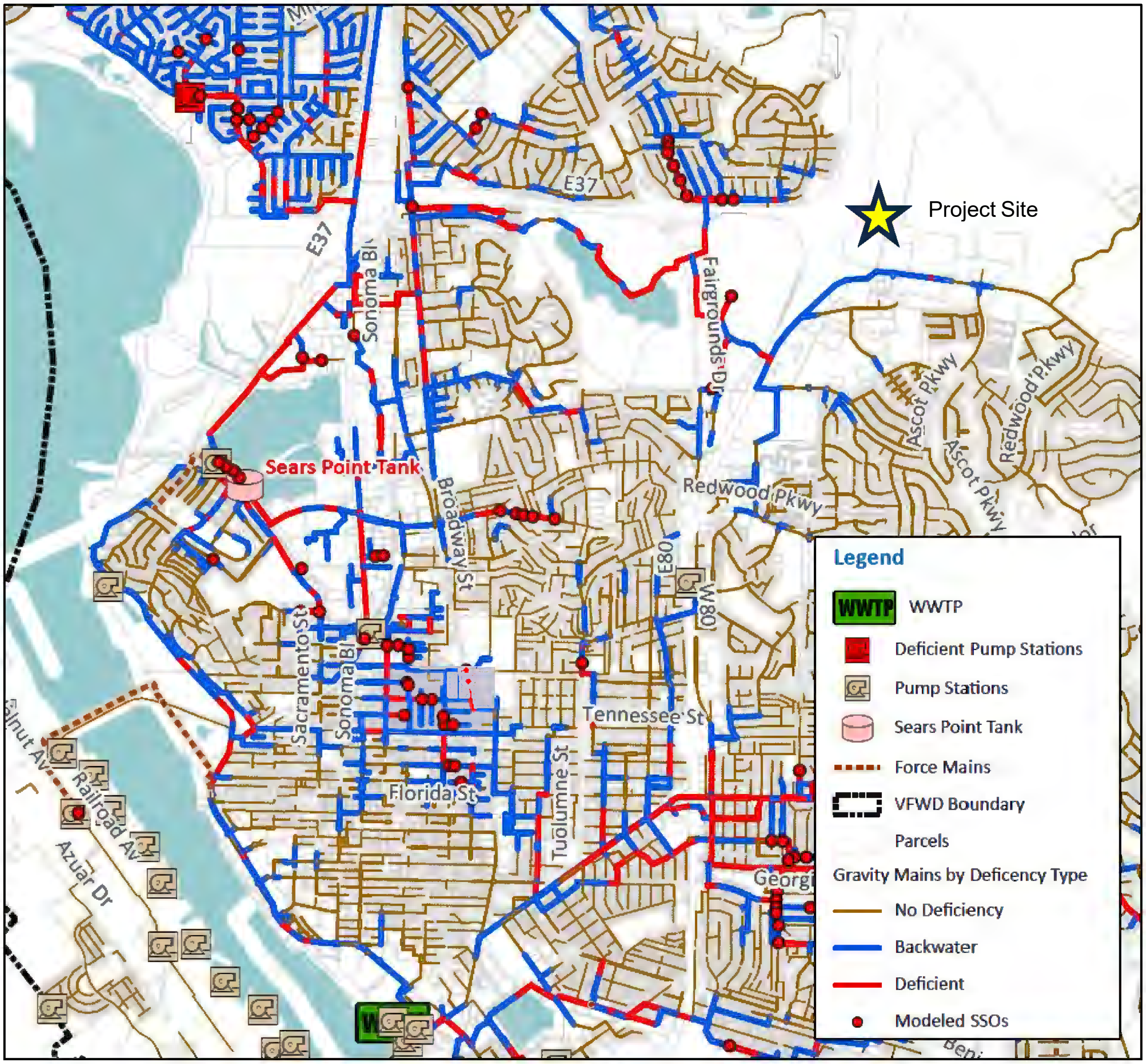
Wastewater from the casino/residential facilities would be conveyed via an on-site gravity sewer collection system. Sewer pipelines would likely be laid along planned roadways within the parcel to facilitate future maintenance. Due to the topography, it is expected that wastewater would flow by gravity to the point of connection to the VFWD system or to a lift station where it would then be pumped to the WWTP headworks.

The ultimate location of the wastewater facilities will be based on the final design of the Project facilities and the chosen method of wastewater disposal. All of the recommended wastewater facilities described in this section are preliminary and should be utilized for planning purposes only.

5.1 VFWD Sewer Connection

There is an existing 12-inch pipeline in Columbus Parkway that currently serves a smaller tributary area to the east along with the Hiddenbrooke development. This area was developed in the 1970s, 1980s, and 1990s and there are no identified deficiencies at, or immediately downstream of, the point of connection to the 12-inch pipeline; however, it is noted that downstream in the collection system there are deficiencies during the design storm causing backwater effects in the 12-inch pipeline along Columbus Parkway, see **Figure 5-1**. Backwater in the pipeline is a result of deficiencies and bottlenecks downstream of the point of connection.

Figure 5-1: Existing System Deficiencies

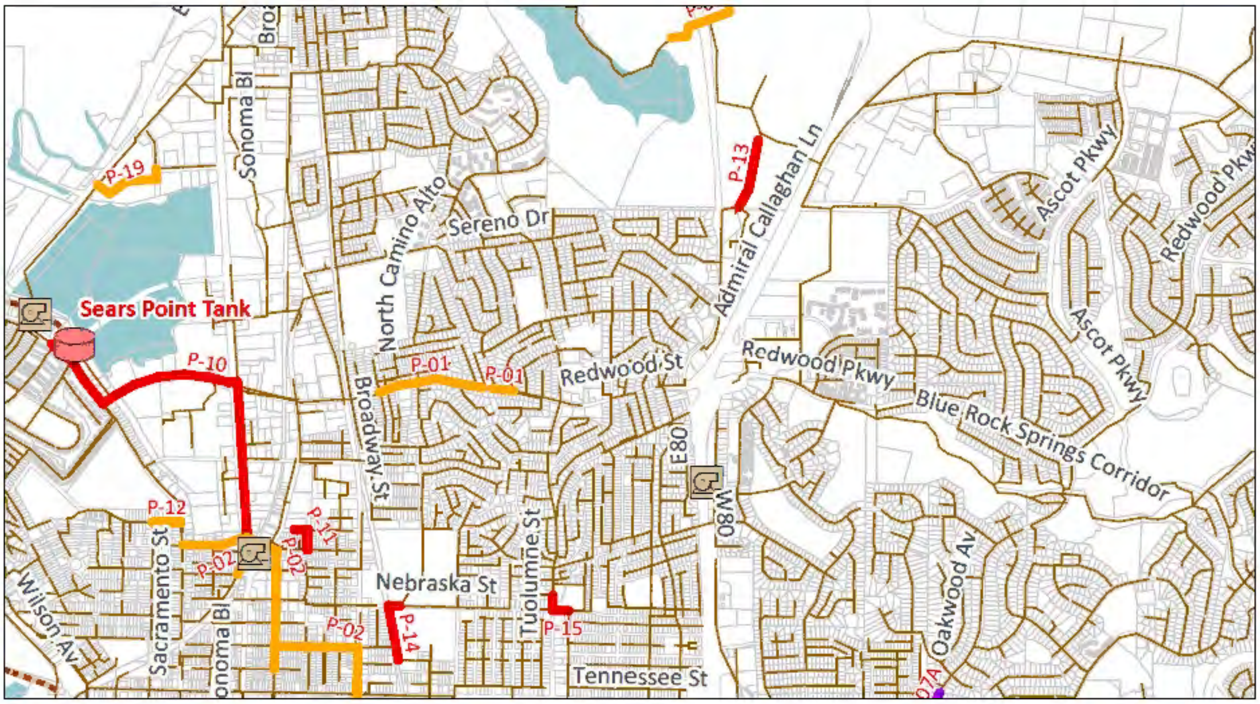


Source: Sanitary Sewer Collection System Master Plan, VFWD, August 2023, Figure 6.21

One of the more significant relevant deficiencies is located at the Sears Point Storage Tank, which is noted to exceed capacity during design storm simulations. Historically, the storage tank has approached capacity during lesser storm events. The District has invested, and continues to invest, millions of dollars to address I/I issues in the collection system to free up collection system capacity.

The District is implementing a number of improvement projects to address system capacity. There is a proposed pipeline upsize as shown in **Figure 5-2** as project *P-13*; while the Sewer Master Plan does not explicitly identify this project to alleviate the backwater effects in Columbus Parkway, it does appear to be one of the bottleneck located downstream of the point of connection likely contributing to the backwater effects in Columbus Parkway. Other projects located further downstream (i.e. *P-01* and *P-10*) may further alleviate the backwater issue. The District is also implementing general system capacity improvement projects to alleviate the impact to the Sears Point Tank and Pump Station.

Figure 5-2: VFWD Planned CIP



Source: Sanitary Sewer Collection System Master Plan, VFWD, August 2023, Figure 8.1 Capital Improvement Plan Phasing

To assess connection feasibility, the District requires applicants to contract with the District to conduct a Sewer Study to assess available capacity of the existing collection system to handle wastewater flow from new developments and identify any potential on-site or off-site impacts. Guidelines for the Study are detailed in the District's 2020 Engineering Standards included as **Appendix E**. The District consultant conducts these analyses. The Tribe would coordinate an agreement with the District to execute this analysis. The Sewer Master Plan acknowledges that while new development flows do not necessarily create the need for additional improvements, the Tribe can anticipate negotiating with the District to contribute to mitigation funding.

5.2 Wastewater Treatment Plant

An alternative to a wastewater connection with the District will be to develop an on-site WWTP and pursue opportunities to use recycled water on-site and partner with the City and District to implement recycled water opportunities within the City.

Traditional wastewater treatment options, such as primary clarifiers, activated sludge, conventional filtration, and disinfection, were not considered as WWTP options due to the limited proposed treatment area layout. Any wastewater treatment process selected for use must be able to handle the high strength waste and react well to wide variations in flow. A proposed on-site WWTP treatment process would include:

- Coarse Screening Facility,
- Influent Pump Station,
- Headworks,
- Equalization,
- Packaged Immersed Membrane Bioreactors (MBRs),
- Ultraviolet (UV) Disinfection & Chlorination,
- Sludge Storage and Dewatering Station,
- Plant Drain and Supernatant Return Pump Station,
- Effluent Pump Station, and
- Operations Building.

The MBR treatment process was selected for various reasons, including: 1) the desire for a small footprint for an on-site WWTP, 2) the proven effectiveness of this process at other similar facilities, and 3) the production of high-quality effluent suitable for reuse and discharge. Additional justification for selection of this treatment process is summarized below.

The MBR treatment process is a tertiary treatment process similar to an activated sludge treatment plant, but with membranes immersed in an aeration basin. A typical MBR system consists of an anoxic tank for denitrification of the plant influent, followed by an aeration tank for oxidation of organic matter and nitrification. Membrane cartridges are suspended at the effluent end of the aeration tank. The membranes have a pore size in the sub-micron range and are able to filter out most of the coliform bacteria and solids. Water is drawn through the membranes by blowers, which pull a slight vacuum and force this permeate into the center of the spaghetti-strand shaped membranes. Solids are left in the aeration tank for recirculation to the anoxic zone and/or wasting to solids handling process(es).

This treatment typically results in producing MBR effluent of excellent quality; effluent from these types plants typically contain no suspended solids and have a turbidity of less than 0.2 NTU. The MBR process also provides aeration, nitrification, and denitrification processes within a compressed footprint. These processes have the effect of producing effluent with a neutral pH, lower nitrogen concentrations, and lower phosphorous concentrations than alternative tertiary treatment processes.

The MBR treatment process is capable of producing effluent meeting the Title 22 coliform bacteria effluent requirements without the use of chlorine or other common disinfectants. Other tertiary treatment systems typically require a disinfection process to meet the effluent coliform requirement. However, in order to comply with treatment and water reuse regulations, both UV disinfection and chlorine disinfection processes will be provided downstream of the MBR processes.

Although the MBR treatment process is somewhat sophisticated, it is relatively simple to operate and maintain due to the absence of traditional WWTP components such as clarifier mechanisms or drives. In addition, there is a long history of effectiveness at similarly-sized gaming facilities with discharge permits to land and surface water.

Proposed locations for the wastewater facilities in each alternative are shown at the end of this section in **Figure 5-11** and **Figure 5-12**.

Operation: Typically, wastewater will flow by gravity from the facilities through a grease interceptor, coarse screening facility, and then into an influent pump station. The coarse screening facility would remove larger solids and debris that are typically found in casino/hotel sewage. The influent pump station will lift the wastewater to the plant headworks facilities through a pressurized sewer main. After passing through the headworks, wastewater will flow by gravity to the influent distribution channel. The distribution channel will be used to distribute wastewater to the parallel MBR trains. Each train will be equipped with an anoxic basin and an aeration basin to provide oxidation, nitrification, and denitrification. Water will flow out of the aeration basin and into a membrane chamber that will be shared by both process trains. Permeate will be extracted through the membranes and conveyed to the UV disinfection process followed by chlorine dosing for residual management.

The proposed wastewater flow diagram is shown in **Figure 5-3**. Major components are described in more detail in the following subsections.

5.2.1 Lift Station

Wastewater will be pumped through a sewage transmission pipeline from the lift station to the headworks of the WWTP. It is likely that a duplex wet well sewage lift station with a standby pump will be required to convey sanitary sewage to the WWTP. The lift station wet well will also be used to collect surface water runoff from the treatment site.

Recommended design criteria for the lift station(s) are shown in **Table 5-1**. A figure showing a typical sewage lift station layout is shown in **Figure 5-4**. The station should be designed to lift the maximum daily flow with one pump out of service.

Table 5-1: Recommended Sanitary Sewage Lift Station Design Criteria

Parameter	Value
Purpose	Lift raw wastewater to WWTP facilities
Type	Submersible non-clog centrifugal
Quantity	Two (one duty, one standby)
Controls	Variable speed, level switch start and shutoff

5.2.2 Coarse Screening Facility

The coarse screening facility for the WWTP is typically gravity fed and upstream of the lift station wet well. Due to the sources and quality of the wastewater, it is important to remove large debris to protect the downstream processes, specifically the pumps. Sewage lift station pumps typically handle solids less than 3 inches in diameter. A typical layout for the coarse screening facility is shown in **Figure 5-5**. **Table 5-2** shows some of the design criteria for the coarse screening facility.

Table 5-2: Coarse Screen Design Criteria

Parameter	Value
Coarse screening facilities	Enclosed bar screen, multi-rake style, ¼-inch bar spacing, washer/compactor system, and bar screen bypass system
Metering facilities	Magnetic flow meter on influent pipe
Odor control	Corrosion resistant plate covered channels, soil filter
Control	Continuous operation

5.2.3 Headworks

The headworks for the WWTP would typically include influent flow measurement, rotary type fine screens, and any required grit removal facilities. Due to the sources and quality of the wastewater, it is not expected that grit removal facilities are required at this time. However, fine screens are required to protect excessive fouling of the MBR membranes. The fine screens typically include a built-in washer/compactor and 2-mm openings that remove hair, inorganics, and wastes to protect the integrity of the membrane filters downstream. The washed and compacted screenings collected at the headworks are typically stored in bins on-site to be periodically disposed of at a landfill.

The raw influent would be pumped by the collection system pump station through the headworks facility. After flow measurement, influent would be routed to a covered headworks influent box for distribution to two influent channels. During normal operation, one channel would be in-service, with the other available as a standby. Slide gates would control flow to each channel. Each headworks channel would be sized to match the hydraulic capacity of the plant. Within the channels would be rotary type fine screens to remove large materials from the raw influent. A map showing a typical layout for the headworks facility is shown in **Figure 5-6**. **Table 5-3** shows some of the design criteria for the headworks facility.

Table 5-3: Headworks Design Criteria

Parameter	Value
Screening facilities	Enclosed cylindrical screen with 2-mm circular perforations, integral shaftless helical scraper/conveyor and compactor, mechanical washer to break up fecal material
Metering facilities	Magnetic flow meter on influent pipe
Odor control	Corrosion resistant plate covered channels, soil filter
Control	Continuous operation

5.2.4 Immersed Membrane Bioreactor System (Packaged)

An MBR is recommended because of the ease of permitting the plant due to the high-quality effluent, and the effluent's potential suitability for recycled water and discharge. Sewage would travel between the headworks and the MBRs within a covered influent distribution force main. The force main would pass through headworks to an influent splitter box that would evenly distribute the flow to the two MBR process trains. Sluice gates would be provided to isolate basins for maintenance.

Each MBR process train is divided into three sections: an anoxic section, an aerobic section with mechanical mixers, and an aerobic section containing the immersed membranes. A typical layout for the MBR is shown in **Figure 5-7**. The proposed WWTP would meet the design flow requirements specified in **Section 2.2.2**. The general configuration of the packaged MBR would be as follows.

Anoxic Basin: Within the anoxic basin, the influent is mixed with mixed liquor in a tank with dissolved oxygen (DO) equal to zero. The mixed liquor is pumped back to the anoxic basin from the immersed membrane section of the MBR. The introduction of new influent wastewater to the basin provides a substrate for the return activated sludge to respire and synthesize. The lack of DO in the basin facilitates nitrification and denitrification. Ammonia compounds are converted to nitrates by nitrifying bacteria. Denitrifying bacteria convert nitrates to nitrogen gas, which volatilize out of the basin. The proportion of recirculated mixed liquor to the volume of influent is approximately 6:1. The anoxic basin has a relatively small retention time compared to the aeration basin or the immersed membrane section, due to its smaller volume.

