Ojai Valley Sanitary District

FACILITIES MASTER PLAN

FINAL | August 2020

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Carollo Project No. 11321A.00

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Abbreviations

AACE International	Association for the Advancement of Cost Engineering International
AADF	annual average daily flow
ADWF	average annual dry weather flow
ASR	alkali silica reactivity
Basin Plan	Los Angeles Region Basin Plan
BOD_5	5-day Biochemical oxygen demand test
cfs	cubic feet per second
chl.a	chlorophyll-a
CIP	Capital Improvement Plan
COD	chemical oxygen demand
DO	dissolved oxygen
EDR	electrodialysis reversal
EQ basins	
ISWEBE Plan	Water Quality Control Plan for Inland Surface Waters, Enclosed Bays, and Estuaries of California
lb/day	Pounds per day
MF	microfiltration
mg/L	milligrams per liter
mg/m2	150 milligram per square meter
mgd	million gallons per day
mL/g	milliliters per gram
MLSS	Mixed liquor suspended solids
NPDES	National Pollutant Discharge Elimination System
O&M	Operations and maintenance
OSWT	Off-site wastewater treatment
OVSD/District	Ojai Valley Sanitary District
PAO	Phosphorus accumulating organisms
PHF	Peak hour flow
PMF	peak month average flow
POTW	publicly owned treatment works
PTF	peak 2-hour flow
RAS	return activated sludge
RO	reverse osmosis
RWQCB	Regional Water Quality Control Board
SRT	solids retention time
SVI	sludge volume index
TKN	total Kjeldahl nitrogen

TMDL	Total Maximum Daily Load
TN	Total Nitrogen
TP	Total Phosphorus
TSS	total suspended solids
TST	Test of Significant Toxicity
USEPA	United States Environmental Protection Agenc-
USGS	United States Geological Survey
UV	Ultraviolet
AOP	Advanced oxidation process
VFD	Variable frequency drive
WWTP	wastewater treatment plant
WY	Water Year

Section 1

EXECUTIVE SUMMARY

The goal of the Facilities Plan is to develop a Capital Improvement Plan (CIP) that will meet the short and long term requirements of the OVSD's wastewater treatment plant by establishing a framework to guide the District through future rehabilitation and replacement projects over the next 20-years. This will allow appropriate budgeting as well as planning and design to take place in a timely manner to allow the projects to be constructed as they are needed.

Short term in this case is defined as 2025. The focus of the short-term plan is to prepare the plant to meet the Total Nitrogen (TN) and Total Phosphorus (TP) Ventura River Nutrient TMDL limits. Five different treatment configurations were evaluated, and the selected project makes full use of the existing site infrastructure and provides new denitirifcation filters that would provide seasonal process redundancy to meet the TN limit. The short-term CIP has a total cost estimate of $13.3 million (2019 dollars).

The long-term CIP looks beyond 2025 to the end of the planning period in 2039. A total of six projects were identified here. These projects address replacement of mainly mechanical equipment that is reaching the end of its useful life. Some of the projects identified in this group may be needed sooner than 2025, and the District can decide whether to bring the project start dates forward or push them out a few years. A potential project driven by future regulatory changes was included. This project was assumed to come online in 2035 and would produce a very high quality effluent. The long-term CIP has a total cost estimate of $34.39 million (2019 dollars).

A summary of the full 20-year CIP, with an estimated cost of about $47.6 million (2019 dollars) is presented as a timeline schedule in Figure 1.

Project No.	Project Name	Project Cost Estimate (2019)	Project Start
01	TMDL Project	$13,278,000	10/2/2020
02	Headworks and Influent Pump Station	$ 3,734,000	9/15/2023
03	Grit Chamber	$ 797,000	1/2/2022
04	Sludge Dewatering	$ 4,673,000	1/2/2022
05	Tertiary Filtration	$ 107,000	1/2/2022
06	UV Disinfection	$ 3,735,000	10/2/2020
07	Recycled Water Program/High Qlty Eff	$21,227,000	1/2/2030
	Total 20-Year CIP	$47,551,000	