Aeration Basins: The mixed liquor produced by the anoxic basin would flow by gravity through a short channel to the adjacent aeration basin. The aeration basin differs from the anoxic basin in that this basin contains DO which is introduced to the tank through a series of fine bubble diffusers connected by headers and pumped by a series of blowers. The DO is required to convert dissolved organic material into a filterable solid material. In this process, aerobic bacteria utilize the carbon in the wastewater for respiration and cell synthesis. The primary outcomes from this basin are an overall reduction in BOD and the production of a filterable floc.

Immersed Membranes: The microfiltration membranes are long, hollow, spaghetti-like fibers with a nominal pore size of between 0.1-0.4 microns. Each of the individual microfiltration membranes is bundled together into modules, and each module is approximately 6 inches in diameter and 5 feet tall. The modules are grouped into sets, called cassettes, which are immersed into the mixed liquor solution. Each of the membrane modules is attached to headers, which create a suction and force water (permeate) through the membrane into the hollow center and onwards to the disinfection process. The mixed liquor that is not forced through the membrane is recirculated back to the anoxic zone. A portion of this recirculated mixed liquor is wasted to the dewatering system and disposed.

Each MBR train contains one permeate pump to force water through the membrane, with one additional standby permeate pump for the overall process that can draw from either train. These pumps can also pump permeate to the backpulse tanks, where water is stored in order to backwash the membrane. The permeate pumps also function as backpulse pumps, which pump permeate from the permeate tanks back to the membranes and keeps solids from accumulating on the membrane surface. The membranes are typically backwashed every 15 minutes, and each backwash lasts about two minutes. The entire backwash process is controlled by a programmable logic controller (PLC), which operates automatic control valves and isolates the membranes from the permeate pumping process. Sodium hypochlorite and/or citric acid is typically injected into the backpulse flow to facilitate membrane cleaning and prevent regrowth in the membrane modules.

Other facilities: A number of pumps, blowers, chemical storage, chemical metering, control, and electronic facilities are required in order to operate the MBR process. Some of these facilities are typically located in a building near the MBR process or are included on an equipment pad near the MBR system fully enclosed with sound attenuation provisions. Typically, an operations building is constructed which houses plant controls, the motor control center, maintenance facilities, chemical storage and metering, a laboratory, restroom/washroom, and offices/space for staff. During design development, these facilities will be further defined. **Figure 5-8** shows the proposed electrical, controls, and operations building.

It is typical for a wastewater facility design to include equalization and emergency storage capacity. Equalization capacity would be accomplished by a concrete tank either at or below grade of a to-be-determined volume and size to moderate the peak daily flows entering the WWTP. Emergency storage is typically a buried concrete or reinforced plastic tank that is gravity fed and drained from the sewage lift station designed to provide sufficient capacity for a peak flow event (or to-be-determined volume) if the lift station fails to deliver.

5.2.5 Ultraviolet Disinfection

Disinfection to meet discharge and reclamation virus and coliform water quality standards would be provided by constructing or installing a UV disinfection system in the operations building. UV disinfection facilities are typically contained within a long, narrow steel channel tank or pipe channel, with banks of UV lamps situated in a laminar flowing channel. A weir would control the water level in the channel, ensuring that the lamps are always submerged. Each UV lamp emits a light with a specific wavelength that is capable of inactivating bacteria and viruses, preventing them from reproducing. A proposed location for UV facilities is shown in **Figure 5-8** in the operations building floor plan. **Table 5-4** shows a summary of the recommended UV disinfection design criteria.

Table 5-4: UV Disinfection Design Criteria

Parameter	Value
Lamp location	In-line
Type of lamps	2020W medium pressure UV lamps
Transmittance	65% through quartz sleeve
Flow metering	Magnetic flow meter

5.2.6 Chlorine Disinfection

Though the UV facilities would be designed to disinfect the treated wastewater, they do not continue to disinfect the wastewater after it leaves the UV channel. In order to prevent regrowth of bacteria in the recycled water distribution system, sodium hypochlorite is typically added in small quantities. The introduction of this chemical creates a residual concentration of chlorine that persists in the recycled water and ensures that it is safe to use after it leaves the WWTP. Typical recycled water distribution systems require at least a positive chlorine residual at the point of use, and the dosing of sodium hypochlorite will be adjusted to meet this goal. It is believed that a dose of between 2-3 mg/L for recycled water used for on-site irrigation, cooling, or toilet/urinal flushing would suffice. Chlorine would be dosed at a location downstream of the UV disinfection facilities, and before recycled water is pumped to the recycled water storage tank.

Sodium hypochlorite is a very common disinfectant in the treatment and disinfection of wastewater. It is used throughout the wastewater industry for chlorine disinfection, and when used in accordance with that chemical's SDS, is safe for use for this purpose.

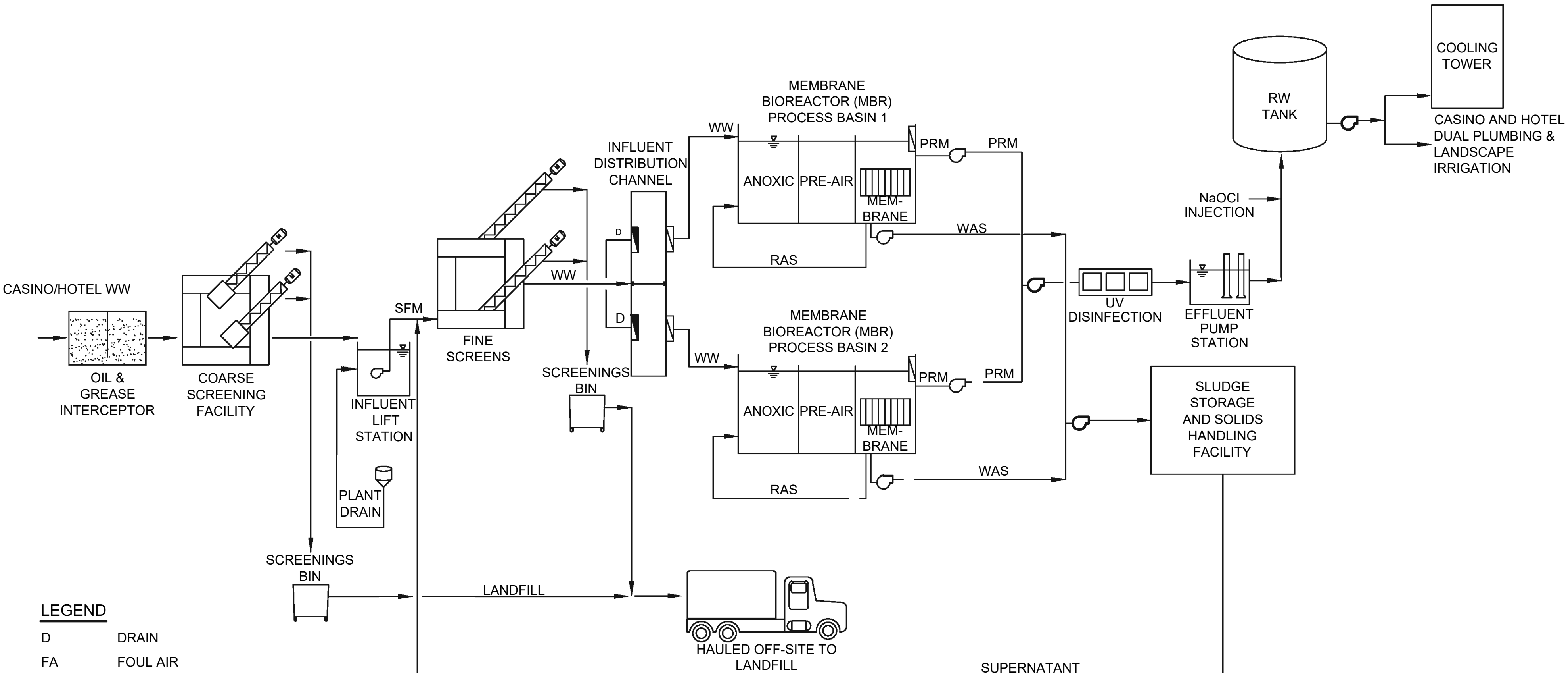
5.2.7 Effluent Pump Station

The purpose of the effluent pump station would be to pump treated wastewater to the recycled water storage tank for storage and use/disposal.

5.2.8 Operation and Maintenance

A detailed description of the operations and maintenance program will be prepared following completion of the WWTP design. However, it is expected that the WWTP would be operated and maintained similarly to the standards of other tertiary WWTPs in California.

To this effect, this WWTP will be staffed with operators who are qualified to operate the plant safely, effectively, and in compliance with all permit requirements and regulations. It is expected that the operators will have qualifications similar to those required by the SWRCB Operator Certification Program. This program specifies that for tertiary level WWTPs with design capacities of 1.0 MGD or less, the chief plant operator must be at least a Grade III operator. Supervisors and Shift Supervisors must be at least a Grade II.



LEGEND

D	DRAIN
FA	FOUL AIR
OF	OVERFLOW
PRM	PERMEATE WATER
PW	POTABLE WATER
RAS	RETURN ACTIVATED SLUDGE
RW	RECYCLED WATER
SFM	SEWER FORCE MAIN
WAS	WASTE ACTIVATED SLUDGE
WW	WASTEWATER

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Figure 5-3
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Wastewater Treatment Process Flow Diagram

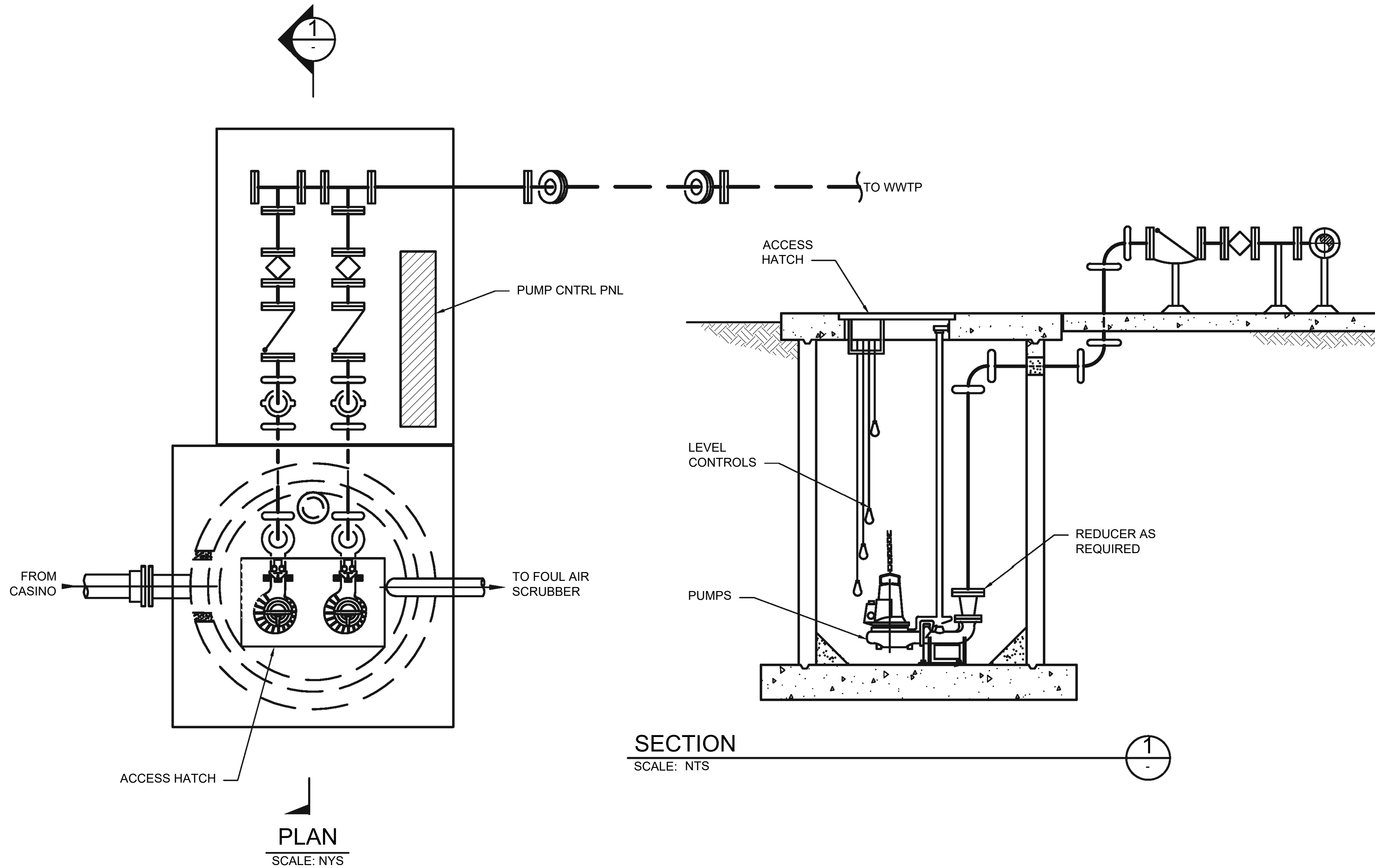

Figure 5-4
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Typical Influent Lift Station

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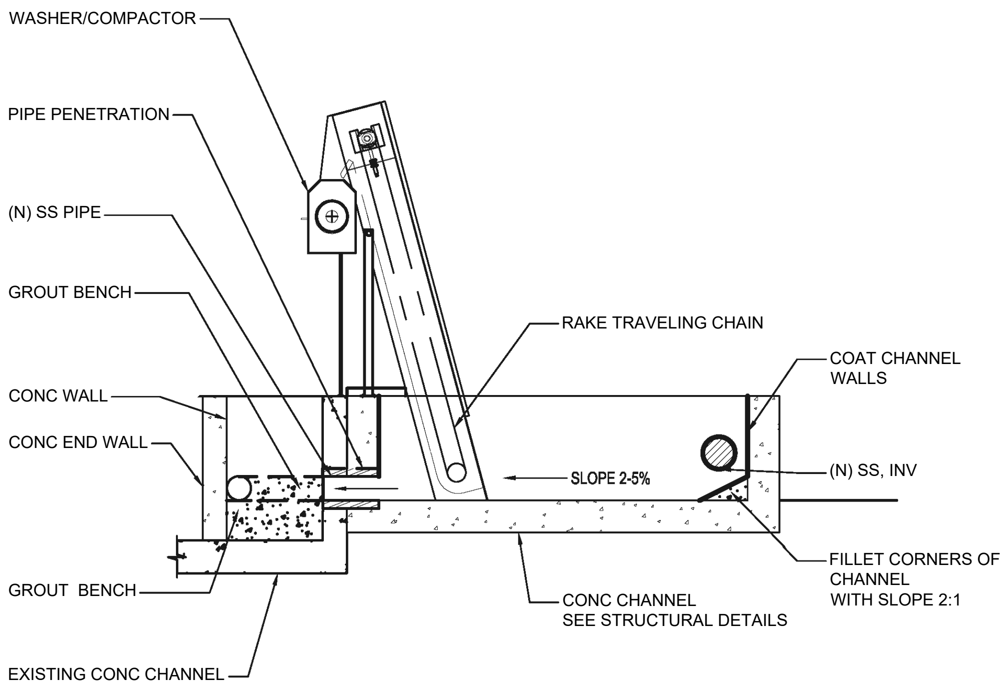
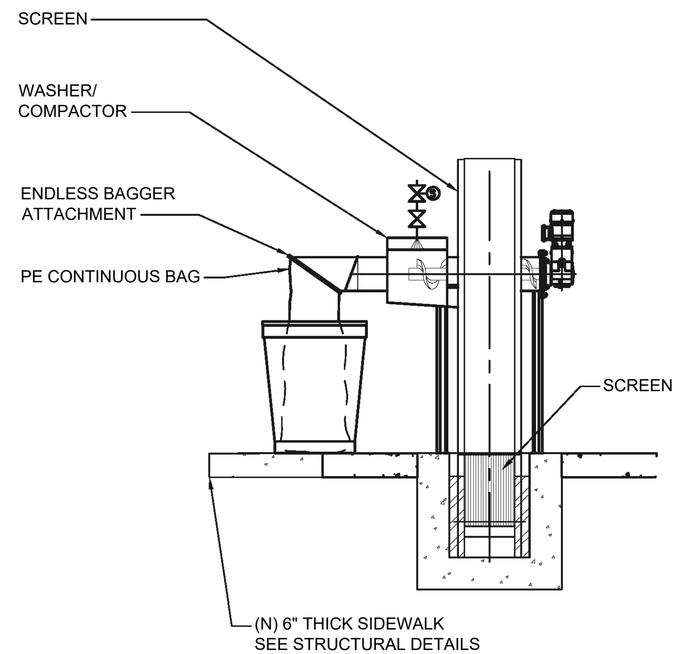


TYPICAL SCREEN SECTION (A / -)

SCALE: NTS



TYPICAL SCREEN SECTION (B / -)

SCALE: NTS

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Figure 5-5

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Scotts Valley Rancheria Water and Wastewater Feasibility Study

Typical Coarse Screening Facility

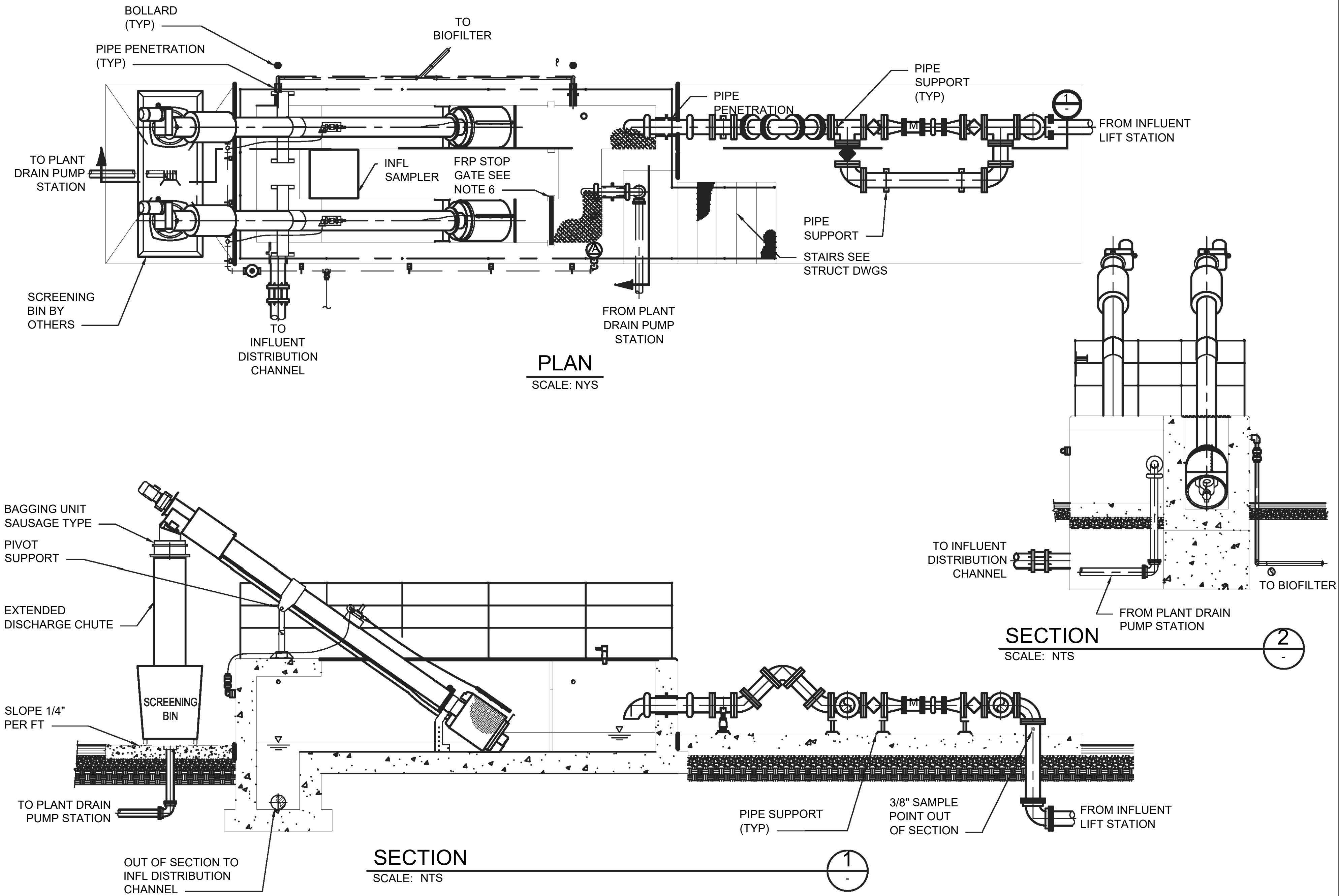

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Figure 5-6
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Typical Headworks Facility

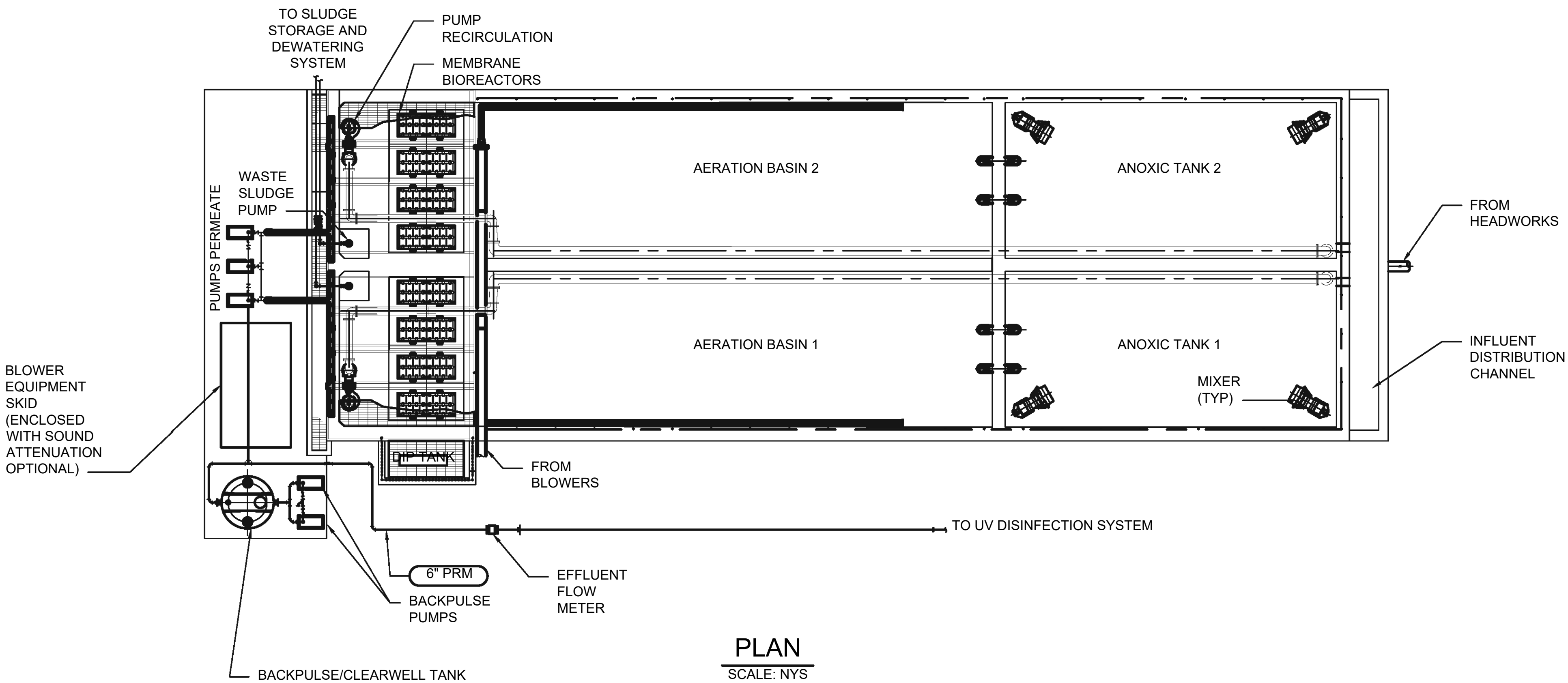

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Figure 5-7
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Preliminary Wastewater Treatment Facility Plan

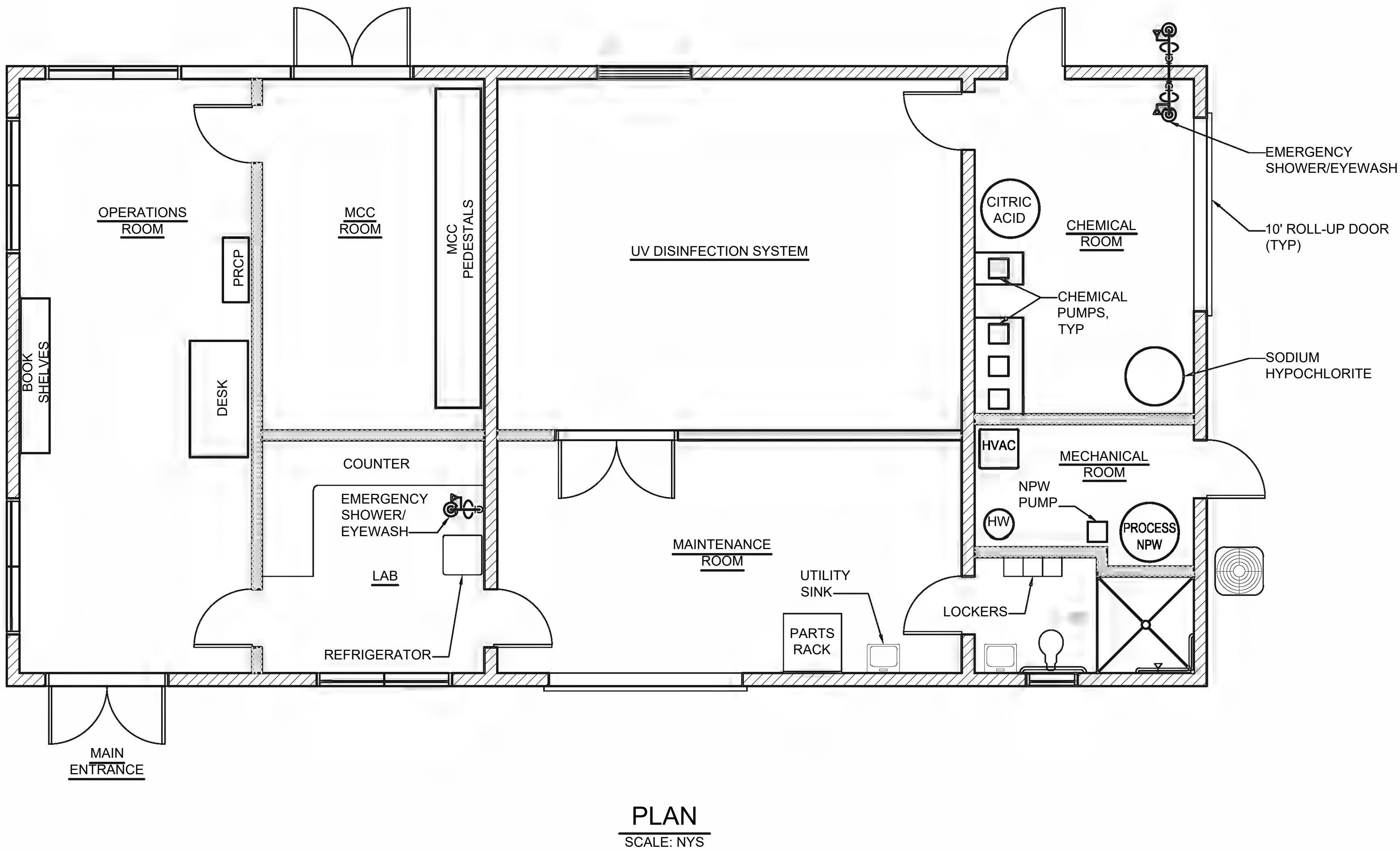

Figure 5-8
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Preliminary Operations Building Floor Plan

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5.3 Recycled Water

The recommended methods for effluent disposal would include maximizing on-site recycled water use including on-site landscape irrigation. It is assumed that recycled water would be supplied primarily to the casino facility for landscape irrigation, toilet and urinal flushing, and cooling tower makeup. Potential off-site options would include providing recycled water to off-site users for irrigation purposes.

The recommended on-site recycled water facilities include a recycled water storage tank. The need for a pump station would be determined based on the location and elevation of the storage tank. The ultimate location of the recycled water facilities will be based on the final design of the Project facilities. All of the recommended facilities described in this section are preliminary and should be utilized for planning purposes only.

5.3.1 On-Site Recycled Water Facilities

In order to maximize recycled water use on-site, it is assumed that the casino building will be dual-plumbed with both potable and recycled water. The primary uses of recycled water will be for toilet and urinal flushing, on-site landscape irrigation, and cooling tower makeup. The on-site recycled water reuse facilities will be designed to ensure that they comply with all USEPA standards (typically deferred to California’s Title 22 standards). The required on-site facilities will be identified and designed upon completion of a site plan and preliminary engineering including:

- Recycled water irrigation facilities marked in a purple color.
- Signage informing the public recycled water is used.
- Pipelines in separate trenches a minimum distance away from other water pipelines.
- Labeling of recycled water valves, boxes, and sprinkler heads.

Within the building, the interior plumbing system will have to be plumbed separately from the building’s potable water system and contain no cross connections. The dual plumbed piping systems must be distinctly marked and color-coded.

Estimated recycled water generated by the project and demands are provided in **Table 5-5**. Irrigation demand assumes landscaped are is approximately 5 acres for each alternative.

Table 5-5: Recycled Water Generated and Project Demands (Average Year)

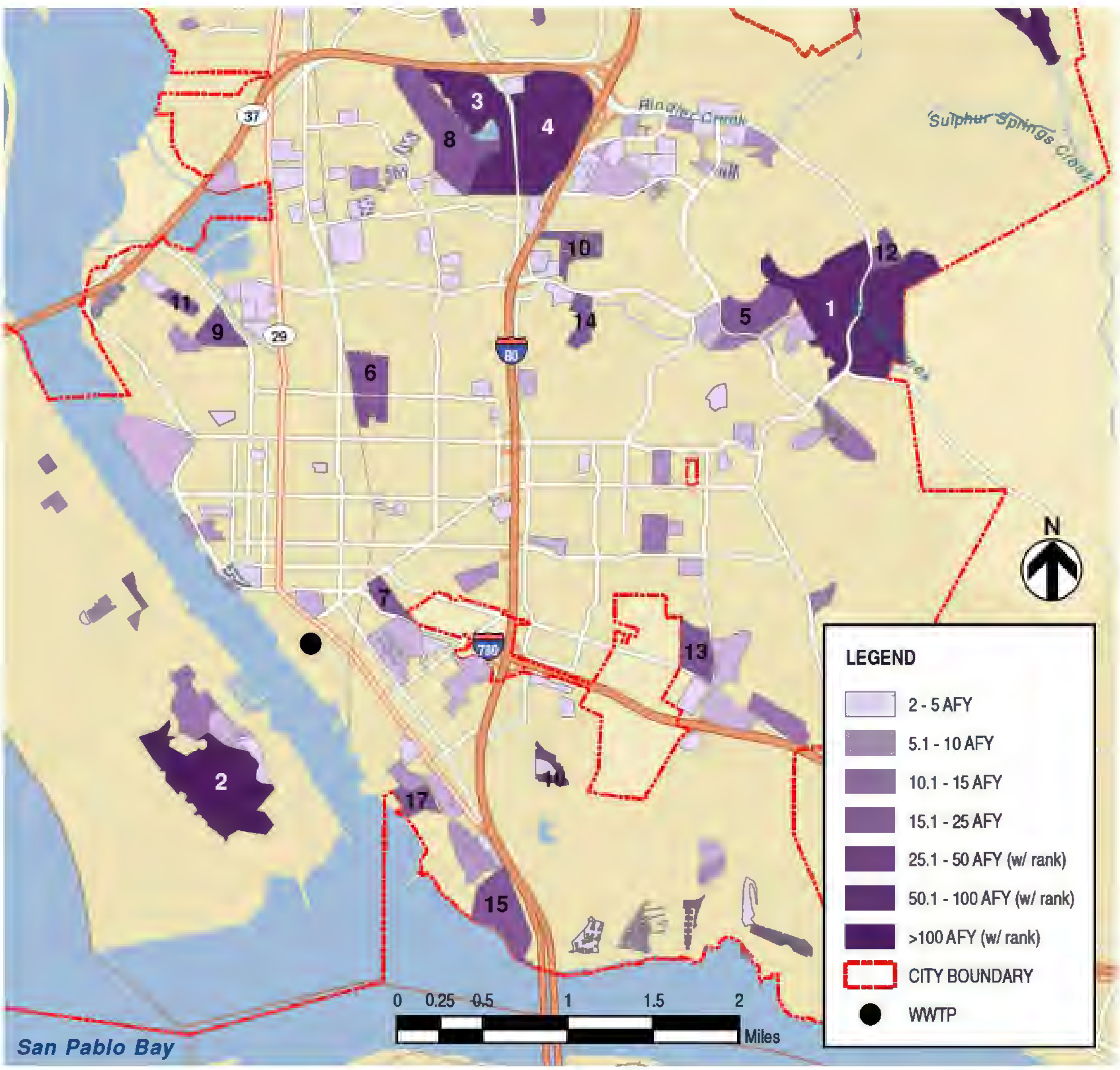
Alternative	RW Generated (AFY)	Dual Plumbed Demand (AFY)	Cooling Demand (AFY)	Landscape Irrigation Demand (AFY)[1]	Excess RW (AFY)
Alternative A	241	62.7	30	12.4	135.9
Alternative B	233	62.7	30	12.4	127.9
Alternative C	70	13.4	6.9	12.4	37.3

Notes:
1. Assuming approximately five acres of landscaped area.

5.3.2 Off-site Recycled Water Opportunities

In 2018, the District prepared a Recycled Water Facilities Plan (RWFP). That plan identified potential recycled water uses and quantified opportunities for recycled water use within the City based on the most cost-effective users. In that analysis, the Blue Rock Springs Golf Club was identified as one of the top potential recycled water users with a demand potential greater than 100 AFY. That site is located along Columbus Parkway, less than two miles southeast of the Project (noted with a "1" in the figure). **Figure 5-9** shows the potential recycled water demands from the RWFP. Blue Rock Springs is irrigated with approximately 500 AFY of untreated raw water provided by the City. There are several water features within the golf club; it is presumed that irrigation water is stored within these and that they could be augmented with recycled water to provide seasonal storage.

Figure 5-9: Potential Recycled Water Demands (VFWD, 2018)

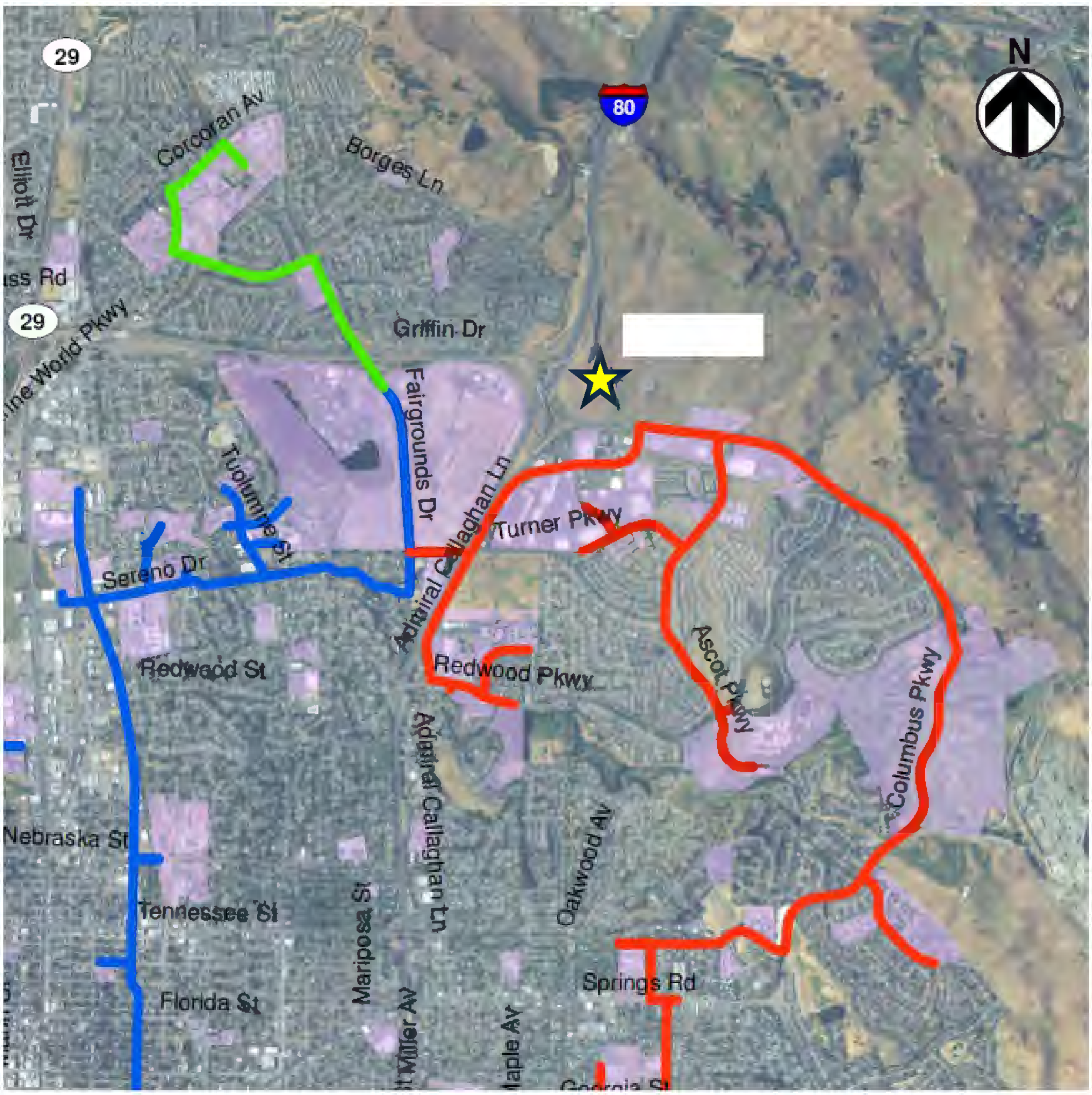


Source: Recycled Water Facilities Plan, VFWD, March 2018, Figure 6-1

A conceptual alignment was identified along Columbus Parkway to serve that site, which also fronts the Project site. **Figure 5-10** shows those conceptual alignments identified in the RWFP.

For an on-site WWTP alternative, it is recommended that the opportunity to develop a recycled water distribution system be explored with the City and District. Augmenting their water supply with recycled water can offset the use of raw water provided by the City. The RWFP is included as **Appendix F**.

Figure 5-10: Conceptual Recycled Water Alignments (VFWD, 2018)



Source: Recycled Water Facilities Plan, VFWD, March 2018, Figure 10-1

5.3.3 Recycled Water Storage Tank and Pump Station

Where seasonal storage and irrigation could be located off-site, the purpose of an onsite recycled water storage tank would be to provide peak day storage for on-site recycled water use for Project toilet and urinal flushing, on-site landscaping (assumed approximately 5 acres), and cooling tower makeup.

This storage tank would be similar to the potable water storage tank with respect to construction methods. A typical section for the tank is shown as **Figure 4-3**. **Table 5-6** shows a summary of the recommended storage tank design criteria assuming the stored recycled water would supply only the Casino facility indoor uses.

Table 5-6: Recycled Water Storage Tank Design Criteria

Parameter	Alternatives A&B	Alternative C
Approximate size	100,000 gallons	50,000 gallons
Approximate diameter	32 feet	24 feet
Approximate height	16 feet	16 feet

The effluent pump station would pump recycled water from the WWTP to the recycled water storage tank. A recycled water pump station combined with a hydropneumatic tank can be used to supply the distribution system and maintain system pressure. **Table 5-7** shows a summary of the recommended pump station design criteria.

Table 5-7: Recycled Water Pump Station Design Criteria

Parameter	Value
Pump number	2
Pump type	Variable speed turbine
Hydropneumatic tank approximate volume range[1]	500 – 1,000 gallons

Notes:
1. Exact volume is to be determined during the design phase of the project. Tank volume is dependent on the desired flowrate and pressure from the hydropneumatic tank.

5.3.4 On-site Seasonal Storage and Irrigation

The onsite recycled water storage tank and pump station may be sized to provide seasonal storage. Seasonal storage would be designed to store the volume of recycled water generated during the wet season when there is little to no irrigation demand.

A water balance was developed to assess the seasonal storage and disposal requirements assuming a 100-year rainfall followed by an average year. The seasonal storage volume required for the project alternatives along with the recycled water irrigation area needed is provided in **Table 5-8**. This represents the maximum irrigation area to achieve the minimum storage volume. Additional storage volume would reduce the irrigation area necessary. Each alternative considers the use of recycled water for dual-plumbing and cooling. These estimates are preliminary and are for planning purposes only. Copies of the water balances for each alternative are provided as **Appendix G**.

Table 5-8: Estimated Seasonal Storage and Irrigation Requirements

Project Alternative	Irrigation Area[1] (Acres)	Irrigation Demand[2] (AF)	Cooling Tower Makeup Demand (AF)	Dual Plumbing Demand (AF)	Minimum Storage (AF)
Alternative A	194	480	30	63	64.5
Alternative B	185	458	30	63	61.2
Alternative C	64	157	7	13	21.3

Notes:
1. This disposal strategy assumes that all effluent will be disposed to the irrigated areas from April to October and stored in a closed storage tank during the wet season. This represents the maximum area required to minimize storage. Irrigation area can be reduced with increased storage volume.
2. Represents irrigation demand for total irrigated area and may be more than available recycled water generated. Location of irrigation areas are to be determined.

The limiting month at the end of the dry season is the month of November when irrigation demand drops to zero. It is noted that the volume of irrigation water is roughly equivalent to the estimated demand of the Blue Rock Springs Golf Club, which is approximately 500 AFY.

Due to the topography and geological challenges within the Project site, the location and design of open seasonal storage ponds requires further investigation. Closed storage tanks are assumed for planning purposes. Capacity, number, and dimensions are provided in **Table 5-9**.

Table 5-9: Seasonal Recycled Water Storage Tank Capacity and Dimensions

Project Alternative	Max Storage (AF)	Max Storage (MG)	No. of Tanks	Height (ft)	Diameter (ft)
Alternative A	64.5	21	3	40	173
Alternative B	61.2	20	3	40	169
Alternative C	21.3	7	1	40	173

Proposed siting of storage tanks is provided in **Figure 5-11** and **Figure 5-12**.

Figure 5-11
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Alternatives A & B Proposed WTP, WWTP, and RW Storage Site Plan

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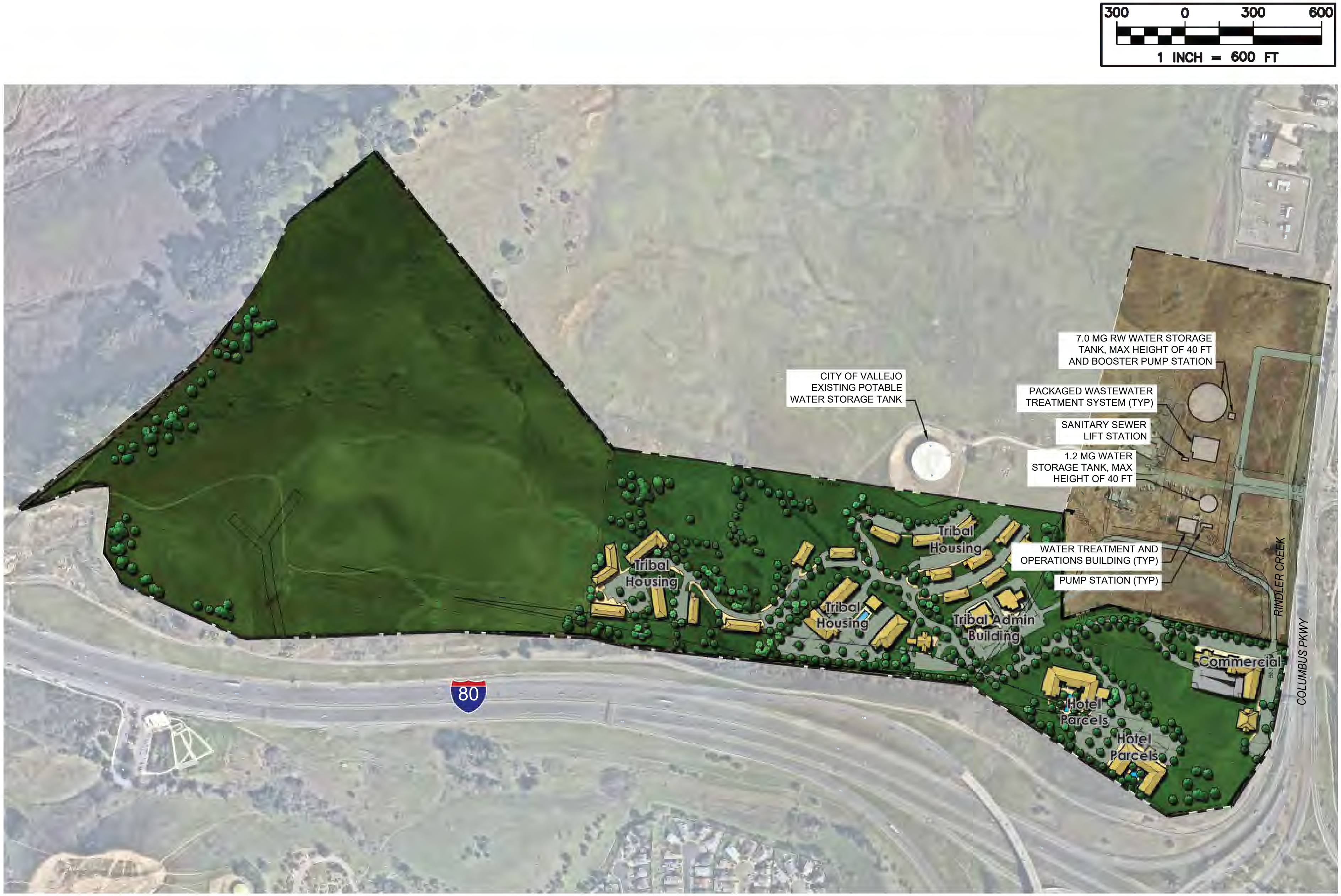

Figure 5-12
Acorn Environmental
Scotts Valley Rancheria Water and Wastewater Feasibility Study
Alternative C Proposed WTP, WWTP, and RW Storage Site Plan

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SECTION 6 – RECOMMENDATIONS

This feasibility study report makes the following preliminary recommendations with respect to the proposed Project.

6.1 Water Supply

As discussed in **Section 2.3.1** and **Appendix B**, there are several water supply limitations identified at the Project site that require further investigation. It is anticipated that connection to the City's municipal water supply system and construction of an on-site water storage tank and pump station will be required for the Project. The configuration of these facilities is based on the water storage tank located within the proposed utility area. If it were possible to construct the storage tank at a higher elevation to take advantage of elevation head to provide pressure to the system, the pumping configuration would be modified.

6.2 Wastewater Handling

If a District connection is not feasible due to District capacity limitations, then a new WWTP should be constructed on-site to treat wastewater generated on-site. The WWTP would be designed to produce tertiary level recycled water for unrestricted reuse. The Project should maximize the on-site recycling of wastewater and seek off-site disposal options in partnership with the City and District.

The following wastewater handling facilities would be recommended:

- Immersed MBR WWTP with UV Disinfection & Chlorination
- Effluent pump station
- Recycled water storage tank and distribution pump station
- Off-site recycled water disposal

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SECTION 7 – REFERENCES

California Regional Water Quality Control Board – San Francisco Bay Region, Order R2-2023-0001, NPDES Permit CA0037699, Vallejo Flood and Wastewater District, February 8, 2023.

City of Vallejo, 2020 Urban Water Management Plan, October 12, 2021

City of Vallejo, General Plan 2040 Land Use Map, February 11, 2020

City of Vallejo, Revised Standard Specifications and Standard Drawings, December 20, 2011

City of Vallejo, Water Master Plan, August 28, 2015

ENGEO, Inc. Hydrogeologic Assessment, 2024.

ENGEO, Inc. Infiltration Report, 2024.

Vallejo Flood and Wastewater District Engineering Standards, July 2020

Vallejo Flood and Wastewater District, Recycled Water Facilities Plan, March 2018

Vallejo Flood and Wastewater District, Sanitary Sewer Collection System Master Plan, August 2023

Vallejo Flood and Wastewater District, Sewer System Management Plan, Update December 2022

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APPENDIX A
Acorn Environmental
Water and Wastewater Feasibility Study
Projected Water Demands and Wastewater Flows

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Project: **Water/Wastewater Feasiblity Study**
Client: **Scotts Valley Rancheria**
Date: **6/27/2024**
Title: **Water Demand and Wastewater Flow Projections**

Element	**Alt A (gpd)**	**Alt B (gpd)**	**Alt C (gpd)**
Wastewater Flow			
Average Day	215,000	208,000	108,000
Peak Day Flow	323,000	312,000	162,000
Peaking Factor	1.5	1.5	1.5
Water Demand			
Average Day	258,000	250,000	130,000
Peak Day Flow	387,000	374,000	194,000
Peaking Factor	1.5	1.5	1.5
Recycled Water Demands / Disposal			
Average Day (Dual Plumbing)	53,000	53,000	12,000
Average Day (Cooling Tower)	27,000	27,000	6,000
Average Day (5.0 acres Landscape)	11,053	11,053	11,053
Net Wastewater Flow			
Average Day	176,947	169,947	90,947
Net Water Demand			
Average Day	205,000	197,000	118,000
Recycled water demand assumed to be 26% of ww inflow to specific facilities (i.e, Casino, hotel, commercial facilities) based on metered data from a similar project. For cooling tower makeup, assumed RW demand 100% of ww inflow for cooling tower.			

Project: **Water/Wastewater Feasiblity Study**
Client: **Scotts Valley Rancheria**
Date: **6/27/2024**
Title: **Water Demand and Wastewater Flow Projections**
Subject: **Alterntive A & B - w/ Gaming/Casino**

					Weekday		Weekend		
Element	**Units**	**Quantity**	**Unit Flow (gpd/unit)**	**Base Flow**	**Factor**	**Average Daily Flow**	**Factor**	**Average Daily Flow**	**WTG Average Daily Flow**
			gpd/unit	**gpd**	**%**	**gpd**	**%**	**gpd**	**gpd**
Employees	per day	3,600	12	43,200	80%	34,560	80%	34,560	34,560
Casino	SF	614,959							
Gaming Floor	SF	238,266	0.6	142,960	50%	71,480	100%	142,960	102,114
BOH	SF	218,533	0	0	20%	0	80%	0	0
Lobby/Cashier/Club	SF	25,189	0	0	20%	0	80%	0	0
Restaurant	Seats	811	70	56,770	50%	28,385	80%	45,416	35,684
Bars & Brew Pub	Seats	602	40	24,080	20%	4,816	80%	19,264	11,008
Coffee Shop	Seats	74	40	2,960	20%	592	80%	2,368	1,353
Food Hall	Seats	182	60	10,920	50%	5,460	80%	8,736	6,864
Ballroom/Pre-Function Area	SF	52,794	0.75	39,596	0%	0	80%	31,676	13,576
Cooling Tower Blowdown	LS	26,737	1	26,737	10%	2,674	10%	2,674	2,674
Subtotal (ALT B)				**347,222**		**147,967**		**287,654**	**207,832**
Tribal Community									
Single-Family Homes	EDU	24	290	6,960	100%	6,960	100%	6,960	6,960
Admin Building	employees	30	12	360	100%	360	100%	360	360
Subtotal				**7,320**		**7,320**		**7,320**	**7,320**
Subtotal (ALT A)				**354,542**		**155,287**		**294,974**	**215,152**
AVG WW FLOWS (INCREMENTAL INCREASE) - ALT A									**215,200**
AVG WW FLOWS (INCREMENTAL INCREASE) - ALT B									**207,900**
MAX DAY FLOWS - ALT A				**PF**	**1.5**				**322,800**
MAX DAY FLOWS - ALT B				**PF**	**1.5**				**311,850**
PEAK HOURLY FLOWS - ALT A (GPM)				**PF**	**3**				**448**
PEAK HOURLY FLOWS - ALT B (GPM)				**PF**	**3**				**433**
Avg Potable Water Demand for Facilities (20% Increase over WW Flow Est)									
AVG W FLOWS (INCREMENTAL INCREASE) - ALT A									**258,200**
AVG W FLOWS (INCREMENTAL INCREASE) - ALT B									**249,500**
MAX DAY FLOWS - ALT A				**PF**	**1.5**				**387,300**
MAX DAY FLOWS - ALT B				**PF**	**1.5**				**374,250**
PEAK HOURLY FLOWS - ALT A (GPM)				**PF**	**3**				**538**
PEAK HOURLY FLOWS - ALT B (GPM)				**PF**	**3**				**520**

Notes:

1. Quantity of employees per day is based on 3-8 hour work shifts in a 24 hour day which is appx 900 employees at any given time.
2. BOH and Lobby/Cashier/Club flows are assumed to be covered in other line items (i.e., employee or gaming).

Project: **Water/Wastewater Feasiblity Study**
Client: **Scotts Valley Rancheria**
Date: **6/27/2024**
Title: **Water Demand and Wastewater Flow Projections**
Subject: **Alterntive C - Non Gaming**

					Weekday		Weekend		
Element	**Units**	**Quantity**	**Unit Flow (gpd/unit)**	**Base Flow**	**Factor**	**Average Daily Flow**	**Factor**	**Average Daily Flow**	**WTG Average Daily Flow**
			gpd/unit	**gpd**	**%**	**gpd**	**%**	**gpd**	**gpd**
Hotel		141,012							
Hotel 1	rooms	132	250	33,000	30%	9,900	100%	33,000	19,800
Hotel 2	rooms	132	250	33,000	30%	9,900	100%	33,000	19,800
Cooling Tower Blowdown	LS	1	6,131	6,131	10%	613	10%	613	613
Subtotal				**66,000**		**19,800**		**66,000**	**40,213**
Commercial	SF	129,702							
Commercial 1	SF	120,474	0.1	12,047	20%	2,409	80%	9,638	5,507
Commercial 2	SF	9,228	0.1	923	20%	185	80%	738	422
Subtotal				**12,970**		**2,594**		**10,376**	**5,929**
Tribal Community									
Single-Family	EDU	50	290	14,500	100%	14,500	100%	14,500	14,500
Admin Building 1	employees	30	12	360	100%	360	100%	360	360
Admin Building 2	employees	30	12	360	100%	360	100%	360	360
Admin Building 3	employees	30	12	360	100%	360	100%	360	360
Subtotal				**15,580**		**15,580**		**15,580**	**15,580**
Subtotal (ALT C)				**94,550**		**37,974**		168,945	107,865
AVG WW FLOWS (INCREMENTAL INCREASE) - ALT C									**107,900**
MAX DAY FLOWS - ALT C				**PF**	**1.5**				**161,850**
PEAK HOURLY FLOWS - ALT C (GPM)				**PF**	**3**				**225**
Avg Potable Water Demand for Facilities (20% Increase over WW Flow Est)									
AVG W FLOWS (INCREMENTAL INCREASE) - ALT C									**129,500**
MAX DAY FLOWS - ALT C				**PF**	**1.5**				**194,250**
PEAK HOURLY FLOWS - ALT C (GPM)				**PF**	**3**				**270**

Notes:

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APPENDIX B

Acorn Environmental
Water and Wastewater Feasibility Study
ENGEO, Inc. Infiltration Report

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Infiltration Report

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651.237.5123

Engeo San Ramon

Scott's Valley - 16484.000.001 - Vallejo, CA

This report summarizes the results of a set of Modified Philip Dunne (MPD) Infiltrometer tests performed at the above referenced site. Engeo San Ramon personnel performed the field tests. The software used to compute saturated hydraulic conductivity (K_{sat}) and generate this report assumes that the field personnel used infiltrometers manufactured by Upstream Technologies Inc. and followed the procedures outlined in “Manual – Modified Philip - Dunne Infiltrometer” by Ahmed, Gulliver, and Nieber.

The following paragraphs describe the individual tests, input values used in the analysis, and methods used to compute the K_{sat} value.

After individual K_{sat} values were calculated, the method used to determine the overall site K_{sat}value ($K_{best\text{-}fit}$) is described in "Effective Saturated Hydraulic Conductivity of an Infiltration-Based Stormwater Control Measure" by Weiss and Gulliver 2015, "A relationship to more consistently and accurately predict the best-fit value of saturated hydraulic conductivity used a weighted sum of 0.32 times the arithmetic mean and 0.68 times the geometric mean."

METHOD USED TO COMPUTE K_{sat}

The MPD Infiltrometer software uses the following procedure described in "The Comparison of Infiltration Devices and Modification of the Philip-Dunne Permeameter for the Assessment of Rain Gardens" by Rebecca Nestigen, University of Minnesota, November 2007.

The steps are as follows:

1. For each measurement of head, use the following equation to find the corresponding distance to the sharp wetting front.

$$[H_0 - H(t)]r_1^2 = \frac{\theta_1 - \theta_2}{3}[2[R(t)]^3 + 3[R(t)]^2 L_{max} - L_{max}^3 - 4r_0^3]$$

2. Estimate the change in head with respect to time and the change in wetting front distance with respect to time by using the backward difference for all values of R(t) equal to or greater than the distance

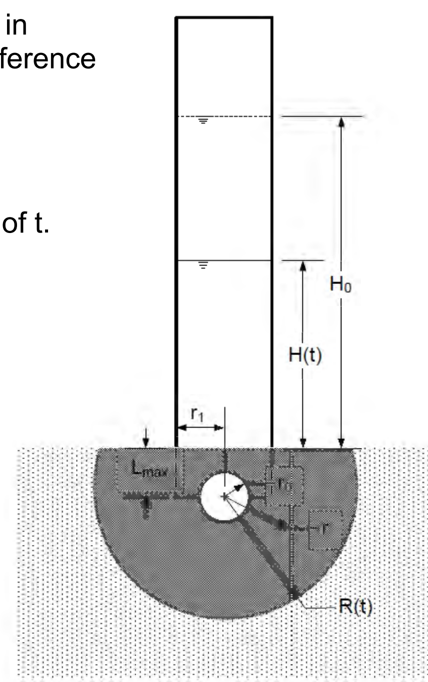
$$\sqrt{r_1^2 + L_{max}^2}$$

3. Make initial guesses for K and C.
4. Solve the following equations for ΔP(t) at each incremental value of t.

$$\Delta P(t) = \frac{\pi^2}{8}\left\{\theta_1 - \theta_0 \frac{[R(t)^2] + [R(t)]L_{max}}{K}\frac{dr}{dt} - 2r_0^2\right\} \frac{ln[\frac{R(t)[r_0 + L_{max}]}{r_0[R(t)+L_{max}]}]}{L_{max}}$$

$$\Delta P(t) = C - H(t) - L_{max} + \frac{L_{max}}{K}\frac{dh}{dt}$$

5. Minimize the absolute difference between the two solutions found in Step 4 by adjusting the values of K and C.



Parameters for Equations

Θ_0 = volumetric water content of soil before MPD test

Θ_1 = volumetric water content of soil after MPD test

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1mpd4

Date	4/9/2024
Time	8:24 AM
Latitude	38.137993
Longitude	-122.216017
Initial Volumetric Moisture	10.00 %
Final Volumetric Moisture	50.00 %
Cylinder Size	3 Liter

1mpd4 Results

Map Pin #	1
Test Number	27665
Ksat - mm/hr	79
Ksat - in/hr	3.12
Capillary Pressure C mm	-64.6
RMS Error of Regression	8.9
Normalized RMS	0.3%

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	34.54 cm	26	749 s	24.33 cm	51	1500 s	17.38 cm	76	2250 s	12.11 cm
2	30 s	34.04 cm	27	780 s	24.0 cm	52	1530 s	17.14 cm	77	2279 s	11.91 cm
3	59 s	33.53 cm	28	810 s	23.69 cm	53	1560 s	16.91 cm	78	2310 s	11.73 cm
4	90 s	33.05 cm	29	840 s	23.37 cm	54	1590 s	16.67 cm	79	2339 s	11.54 cm
5	120 s	32.56 cm	30	870 s	23.06 cm	55	1620 s	16.45 cm	80	2370 s	11.36 cm
6	150 s	32.11 cm	31	899 s	22.76 cm	56	1650 s	16.22 cm	81	2400 s	11.18 cm
7	180 s	31.64 cm	32	930 s	22.45 cm	57	1679 s	15.99 cm	82	2429 s	11.0 cm
8	210 s	31.19 cm	33	959 s	22.15 cm	58	1710 s	15.77 cm	83	2460 s	10.82 cm
9	239 s	30.74 cm	34	990 s	21.86 cm	59	1739 s	15.55 cm	84	2489 s	10.65 cm
10	270 s	30.31 cm	35	1019 s	21.57 cm	60	1770 s	15.33 cm	85	2520 s	10.47 cm
11	299 s	29.89 cm	36	1050 s	21.29 cm	61	1799 s	15.12 cm	86	2550 s	10.28 cm
12	330 s	29.48 cm	37	1079 s	21.0 cm	62	1830 s	14.91 cm	87	2579 s	10.11 cm
13	359 s	29.06 cm	38	1110 s	20.72 cm	63	1859 s	14.69 cm	88	2610 s	9.94 cm
14	390 s	28.67 cm	39	1139 s	20.43 cm	64	1890 s	14.48 cm	89	2640 s	9.77 cm
15	419 s	28.27 cm	40	1170 s	20.17 cm	65	1919 s	14.27 cm	90	2669 s	9.6 cm
16	450 s	27.89 cm	41	1200 s	19.89 cm	66	1950 s	14.06 cm	91	2700 s	9.42 cm
17	479 s	27.49 cm	42	1230 s	19.62 cm	67	1979 s	13.86 cm	92	2729 s	9.25 cm
18	510 s	27.12 cm	43	1260 s	19.36 cm	68	2010 s	13.66 cm	93	2759 s	9.09 cm
19	539 s	26.75 cm	44	1290 s	19.11 cm	69	2039 s	13.45 cm	94	2790 s	8.92 cm
20	570 s	26.39 cm	45	1320 s	18.85 cm	70	2070 s	13.26 cm	95	2819 s	8.76 cm
21	600 s	26.02 cm	46	1350 s	18.59 cm	71	2100 s	13.05 cm	96	2849 s	8.59 cm
22	629 s	25.68 cm	47	1380 s	18.35 cm	72	2129 s	12.86 cm	97	2880 s	8.43 cm
23	660 s	25.33 cm	48	1410 s	18.1 cm	73	2160 s	12.67 cm	98	2909 s	8.27 cm
24	689 s	24.99 cm	49	1440 s	17.86 cm	74	2189 s	12.48 cm	99	2939 s	8.11 cm
25	720 s	24.66 cm	50	1470 s	17.61 cm	75	2220 s	12.29 cm	100	2970 s	7.96 cm

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1mpd4 Readings continued

#	Time	Head
101	2999 s	7.8 cm
102	3029 s	7.65 cm
103	3060 s	7.49 cm
104	3089 s	7.33 cm
105	3120 s	7.2 cm
106	3150 s	7.05 cm
107	3179 s	6.89 cm
108	3210 s	6.75 cm
109	3239 s	6.6 cm
110	3270 s	6.46 cm
111	3300 s	6.31 cm
112	3329 s	6.17 cm
113	3360 s	6.03 cm
114	3389 s	5.9 cm
115	3420 s	5.76 cm
116	3450 s	5.61 cm
117	3479 s	5.47 cm
118	3510 s	5.33 cm

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651.237.5123

1mpd3

Date	4/9/2024
Time	9:42 AM
Latitude	38.138578
Longitude	-122.215725
Initial Volumetric Moisture	30.00 %
Final Volumetric Moisture	70.00 %
Cylinder Size	3 Liter

1mpd3 Results

Map Pin #	2
Test Number	27669
Ksat - mm/hr	NULL
Ksat - in/hr	NULL
Capillary Pressure C mm	NULL
RMS Error of Regression	NULL
Normalized RMS	NULL

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	36.39 cm	26	748 s	36.53 cm	51	1498 s	36.58 cm	76	2248 s	36.63 cm
2	28 s	36.39 cm	27	778 s	36.53 cm	52	1528 s	36.58 cm	77	2278 s	36.64 cm
3	58 s	36.39 cm	28	808 s	36.54 cm	53	1558 s	36.59 cm	78	2308 s	36.64 cm
4	88 s	36.39 cm	29	838 s	36.54 cm	54	1588 s	36.59 cm	79	2338 s	36.65 cm
5	118 s	36.4 cm	30	868 s	36.55 cm	55	1618 s	36.59 cm	80	2368 s	36.65 cm
6	148 s	36.41 cm	31	898 s	36.55 cm	56	1648 s	36.59 cm	81	2398 s	36.65 cm
7	178 s	36.41 cm	32	928 s	36.55 cm	57	1678 s	36.59 cm	82	2428 s	36.65 cm
8	208 s	36.42 cm	33	958 s	36.55 cm	58	1708 s	36.59 cm	83	2458 s	36.65 cm
9	238 s	36.43 cm	34	988 s	36.55 cm	59	1738 s	36.6 cm	84	2488 s	36.65 cm
10	268 s	36.44 cm	35	1018 s	36.56 cm	60	1768 s	36.6 cm	85	2518 s	36.66 cm
11	298 s	36.44 cm	36	1048 s	36.56 cm	61	1798 s	36.6 cm	86	2548 s	36.66 cm
12	328 s	36.46 cm	37	1078 s	36.56 cm	62	1828 s	36.6 cm	87	2578 s	36.66 cm
13	358 s	36.46 cm	38	1108 s	36.56 cm	63	1858 s	36.61 cm	88	2608 s	36.66 cm
14	388 s	36.47 cm	39	1138 s	36.56 cm	64	1888 s	36.61 cm	89	2638 s	36.66 cm
15	418 s	36.48 cm	40	1168 s	36.56 cm	65	1918 s	36.6 cm	90	2668 s	36.67 cm
16	448 s	36.48 cm	41	1198 s	36.56 cm	66	1948 s	36.61 cm	91	2698 s	36.67 cm
17	478 s	36.49 cm	42	1228 s	36.56 cm	67	1978 s	36.59 cm	92	2728 s	36.67 cm
18	508 s	36.5 cm	43	1258 s	36.56 cm	68	2008 s	36.6 cm	93	2758 s	36.69 cm
19	538 s	36.52 cm	44	1288 s	36.57 cm	69	2038 s	36.61 cm	94	2788 s	36.67 cm
20	568 s	36.52 cm	45	1318 s	36.57 cm	70	2068 s	36.61 cm	95	2818 s	36.69 cm
21	598 s	36.53 cm	46	1348 s	36.57 cm	71	2098 s	36.62 cm	96	2848 s	36.69 cm
22	628 s	36.49 cm	47	1378 s	36.58 cm	72	2128 s	36.62 cm	97	2878 s	36.69 cm
23	658 s	36.5 cm	48	1408 s	36.58 cm	73	2158 s	36.62 cm	98	2908 s	36.69 cm
24	688 s	36.52 cm	49	1438 s	36.58 cm	74	2188 s	36.63 cm	99	2938 s	36.69 cm
25	718 s	36.52 cm	50	1468 s	36.58 cm	75	2218 s	36.63 cm	100	2968 s	36.69 cm

1mpd3 Readings continued

#	Time	Head
101	2998 s	36.7 cm
102	3028 s	36.7 cm
103	3058 s	36.7 cm
104	3088 s	36.69 cm
105	3118 s	36.66 cm
106	3148 s	36.67 cm

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1mpd1

Date	4/9/2024
Time	10:56 AM
Latitude	38.140518
Longitude	-122.215576
Initial Volumetric Moisture	60.00 %
Final Volumetric Moisture	80.00 %
Cylinder Size	3 Liter

1mpd1 Results

Map Pin #	3
Test Number	27670
Ksat - mm/hr	NULL
Ksat - in/hr	NULL
Capillary Pressure C mm	NULL
RMS Error of Regression	NULL
Normalized RMS	NULL

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	29 s	31.47 cm	26	778 s	31.54 cm	51	1529 s	31.64 cm	76	2279 s	31.73 cm
2	58 s	31.47 cm	27	809 s	31.55 cm	52	1559 s	31.64 cm	77	2309 s	31.73 cm
3	89 s	31.49 cm	28	839 s	31.55 cm	53	1588 s	31.64 cm	78	2338 s	31.74 cm
4	118 s	31.48 cm	29	868 s	31.56 cm	54	1619 s	31.64 cm	79	2369 s	31.74 cm
5	149 s	31.49 cm	30	899 s	31.56 cm	55	1648 s	31.65 cm	80	2399 s	31.74 cm
6	178 s	31.5 cm	31	928 s	31.56 cm	56	1679 s	31.65 cm	81	2428 s	31.75 cm
7	209 s	31.5 cm	32	959 s	31.57 cm	57	1709 s	31.66 cm	82	2459 s	31.7 cm
8	238 s	31.51 cm	33	988 s	31.57 cm	58	1738 s	31.66 cm	83	2488 s	31.71 cm
9	269 s	31.46 cm	34	1019 s	31.57 cm	59	1769 s	31.66 cm	84	2519 s	31.71 cm
10	298 s	31.47 cm	35	1049 s	31.58 cm	60	1798 s	31.67 cm	85	2549 s	31.72 cm
11	329 s	31.47 cm	36	1078 s	31.58 cm	61	1829 s	31.67 cm	86	2578 s	31.73 cm
12	358 s	31.48 cm	37	1109 s	31.58 cm	62	1859 s	31.68 cm	87	2609 s	31.74 cm
13	389 s	31.49 cm	38	1138 s	31.59 cm	63	1888 s	31.68 cm	88	2638 s	31.75 cm
14	418 s	31.5 cm	39	1169 s	31.59 cm	64	1919 s	31.69 cm	89	2669 s	31.75 cm
15	449 s	31.51 cm	40	1198 s	31.59 cm	65	1948 s	31.69 cm	90	2699 s	31.77 cm
16	479 s	31.51 cm	41	1229 s	31.59 cm	66	1979 s	31.69 cm	91	2728 s	31.75 cm
17	509 s	31.52 cm	42	1258 s	31.61 cm	67	2009 s	31.7 cm	92	2759 s	31.77 cm
18	539 s	31.52 cm	43	1289 s	31.61 cm	68	2038 s	31.7 cm	93	2788 s	31.77 cm
19	568 s	31.53 cm	44	1319 s	31.61 cm	69	2069 s	31.71 cm	94	2819 s	31.77 cm
20	599 s	31.53 cm	45	1348 s	31.62 cm	70	2098 s	31.71 cm	95	2849 s	31.78 cm
21	628 s	31.53 cm	46	1379 s	31.62 cm	71	2129 s	31.71 cm	96	2878 s	31.78 cm
22	659 s	31.53 cm	47	1408 s	31.62 cm	72	2159 s	31.72 cm	97	2909 s	31.78 cm
23	688 s	31.54 cm	48	1439 s	31.63 cm	73	2188 s	31.72 cm	98	2939 s	31.78 cm
24	719 s	31.54 cm	49	1469 s	31.63 cm	74	2219 s	31.72 cm	99	2968 s	31.78 cm
25	749 s	31.54 cm	50	1498 s	31.63 cm	75	2248 s	31.73 cm	100	2999 s	31.78 cm

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1mpd1 Readings continued

#	Time	Head
101	3028 s	31.79 cm

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1mpd5

Date	4/9/2024
Time	12:26 PM
Latitude	38.140563
Longitude	-122.217133
Initial Volumetric Moisture	10.00 %
Final Volumetric Moisture	70.00 %
Cylinder Size	3 Liter

1mpd5 Results

Map Pin #	4
Test Number	27671
Ksat - mm/hr	NULL
Ksat - in/hr	NULL
Capillary Pressure C mm	NULL
RMS Error of Regression	NULL
Normalized RMS	NULL

Readings

#	Time	Head
1	0 s	32.37 cm
2	29 s	32.15 cm
3	59 s	32.03 cm
4	89 s	31.91 cm
5	119 s	31.82 cm
6	149 s	31.72 cm
7	179 s	31.63 cm
8	209 s	31.54 cm
9	239 s	31.46 cm
10	269 s	31.38 cm
11	299 s	31.3 cm
12	329 s	31.22 cm
13	359 s	31.14 cm
14	389 s	31.07 cm
15	419 s	31.0 cm
16	449 s	30.92 cm
17	479 s	30.85 cm
18	509 s	30.77 cm
19	539 s	30.69 cm
20	569 s	30.6 cm
21	599 s	30.53 cm
22	629 s	30.46 cm
23	659 s	30.38 cm
24	689 s	30.3 cm
25	719 s	30.22 cm

#	Time	Head
26	749 s	30.15 cm
27	779 s	30.07 cm
28	809 s	29.99 cm
29	839 s	29.91 cm
30	869 s	29.84 cm
31	899 s	29.76 cm
32	929 s	29.69 cm
33	959 s	29.61 cm
34	989 s	29.54 cm
35	1019 s	29.46 cm
36	1049 s	29.39 cm
37	1079 s	29.33 cm
38	1109 s	29.25 cm
39	1139 s	29.18 cm
40	1169 s	29.1 cm
41	1199 s	29.03 cm
42	1229 s	28.96 cm
43	1259 s	28.89 cm
44	1289 s	28.8 cm
45	1319 s	28.74 cm
46	1349 s	28.67 cm
47	1379 s	28.59 cm
48	1409 s	28.52 cm
49	1439 s	28.45 cm
50	1469 s	28.38 cm

#	Time	Head
51	1499 s	28.31 cm
52	1529 s	28.24 cm
53	1559 s	28.18 cm
54	1589 s	28.11 cm
55	1619 s	28.05 cm
56	1649 s	27.97 cm
57	1679 s	27.91 cm
58	1709 s	27.85 cm
59	1739 s	27.78 cm
60	1769 s	27.72 cm
61	1799 s	27.65 cm
62	1829 s	27.6 cm
63	1859 s	27.54 cm
64	1889 s	27.47 cm
65	1919 s	27.42 cm
66	1949 s	27.35 cm
67	1979 s	27.28 cm
68	2009 s	27.22 cm
69	2039 s	27.16 cm
70	2069 s	27.07 cm
71	2099 s	27.02 cm
72	2129 s	26.95 cm
73	2159 s	26.89 cm
74	2189 s	26.82 cm
75	2219 s	26.76 cm

#	Time	Head
76	2249 s	26.71 cm
77	2279 s	26.64 cm
78	2309 s	26.58 cm
79	2339 s	26.51 cm
80	2369 s	26.46 cm
81	2399 s	26.39 cm
82	2429 s	26.33 cm
83	2459 s	26.27 cm
84	2489 s	26.21 cm
85	2519 s	26.14 cm
86	2549 s	26.08 cm
87	2579 s	26.02 cm
88	2609 s	25.96 cm
89	2639 s	25.9 cm
90	2669 s	25.84 cm
91	2699 s	25.79 cm
92	2729 s	25.73 cm
93	2759 s	25.67 cm
94	2789 s	25.61 cm
95	2819 s	25.55 cm
96	2849 s	25.49 cm
97	2879 s	25.43 cm
98	2909 s	25.38 cm
99	2939 s	25.31 cm
100	2969 s	25.25 cm

1mpd5 Readings continued

#	Time	Head	#	Time	Head
101	2999 s	25.19 cm	133	3959 s	23.4 cm
102	3029 s	25.14 cm	134	3989 s	23.35 cm
103	3059 s	25.08 cm	135	4019 s	23.3 cm
104	3089 s	25.02 cm	136	4049 s	23.25 cm
105	3119 s	24.97 cm	137	4079 s	23.19 cm
106	3149 s	24.91 cm	138	4109 s	23.14 cm
107	3179 s	24.85 cm			
108	3209 s	24.79 cm			
109	3239 s	24.74 cm			
110	3269 s	24.67 cm			
111	3299 s	24.62 cm			
112	3329 s	24.57 cm			
113	3359 s	24.5 cm			
114	3389 s	24.45 cm			
115	3419 s	24.4 cm			
116	3449 s	24.34 cm			
117	3479 s	24.29 cm			
118	3509 s	24.22 cm			
119	3539 s	24.17 cm			
120	3569 s	24.12 cm			
121	3599 s	24.07 cm			
122	3629 s	24.01 cm			
123	3659 s	23.96 cm			
124	3689 s	23.89 cm			
125	3719 s	23.84 cm			
126	3749 s	23.79 cm			
127	3779 s	23.74 cm			
128	3809 s	23.67 cm			
129	3839 s	23.62 cm			
130	3869 s	23.56 cm			
131	3899 s	23.51 cm			
132	3929 s	23.46 cm			

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1mpd2

Date	4/9/2024
Time	1:46 PM
Latitude	38.139652
Longitude	-122.216595
Initial Volumetric Moisture	10.00 %
Final Volumetric Moisture	90.00 %
Cylinder Size	3 Liter

1mpd2 Results

Map Pin #	5
Test Number	27672
Ksat - mm/hr	27
Ksat - in/hr	1.05
Capillary Pressure C mm	-84.2
RMS Error of Regression	1.8
Normalized RMS	0.3%

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	29.75 cm	26	748 s	25.77 cm	51	1498 s	22.67 cm	76	2248 s	19.95 cm
2	28 s	29.52 cm	27	778 s	25.64 cm	52	1528 s	22.55 cm	77	2278 s	19.85 cm
3	58 s	29.3 cm	28	808 s	25.5 cm	53	1558 s	22.45 cm	78	2308 s	19.75 cm
4	88 s	29.11 cm	29	838 s	25.38 cm	54	1588 s	22.33 cm	79	2338 s	19.65 cm
5	118 s	28.92 cm	30	868 s	25.25 cm	55	1618 s	22.22 cm	80	2368 s	19.55 cm
6	148 s	28.74 cm	31	898 s	25.12 cm	56	1648 s	22.11 cm	81	2398 s	19.44 cm
7	178 s	28.56 cm	32	928 s	24.98 cm	57	1678 s	21.99 cm	82	2428 s	19.35 cm
8	208 s	28.39 cm	33	958 s	24.85 cm	58	1708 s	21.88 cm	83	2458 s	19.23 cm
9	238 s	28.24 cm	34	988 s	24.73 cm	59	1738 s	21.76 cm	84	2488 s	19.13 cm
10	268 s	28.09 cm	35	1018 s	24.6 cm	60	1768 s	21.66 cm	85	2518 s	19.03 cm
11	298 s	27.94 cm	36	1048 s	24.47 cm	61	1798 s	21.55 cm	86	2548 s	18.93 cm
12	328 s	27.79 cm	37	1078 s	24.34 cm	62	1828 s	21.43 cm	87	2578 s	18.84 cm
13	358 s	27.65 cm	38	1108 s	24.22 cm	63	1858 s	21.33 cm	88	2608 s	18.73 cm
14	388 s	27.51 cm	39	1138 s	24.11 cm	64	1888 s	21.22 cm	89	2638 s	18.63 cm
15	418 s	27.36 cm	40	1168 s	23.98 cm	65	1918 s	21.12 cm	90	2668 s	18.54 cm
16	448 s	27.22 cm	41	1198 s	23.85 cm	66	1948 s	21.01 cm	91	2698 s	18.44 cm
17	478 s	27.07 cm	42	1228 s	23.74 cm	67	1978 s	20.9 cm	92	2728 s	18.34 cm
18	508 s	26.93 cm	43	1258 s	23.62 cm	68	2008 s	20.8 cm	93	2758 s	18.25 cm
19	538 s	26.78 cm	44	1288 s	23.49 cm	69	2038 s	20.69 cm	94	2788 s	18.14 cm
20	568 s	26.64 cm	45	1318 s	23.37 cm	70	2068 s	20.58 cm	95	2818 s	18.05 cm
21	598 s	26.49 cm	46	1348 s	23.26 cm	71	2098 s	20.48 cm	96	2848 s	17.95 cm
22	628 s	26.34 cm	47	1378 s	23.14 cm	72	2128 s	20.37 cm	97	2878 s	17.85 cm
23	658 s	26.2 cm	48	1408 s	23.02 cm	73	2158 s	20.26 cm	98	2908 s	17.76 cm
24	688 s	26.06 cm	49	1438 s	22.9 cm	74	2188 s	20.16 cm	99	2938 s	17.66 cm
25	718 s	25.92 cm	50	1468 s	22.79 cm	75	2218 s	20.05 cm	100	2968 s	17.57 cm

1mpd2 Readings continued

#	Time	Head
101	2998 s	17.47 cm
102	3028 s	17.37 cm

MPD INFILTROMETER
www.upstreamtechnologies.us

Infiltration Report

Engeo San Ramon

Scott's Valley - 16484.000.001 - Vallejo, CA

Upstream Technologies
651.237.5123

1mpd6

Date	4/9/2024
Time	3:19 PM
Latitude	38.146098
Longitude	-122.214913
Initial Volumetric Moisture	30.00 %
Final Volumetric Moisture	80.00 %
Cylinder Size	3 Liter

1mpd6 Results

Map Pin #	6
Test Number	27673
Ksat - mm/hr	NULL
Ksat - in/hr	NULL
Capillary Pressure C mm	NULL
RMS Error of Regression	NULL
Normalized RMS	NULL

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	26.89 cm	26	748 s	25.56 cm	51	1498 s	24.98 cm	76	2248 s	24.38 cm
2	28 s	26.79 cm	27	778 s	25.52 cm	52	1528 s	24.96 cm	77	2278 s	24.36 cm
3	58 s	26.72 cm	28	808 s	25.49 cm	53	1558 s	24.94 cm	78	2308 s	24.34 cm
4	88 s	26.62 cm	29	838 s	25.47 cm	54	1588 s	24.92 cm	79	2338 s	24.31 cm
5	118 s	26.54 cm	30	868 s	25.44 cm	55	1618 s	24.9 cm	80	2368 s	24.29 cm
6	148 s	26.45 cm	31	898 s	25.42 cm	56	1648 s	24.89 cm	81	2398 s	24.26 cm
7	178 s	26.37 cm	32	928 s	25.39 cm	57	1678 s	24.86 cm	82	2428 s	24.24 cm
8	208 s	26.29 cm	33	958 s	25.36 cm	58	1708 s	24.84 cm	83	2458 s	24.21 cm
9	238 s	26.24 cm	34	988 s	25.33 cm	59	1738 s	24.82 cm	84	2488 s	24.19 cm
10	268 s	26.17 cm	35	1018 s	25.31 cm	60	1768 s	24.81 cm	85	2518 s	24.18 cm
11	298 s	26.13 cm	36	1048 s	25.29 cm	61	1798 s	24.79 cm	86	2548 s	24.14 cm
12	328 s	26.06 cm	37	1078 s	25.27 cm	62	1828 s	24.76 cm	87	2578 s	24.11 cm
13	358 s	26.02 cm	38	1108 s	25.25 cm	63	1858 s	24.75 cm	88	2608 s	24.09 cm
14	388 s	25.99 cm	39	1138 s	25.22 cm	64	1888 s	24.73 cm	89	2638 s	24.07 cm
15	418 s	25.95 cm	40	1168 s	25.2 cm	65	1918 s	24.71 cm	90	2668 s	24.04 cm
16	448 s	25.91 cm	41	1198 s	25.18 cm	66	1948 s	24.69 cm	91	2698 s	24.02 cm
17	478 s	25.88 cm	42	1228 s	25.16 cm	67	1978 s	24.67 cm	92	2728 s	24.0 cm
18	508 s	25.83 cm	43	1258 s	25.14 cm	68	2008 s	24.65 cm	93	2758 s	23.98 cm
19	538 s	25.8 cm	44	1288 s	25.12 cm	69	2038 s	24.63 cm	94	2788 s	23.95 cm
20	568 s	25.76 cm	45	1318 s	25.1 cm	70	2068 s	24.58 cm	95	2818 s	23.93 cm
21	598 s	25.73 cm	46	1348 s	25.08 cm	71	2098 s	24.53 cm	96	2848 s	23.91 cm
22	628 s	25.68 cm	47	1378 s	25.06 cm	72	2128 s	24.5 cm	97	2878 s	23.88 cm
23	658 s	25.65 cm	48	1408 s	25.03 cm	73	2158 s	24.47 cm	98	2908 s	23.86 cm
24	688 s	25.62 cm	49	1438 s	25.01 cm	74	2188 s	24.44 cm	99	2938 s	23.83 cm
25	718 s	25.59 cm	50	1468 s	25.0 cm	75	2218 s	24.42 cm	100	2968 s	23.82 cm

1mpd6 Readings continued

#	Time	Head	#	Time	Head
101	2998 s	23.8 cm	133	3958 s	23.46 cm
102	3028 s	23.78 cm	134	3988 s	23.43 cm
103	3058 s	23.76 cm	135	4018 s	23.42 cm
104	3088 s	23.74 cm	136	4048 s	23.4 cm
105	3118 s	23.71 cm	137	4078 s	23.38 cm
106	3148 s	23.69 cm	138	4108 s	23.35 cm
107	3178 s	23.67 cm	139	4138 s	23.34 cm
108	3208 s	23.66 cm	140	4168 s	23.33 cm
109	3238 s	23.63 cm	141	4198 s	23.31 cm
110	3268 s	23.62 cm	142	4228 s	23.29 cm
111	3298 s	23.6 cm	143	4258 s	23.28 cm
112	3328 s	23.58 cm	144	4288 s	23.26 cm
113	3358 s	23.65 cm	145	4318 s	23.25 cm
114	3388 s	23.65 cm	146	4348 s	23.22 cm
115	3418 s	23.66 cm	147	4378 s	23.2 cm
116	3448 s	23.66 cm	148	4408 s	23.17 cm
117	3478 s	23.66 cm	149	4438 s	23.14 cm
118	3508 s	23.65 cm	150	4468 s	23.11 cm
119	3538 s	23.64 cm	151	4498 s	23.07 cm
120	3568 s	23.63 cm	152	4528 s	23.03 cm
121	3598 s	23.61 cm	153	4558 s	23.0 cm
122	3628 s	23.6 cm	154	4588 s	22.98 cm
123	3658 s	23.61 cm	155	4618 s	22.95 cm
124	3688 s	23.61 cm	156	4648 s	22.91 cm
125	3718 s	23.59 cm	157	4678 s	22.89 cm
126	3748 s	23.58 cm	158	4708 s	22.84 cm
127	3778 s	23.55 cm	159	4738 s	22.8 cm
128	3808 s	23.54 cm			
129	3838 s	23.53 cm			
130	3868 s	23.51 cm			
131	3898 s	23.49 cm			
132	3928 s	23.47 cm			

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APPENDIX C

Acorn Environmental
Water and Wastewater Feasibility Study
ENGEO, Inc. Hydrogeologic Assessment

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Project No.
16484.000.001

May 2, 2024

Ms. Bibiana Sparks
Acorn Environmental
5170 Golden Foothill Parkway
El Dorado Hills, CA 95762

Subject: Scotts Valley Development
Admiral Callaghan Lane and Columbus Parkway
Vallejo, California

HYDROGEOLOGIC ASSESSMENT

Dear Ms. Sparks:

At your request, we have prepared this hydrogeologic assessment for the Scotts Valley Development in Vallejo, California. The purpose of this report is to assess the existing sources of groundwater at the site for potential use within the project.

Our scope of services included the following items.

- Research and review of relevant and available data for the site, including:
  - published geologic maps,
  - groundwater reports prepared by California Department of Water Resources (DWR),
  - available well records and reports from DWR and local agencies, and
  - published Caltrans records of Hunter Hill Landslide and associated drainage gallery.
- Characterization of surface and subsurface geology based on site exploration and published geologic maps
- Field reconnaissance of springs
- Preparation of this report

DOCUMENT REVIEW

Hunter Hill Landslide

An existing landslide, called the Hunter Hill landslide, is located on the northwestern portion of the site. The landslide crosses Interstate 80 (I-80), and is estimated to be approximately 1,300 feet long, 600 feet wide, and approximately 60 feet deep. Ongoing roadway distress has been documented due to continued movement of the landslide. Inclinometers installed by Caltrans near the slide showed movement below I-80 at approximately 30 feet below the roadway surface between 2003 and 2005 (Caltrans, 2005).

According to documentation by Caltrans, a vertical drainage gallery was partially constructed in 1990 through the existing landslide above I-80 in order to reduce water pressures in the landslide, at the approximate location shown in Exhibit 1. The drainage gallery was to consist of vertical sand drains 3 feet in diameter, approximately 53 feet deep, and spaced at 6 feet on-center, interconnected at the bottom by overlapping bells. The gallery was intended to be drained to the southwest under I-80 by a horizontal perforated pipe (Caltrans, 1988).

We did not observe the drainage gallery during our site reconnaissance. According to Caltrans documentation, the bottom drain from the drainage gallery was never completed due to the presence of hard rock and difficult drilling conditions. Additionally, the final constructed depth and extents of the vertical wells is not known since construction was terminated before project completion (Caltrans 1990a, 1990b). Therefore, an elevated water table may still be present in this area of the slide. Groundwater depth fluctuates between approximately 10 and 14 feet below ground surface near the gallery (Caltrans, 2005).

Existing Wells

Based on our review of the available DWR Well Completion Report (WCR) database, no groundwater wells were identified on the site or within a ½ mile radius of the site.

Napa-Sonoma Lowlands Subbasin

The site is located in upland bedrock terrain and outside of a designated groundwater basin. The site lies about $^{1}/_{3}$ mile east of the eastern boundary of the Napa-Sonoma Lowlands Groundwater Subbasin. The typical “water bearing formations” in the basin include Holocene and Pleistocene Alluvium, and Pleistocene Huichica Formation. We encountered Pleistocene alluvium and colluvium during our explorations to depths of up to 13 feet. The local groundwater conditions at the site would be characterized as fractured bedrock with an unknown water-bearing capacity within the Great Valley Sequence and silica-carbonate rock.

GEOLOGY

Our hydrogeologic characterization is based on our preliminary geotechnical exploration at the site. Geologic units encountered during our exploration include:

- **Artificial fill (af)** – In our explorations, artificial fill consists of bedrock-derived sand and gravel mixed with clay.
- **Alluvium and colluvium, undivided (Qa, Qc)** – Holocene and late Pleistocene deposits. In our explorations, this material generally consists of sandy and gravelly stiff to very stiff clay, with local lenses of increased sand and gravel fractions underlying surficial clay deposits.
- **Landslide Deposits (Qls)** – Holocene and Pleistocene deposits. Deposits near the north landslide (Hunter Hill Landslide) consisted primarily of gravelly lean clay and highly sheared shale and sandstone. Deposits near the south landslide consisted of sheared shale and mudstone in a clay matrix.
- **Great Valley Sequence (Kgv)** – Cretaceous age sandstone, siltstone, shale, and minor conglomerates. On the project site, this unit predominantly consists of siltstone and shale with minor sandstone.
- **Silica-Carbonate Rock (sc)** – Part of the Jurassic-age Coast Range Ophiolite sequence, which contains basalt, gabbro, and serpentinite. Serpentinite locally contains pyroxenite and silica-carbonate rock.

GROUNDWATER

During our field exploration, we encountered groundwater in one of our borings (1-B2) at a depth of 14 feet below the existing ground surface within Great Valley Sequence rock. Water was not encountered in Boring 1-B3 to final depth of the boring (60 feet). The depth to groundwater was not identified in Boring 1-B1 due to the drilling methods used. We also observed surface water flowing in small streams at the locations shown in blue in Exhibit 1. Reports from Caltrans indicate that groundwater depths near the drainage gallery (shown in Exhibit 1) fluctuate seasonally between approximately 10 to 14 feet (Caltrans, 2005).

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

FIELD RECONNAISSANCE OF SPRINGS

Four springs are present on or near the project site, as shown in Exhibit 1 – Site Plan. During our field exploration between April 22 and April 25, 2024, we performed a reconnaissance of the springs to assess their current condition. In a channel flowing from the easternmost spring, we estimated flow rates at three locations that ranged from ¼ gallon per minute (gpm) to 2½ gpm. Additionally, we observed water flowing from a culvert out of the southernmost spring at a rate of approximately 3 gpm. We consider these field estimates to be preliminary, and not representative of the total flow from the springs.

We also reviewed aerial imagery available on Google Earth from 1993 to 2023 to understand and estimate the seasonal fluctuation in flow from the springs. The streams are generally more active during winter and spring months and have a reduced vegetated area during summer and fall months, especially during drought years. Dry or drought conditions are evident in aerial imagery from May 2022, September 2010, and July 1993, as shown in Appendix A.

EXHIBIT 1: Site Plan

CONCLUSIONS

Water sources present on the site include surface water, four springs located along the boundaries of existing landslides and at geologic contacts, groundwater within alluvium and colluvium soil layers, and groundwater within fractured bedrock.

We note the following considerations regarding using water from these sources.

- Groundwater supply wells are not located on the project site or nearby. Our research did not identify previous well pump tests conducted in either soil or rock units on or near the site. It is also not known whether fractures throughout the Great Valley rock and silica-carbonate rock will provide sufficient flow to develop groundwater supply wells. Therefore, the potential yield of these materials is uncertain.
- The output from the springs is not known, although seasonal fluctuation and drought periods will result in reduced spring flow.
- The depth of colluvium and alluvium at the site is variable. In our explorations, we identified colluvium/alluvium thicknesses ranging from 3 to 13 feet, with alluvium and colluvium deposits covering approximately one quarter of the site. The lateral continuity or presence of groundwater in these deposits is unknown.
- Colluvium contains high concentrations of clay which may result in low yield conditions. We did not encounter continuous layers of sand or gravel in our explorations.
- Historical mercury mining operations were present at multiple locations near the site, including St. John's Mine located less than 1 mile northeast of the site. We consider it feasible that groundwater from both upper soil units and deeper bedrock in this area may be contaminated with heavy metals due to the historical mining operations and possible flow of water through rocks containing heavy metals.

If you have any questions or comments regarding this letter, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Anne Robertson, PE   James Thurber, CEG

awr/jet/ca

Attachments: Selected References
Appendix A

SELECTED REFERENCES

1. California Department of Water Resources (DWR). 2024. Online System for Well Completion Reports.

2. Caltrans. 2005. Memorandum: Geotechnical Recommendation for Roadway Rehab Project, File No. 04-SOL-80, KP 6.3-13.0/PM 3.9-8.1.

3. Caltrans. 1990a. Memorandum: Results of Field Investigation and Decision regarding Future of Project, File No. 10-339203, 10-SOL-80, PM 6.3.

4. Caltrans. 1990b. Memorandum: Field Investigation for Redesign of Project, File No. 10-339203, 10-SOL-80, PM 6.4.

5. Caltrans. 1988. Memorandum: Seismic Investigation of the Hunter Hill Slide near Vallejo, File No. 10-5S6000, 10-SOL-80-6.0.

6. California Department of Water Resources (DWR). 2014. Bulletin 118, Napa-Sonoma Valley groundwater Basin, Napa-Sonoma Lowlands Subbasin.

APPENDIX A

APPENDIX A

AERIAL PHOTO REVIEW

PHOTO A-1: Google Earth Imagery, August 2023, Summer Conditions Following Historical Winter and Spring Rainfall

PHOTO A-2: Google Earth Imagery, May 2023, Spring Conditions Following Historical Rainfall

PHOTO A-3: Google Earth Imagery, May 2022, Spring Conditions Following 10+ Year Drought

PHOTO A-4: Google Earth Imagery, October 2020, Fall Conditions Following Second Driest October on Record in California and 8+ Year Drought

PHOTO A-5: Google Earth Imagery, September 2018, Fall Conditions Following Sixth Driest September on Record in California

PHOTO A-6: Google Earth Imagery, August 2014, Summer Conditions after a Severely Dry Month, and at Beginning of Exceptional Drought Levels

PHOTO A-7: Google Earth Imagery, September 2010, Fall Conditions Following 3+ Year Drought

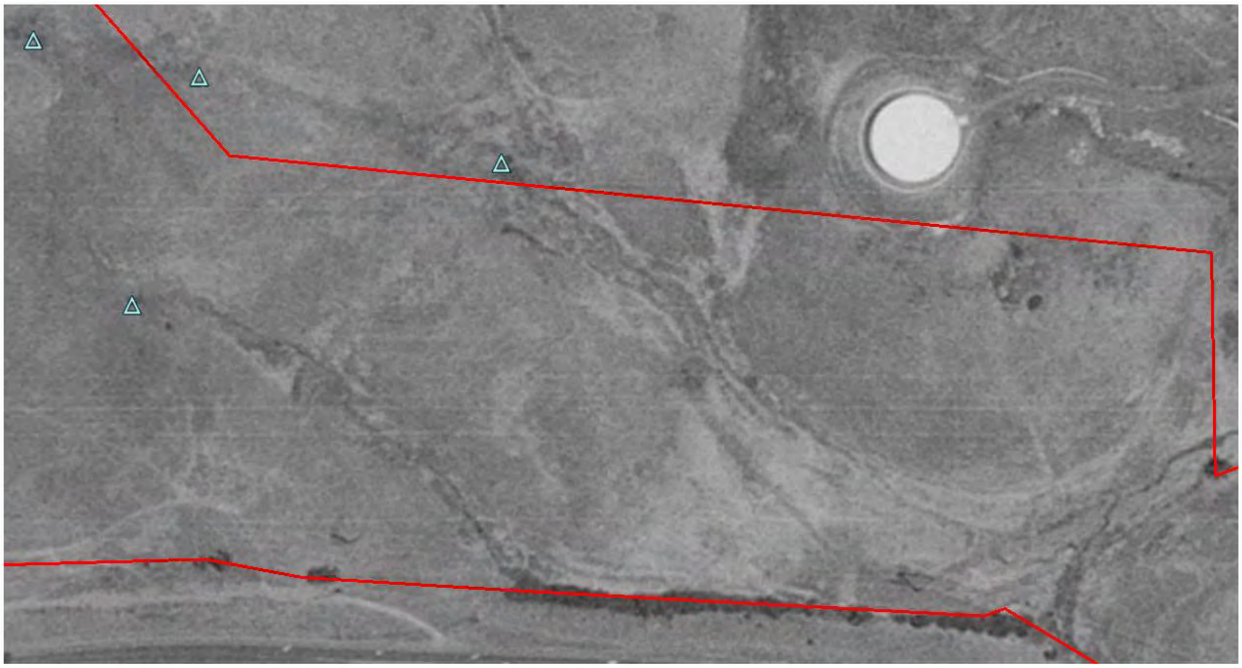
PHOTO A-8: Google Earth Imagery, May 2008, Summer Conditions Following One Year of Extreme Drought

PHOTO A-9: Google Earth Imagery, August 2004, Summer Conditions Following 3+ Year Drought

PHOTO A-10: Google Earth Imagery, July 2003, Summer Conditions Amid Extreme Drought

PHOTO A-11: Google Earth Imagery, July 2002, Summer Conditions Amid Extreme Drought

PHOTO A-12: Google Earth Imagery, July 1993, Summer Conditions Following 6+ Year Drought from 1986 to 1992



APPENDIX D
Acorn Environmental
Water and Wastewater Feasibility Study
City of Vallejo Standard Specifications and Standard Drawings

MICROSOFT WORD - 20110913 COV STD SPECS.DOCX (CITYOFVALLEJO.NET)

APPENDIX E

Acorn Environmental

Water and Wastewater Feasibility Study

Vallejo Flood and Wastewater District Engineering Standards

[ENGINEERING-DESIGN-STANDARDS---COMBINED-PDF (VALLEJOWASTEWATER.ORG)]

APPENDIX F

Acorn Environmental

Water and Wastewater Feasibility Study

Vallejo Flood and Wastewater District

Recycled Water Facilities Plan

[RECYCLED-WATER-FACILITIES-PLAN-2018 (VALLEJOWASTEWATER.ORG)]

APPENDIX G

Acorn Environmental

Water and Wastewater Feasibility Study

Project Alternative Water Balances

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Water Balance - Scotts Valley Feasibility Study - Proposed (Alternative A)

Scenario: Alternative A

June 2024 By: Jory Benitez/Angela Singer, HydroScience

INPUT

INPUT-Adjust as necessary

OUTPUT-Max Elevation

WASTEWATER INFLUENT FLOW			STORAGE DATA		OTHER INPUTS			RECYCLED WATER DISTRIBUTION AND DISPOSAL ALTERNATIVES[2]					
Daily Average Wastewater Influent Flow	215,000	gpd	Tank(s) Total Volume	21.0 MG	100-YR Multiplier	1.81	unitless	Landscape Irrigation	5.0	acres	Other Irrig (TBD)	189.0	acres
I/I (PWWF-PDWF)	-	gpd			Pan Evap Coefficient	0.75	unitless	Dual Plumbing	20.4	MG			

100-YEAR ANNUAL PRECIPITATION RETURN PERIOD

	Units	October	November	December	January	February	March	April	May	June	July	August	September	Water Year
No. Days		31	30	31	31	28	31	30	31	30	31	31	30	
CLIMATE INPUTS														
Precipitation	in	1.83	4.43	10.29	10.07	9.33	6.11	2.23	1.42	0.40	0.00	0.11	0.24	46.45
Pan Evaporation	in	5.72	2.48	1.66	1.53	2.15	3.79	5.82	8.90	11.00	13.22	12.06	8.67	77.00
Effective Water Surface Evaporation	in	4.29	1.40	0.93	0.86	1.21	2.13	4.37	6.68	8.25	9.92	9.05	6.50	55.57
WASTEWATER GENERATION														
Facility Wastewater Influent (ADWF)	MG	6.7	6.5	6.7	6.7	6.0	6.7	6.5	6.7	6.5	6.7	6.7	6.5	78.5
I/I Contributions	MG	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.1
TOTAL Wastewater Influent	ac-ft	20.5	19.8	20.5	20.5	18.5	20.5	19.8	20.5	19.8	20.5	20.5	19.8	241.1
WWTP CONTRIBUTIONS														
Site Run-off	ac-ft	0.0	0.1	0.2	0.2	0.2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.8
Cooling Tower Blowdown	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cooling Tower Evaporation/Drift Loss	ac-ft	-0.04	-0.03	-0.03	-0.03	-0.03	-0.03	-0.04	-0.04	-0.04	-0.04	-0.04	-0.04	-0.4
RECYCLED WATER DISTRIBUTION														
Dual Plumbing	ac-ft	-5.3	-5.2	-5.3	-5.3	-4.8	-5.3	-5.2	-5.3	-5.2	-5.3	-5.3	-5.2	-62.7
Cooling Tower	ac-ft	-2.5	-2.0	-2.0	-2.0	-1.8	-2.0	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-27.5
Landscape Irrigation	ac-ft	-0.4	0.0	0.0	0.0	0.0	0.0	-0.3	-1.4	-2.3	-2.5	-2.3	-1.7	-10.9
Other Irrigation (TBD)	ac-ft	-14.9	0.0	0.0	0.0	0.0	0.0	-11.9	-51.8	-88.3	-95.1	-86.7	-64.6	-413.1
RAW WATER MAKE-UP														
Blend Raw Water[1]	ac-ft	2.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	54.5	85.0	76.4	54.2	272.8
MONTHLY STORAGE BALANCE														
Beginning Storage Volume	ac-ft	0.0	0.0	12.8	26.0	39.3	51.3	64.5	64.5	24.0	0.0	0.0	0.0	
Change in Water Volume	ac-ft	0.0	12.8	13.3	13.3	12.0	13.2	0.0	-40.5	-24.0	0.0	0.0	0.0	
Final Storage Volume	ac-ft	0.0	12.8	26.0	39.3	51.3	64.5	64.5	24.0	0.0	0.0	0.0	0.0	

Maximum Seasonal Storage (ac-ft) **64.5**
mg **21.0**

AVERAGE ANNUAL PRECIPITATION RETURN PERIOD

	Units	October	November	December	January	February	March	April	May	June	July	August	September	Water Year
No. Days		31	30	31	31	28	31	30	31	30	31	31	30	
CLIMATE INPUTS														
Precipitation	in	1.01	2.44	5.67	5.55	5.14	3.37	1.23	0.78	0.22	0.00	0.06	0.13	25.60
Pan Evaporation	in	5.72	2.48	1.66	1.53	2.15	3.79	5.82	8.90	11.00	13.22	12.06	8.67	77.00
Effective Water Surface Evaporation	in	4.29	1.86	1.25	1.15	1.61	2.84	4.37	6.68	8.25	9.92	9.05	6.50	57.75
WASTEWATER GENERATION														
Facility Wastewater Influent (ADWF)	MG	6.7	6.5	6.7	6.7	6.0	6.7	6.5	6.7	6.5	6.7	6.7	6.5	78.5
I/I Contributions	MG	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.1
TOTAL Wastewater Influent	ac-ft	20.5	19.8	20.5	20.5	18.5	20.5	19.8	20.5	19.8	20.5	20.5	19.8	241.1
WWTP CONTRIBUTIONS														
Site Run-off	ac-ft	0.0	0.0	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.5
Cooling Tower Blowdown	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cooling Tower Evaporation/Drift Loss	ac-ft	-0.04	-0.04	-0.04	-0.04	-0.03	-0.04	-0.04	-0.04	-0.04	-0.04	-0.04	-0.04	-0.4
RECYCLED WATER DISTRIBUTION														
Dual Plumbing	ac-ft	-5.3	-5.2	-5.3	-5.3	-4.8	-5.3	-5.2	-5.3	-5.2	-5.3	-5.3	-5.2	-62.7
Cooling Tower	ac-ft	-2.5	-2.5	-2.5	-2.5	-2.3	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-30.0
Landscape Irrigation	ac-ft	-0.8	0.0	0.0	0.0	0.0	0.0	-0.8	-1.7	-2.4	-2.5	-2.3	-1.8	-12.4
Other Irrigation (TBD)	ac-ft	-31.0	0.0	0.0	0.0	0.0	0.0	-31.5	-64.2	-91.8	-95.1	-87.6	-66.7	-468.0
RAW WATER MAKE-UP														
Blend Raw Water[1]	ac-ft	19.3	0.0	0.0	0.0	0.0	0.0	0.0	11.9	82.1	85.0	77.4	56.3	332.0
MONTHLY STORAGE BALANCE														
Beginning Storage Volume	ac-ft	0.0	0.0	12.2	24.9	37.6	49.0	61.7	41.5	0.0	0.0	0.0	0.0	
Change in Water Volume	ac-ft	0.0	12.2	12.7	12.7	11.5	12.6	-20.2	-41.5	0.0	0.0	0.0	0.0	
Final Storage Volume	ac-ft	0.0	12.2	24.9	37.6	49.0	61.7	41.5	0.0	0.0	0.0	0.0	0.0	

Maximum Seasonal Storage (ac-ft) **61.7**
mg **20.1**

Note:
1. Blend Raw Water is the deficit in ww flow generated to meet recycled water demands, to resolve then less water would be discharged for irrigation.
2. Assumed all equipment open basin/tankage would include covers and won't contribute to ww flows, confirm as more information becomes available.
3. Cooling tower blowdown is estimated at 10% of daily water demand and is included in the facility wastewater influent projection.
4. Cooling tower evaporation loss estimated at 1.5% of monthly water demand.

Water Balance - Scotts Valley Feasibility Study - Proposed (Alternative B)

Scenario: Alternative B

June 2024 By: Jory Benitez/Angela Singer, HydroScience

INPUT

INPUT-Adjust as necessary

OUTPUT-Max Elevation

WASTEWATER INFLUENT FLOW			STORAGE DATA		OTHER INPUTS		RECYCLED WATER DISTRIBUTION AND DISPOSAL ALTERNATIVES[2]			
Daily Average Wastewater Influent Flow	208,000	gpd	Tank(s) Total Volume	20.0 MG	100-YR Multiplier	1.81 unitless	Landscape Irrigation	5.0 acres	Other Irrig (TBD)	180.0 acres
I/I (PWWF-PDWF)	-	gpd			Pan Evap Coefficient	0.75 unitless	Dual Plumbing	20.4 MG		

100-YEAR ANNUAL PRECIPITATION RETURN PERIOD

	Units	October	November	December	January	February	March	April	May	June	July	August	September	Water Year
No. Days		31	30	31	31	28	31	30	31	30	31	31	30	
CLIMATE INPUTS														
Precipitation	in	1.83	4.43	10.29	10.07	9.33	6.11	2.23	1.42	0.40	0.00	0.11	0.24	46.45
Pan Evaporation	in	5.72	2.48	1.66	1.53	2.15	3.79	5.82	8.90	11.00	13.22	12.06	8.67	77.00
Effective Water Surface Evaporation	in	4.29	1.40	0.93	0.86	1.21	2.13	4.37	6.68	8.25	9.92	9.05	6.50	55.57
WASTEWATER GENERATION														
Facility Wastewater Influent (ADWF)	MG	6.4	6.2	6.4	6.4	5.8	6.4	6.2	6.4	6.2	6.4	6.4	6.2	75.9
I/I Contributions	MG	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.1
TOTAL Wastewater Influent	ac-ft	19.8	19.2	19.8	19.8	17.9	19.8	19.2	19.8	19.2	19.8	19.8	19.2	233.2
WWTP CONTRIBUTIONS														
Site Run-off	ac-ft	0.0	0.1	0.2	0.2	0.2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.8
Cooling Tower Blowdown	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cooling Tower Evaporation/Drift Loss	ac-ft	-0.04	-0.03	-0.03	-0.03	-0.03	-0.03	-0.04	-0.04	-0.04	-0.04	-0.04	-0.04	-0.4
RECYCLED WATER DISTRIBUTION														
Dual Plumbing	ac-ft	-5.3	-5.2	-5.3	-5.3	-4.8	-5.3	-5.2	-5.3	-5.2	-5.3	-5.3	-5.2	-62.7
Cooling Tower	ac-ft	-2.5	-2.0	-2.0	-2.0	-1.8	-2.0	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-27.5
Landscape Irrigation	ac-ft	-0.4	0.0	0.0	0.0	0.0	0.0	-0.3	-1.4	-2.3	-2.5	-2.3	-1.7	-10.9
Other Irrigation (TBD)	ac-ft	-14.2	0.0	0.0	0.0	0.0	0.0	-11.3	-49.3	-84.1	-90.5	-82.5	-61.5	-393.5
RAW WATER MAKE-UP														
Blend Raw Water[1]	ac-ft	2.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	52.5	81.2	72.9	51.8	261.0
MONTHLY STORAGE BALANCE														
Beginning Storage Volume	ac-ft	0.0	0.0	12.1	24.7	37.3	48.7	61.2	61.2	22.5	0.0	0.0	0.0	
Change in Water Volume	ac-ft	0.0	12.1	12.6	12.6	11.4	12.5	0.0	-38.7	-22.5	0.0	0.0	0.0	
Final Storage Volume	ac-ft	0.0	12.1	24.7	37.3	48.7	61.2	61.2	22.5	0.0	0.0	0.0	0.0	

Maximum Seasonal Storage (ac-ft) **61.2**
mg **20.0**

AVERAGE ANNUAL PRECIPITATION RETURN PERIOD

	Units	October	November	December	January	February	March	April	May	June	July	August	September	Water Year
No. Days		31	30	31	31	28	31	30	31	30	31	31	30	
CLIMATE INPUTS														
Precipitation	in	1.01	2.44	5.67	5.55	5.14	3.37	1.23	0.78	0.22	0.00	0.06	0.13	25.60
Pan Evaporation	in	5.72	2.48	1.66	1.53	2.15	3.79	5.82	8.90	11.00	13.22	12.06	8.67	77.00
Effective Water Surface Evaporation	in	4.29	1.86	1.25	1.15	1.61	2.84	4.37	6.68	8.25	9.92	9.05	6.50	57.75
WASTEWATER GENERATION														
Facility Wastewater Influent (ADWF)	MG	6.4	6.2	6.4	6.4	5.8	6.4	6.2	6.4	6.2	6.4	6.4	6.2	75.9
I/I Contributions	MG	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.1
TOTAL Wastewater Influent	ac-ft	19.8	19.2	19.8	19.8	17.9	19.8	19.2	19.8	19.2	19.8	19.8	19.2	233.2
WWTP CONTRIBUTIONS														
Site Run-off	ac-ft	0.0	0.0	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.5
Cooling Tower Blowdown	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cooling Tower Evaporation/Drift Loss	ac-ft	-0.04	-0.04	-0.04	-0.04	-0.03	-0.04	-0.04	-0.04	-0.04	-0.04	-0.04	-0.04	-0.4
RECYCLED WATER DISTRIBUTION														
Dual Plumbing	ac-ft	-5.3	-5.2	-5.3	-5.3	-4.8	-5.3	-5.2	-5.3	-5.2	-5.3	-5.3	-5.2	-62.7
Cooling Tower	ac-ft	-2.5	-2.5	-2.5	-2.5	-2.3	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-30.0
Landscape Irrigation	ac-ft	-0.8	0.0	0.0	0.0	0.0	0.0	-0.8	-1.7	-2.4	-2.5	-2.3	-1.8	-12.4
Other Irrigation (TBD)	ac-ft	-29.5	0.0	0.0	0.0	0.0	0.0	-30.0	-61.2	-87.4	-90.5	-83.4	-63.5	-445.7
RAW WATER MAKE-UP														
Blend Raw Water[1]	ac-ft	18.5	0.0	0.0	0.0	0.0	0.0	0.0	11.9	78.4	81.2	73.9	53.8	317.5
MONTHLY STORAGE BALANCE														
Beginning Storage Volume	ac-ft	0.0	0.0	11.6	23.6	35.6	46.4	58.4	39.1	0.0	0.0	0.0	0.0	
Change in Water Volume	ac-ft	0.0	11.6	12.0	12.0	10.9	12.0	-19.3	-39.1	0.0	0.0	0.0	0.0	
Final Storage Volume	ac-ft	0.0	11.6	23.6	35.6	46.4	58.4	39.1	0.0	0.0	0.0	0.0	0.0	

Maximum Seasonal Storage (ac-ft) **58.4**
mg **19.0**

Note:
1. Blend Raw Water is the deficit in ww flow generated to meet recycled water demands, to resolve then less water would be discharged for irrigation.
2. Assumed all equipment open basin/tankage would include covers and won't contribute to ww flows, confirm as more information becomes available.
3. Cooling tower blowdown is estimated at 10% of daily water demand and is included in the facility wastewater influent projection.
4. Cooling tower evaporation loss estimated at 1.5% of monthly water demand.

Water Balance - Scotts Valley Feasibility Study - Proposed (Alternative C)

Scenario: Alternative C

June 2024 By: Jory Benitez/Angela Singer, HydroScience

INPUT

INPUT-Adjust as necessary

OUTPUT-Max Elevation

WASTEWATER INFLUENT FLOW			STORAGE DATA		OTHER INPUTS			RECYCLED WATER DISTRIBUTION AND DISPOSAL ALTERNATIVES[2]					
Daily Average Wastewater Influent Flow	62,000	gpd	Tank(s) Total Volume	7.0 MG	100-YR Multiplier	1.81	unitless	Landscape Irrigation	5.0	acres	Other Irrig (TBD)	58.5	acres
I/I (PWWF-PDWF)	-	gpd			Pan Evap Coefficient	0.75	unitless	Dual Plumbing	4.4	MG			

100-YEAR ANNUAL PRECIPITATION RETURN PERIOD

	Units	October	November	December	January	February	March	April	May	June	July	August	September	Water Year
No. Days		31	30	31	31	28	31	30	31	30	31	31	30	
CLIMATE INPUTS														
Precipitation	in	1.83	4.43	10.29	10.07	9.33	6.11	2.23	1.42	0.40	0.00	0.11	0.24	46.45
Pan Evaporation	in	5.72	2.48	1.66	1.53	2.15	3.79	5.82	8.90	11.00	13.22	12.06	8.67	77.00
Effective Water Surface Evaporation	in	4.29	1.40	0.93	0.86	1.21	2.13	4.37	6.68	8.25	9.92	9.05	6.50	55.57
WASTEWATER GENERATION														
Facility Wastewater Influent (ADWF)	MG	1.9	1.9	1.9	1.9	1.7	1.9	1.9	1.9	1.9	1.9	1.9	1.9	22.6
I/I Contributions	MG	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.1
TOTAL Wastewater Influent	ac-ft	5.9	5.7	5.9	5.9	5.4	5.9	5.7	5.9	5.7	5.9	5.9	5.7	69.7
WWTP CONTRIBUTIONS														
Site Run-off	ac-ft	0.0	0.0	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3
Cooling Tower Blowdown	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cooling Tower Evaporation/Drift Loss	ac-ft	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.1
RECYCLED WATER DISTRIBUTION														
Dual Plumbing	ac-ft	-1.1	-1.1	-1.1	-1.1	-1.0	-1.1	-1.1	-1.1	-1.1	-1.1	-1.1	-1.1	-13.4
Cooling Tower	ac-ft	-0.6	-0.5	-0.5	-0.5	-0.4	-0.5	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-6.3
Landscape Irrigation	ac-ft	-0.4	0.0	0.0	0.0	0.0	0.0	-0.3	-1.4	-2.3	-2.5	-2.3	-1.7	-10.9
Other Irrigation (TBD)	ac-ft	-4.6	0.0	0.0	0.0	0.0	0.0	-3.7	-16.0	-27.3	-29.4	-26.8	-20.0	-127.9
RAW WATER MAKE-UP														
Blend Raw Water[1]	ac-ft	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	17.5	27.8	24.9	17.7	88.7
MONTHLY STORAGE BALANCE														
Beginning Storage Volume	ac-ft	0.0	0.0	4.2	8.6	13.0	16.9	21.3	21.3	8.2	0.0	0.0	0.0	
Change in Water Volume	ac-ft	0.0	4.2	4.4	4.4	4.0	4.4	0.1	-13.2	-8.2	0.0	0.0	0.0	
Final Storage Volume	ac-ft	0.0	4.2	8.6	13.0	16.9	21.3	21.3	8.2	0.0	0.0	0.0	0.0	

Maximum Seasonal Storage (ac-ft) **21.3**
mg **7.0**

AVERAGE ANNUAL PRECIPITATION RETURN PERIOD

	Units	October	November	December	January	February	March	April	May	June	July	August	September	Water Year
No. Days		31	30	31	31	28	31	30	31	30	31	31	30	
CLIMATE INPUTS														
Precipitation	in	1.01	2.44	5.67	5.55	5.14	3.37	1.23	0.78	0.22	0.00	0.06	0.13	25.60
Pan Evaporation	in	5.72	2.48	1.66	1.53	2.15	3.79	5.82	8.90	11.00	13.22	12.06	8.67	77.00
Effective Water Surface Evaporation	in	4.29	1.86	1.25	1.15	1.61	2.84	4.37	6.68	8.25	9.92	9.05	6.50	57.75
WASTEWATER GENERATION														
Facility Wastewater Influent (ADWF)	MG	1.9	1.9	1.9	1.9	1.7	1.9	1.9	1.9	1.9	1.9	1.9	1.9	22.6
I/I Contributions	MG	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.1
TOTAL Wastewater Influent	ac-ft	5.9	5.7	5.9	5.9	5.4	5.9	5.7	5.9	5.7	5.9	5.9	5.7	69.7
WWTP CONTRIBUTIONS														
Site Run-off	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2
Cooling Tower Blowdown	ac-ft	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cooling Tower Evaporation/Drift Loss	ac-ft	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.01	-0.1
RECYCLED WATER DISTRIBUTION														
Dual Plumbing	ac-ft	-1.1	-1.1	-1.1	-1.1	-1.0	-1.1	-1.1	-1.1	-1.1	-1.1	-1.1	-1.1	-13.4
Cooling Tower	ac-ft	-0.6	-0.6	-0.6	-0.6	-0.5	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-6.9
Landscape Irrigation	ac-ft	-0.8	0.0	0.0	0.0	0.0	0.0	-0.8	-1.7	-2.4	-2.5	-2.3	-1.8	-12.4
Other Irrigation (TBD)	ac-ft	-9.6	0.0	0.0	0.0	0.0	0.0	-9.8	-19.9	-28.4	-29.4	-27.1	-20.6	-144.9
RAW WATER MAKE-UP														
Blend Raw Water[1]	ac-ft	6.2	0.0	0.0	0.0	0.0	0.0	0.0	3.3	26.8	27.8	25.3	18.4	107.8
MONTHLY STORAGE BALANCE														
Beginning Storage Volume	ac-ft	0.0	0.0	4.1	8.3	12.5	16.4	20.6	14.0	0.0	0.0	0.0	0.0	
Change in Water Volume	ac-ft	0.0	4.1	4.2	4.2	3.8	4.2	-6.5	-14.0	0.0	0.0	0.0	0.0	
Final Storage Volume	ac-ft	0.0	4.1	8.3	12.5	16.4	20.6	14.0	0.0	0.0	0.0	0.0	0.0	

Maximum Seasonal Storage (ac-ft) **20.6**
mg **6.7**

Note:
1. Blend Raw Water is the deficit in ww flow generated to meet recycled water demands, to resolve then less water would be discharged for irrigation.
2. Assumed all equipment open basin/tankage would include covers and won't contribute to ww flows, confirm as more information becomes available.
3. Cooling tower blowdown is estimated at 10% of daily water demand and is included in the facility wastewater influent projection.
4. Cooling tower evaporation loss estimated at 1.5% of monthly water demand.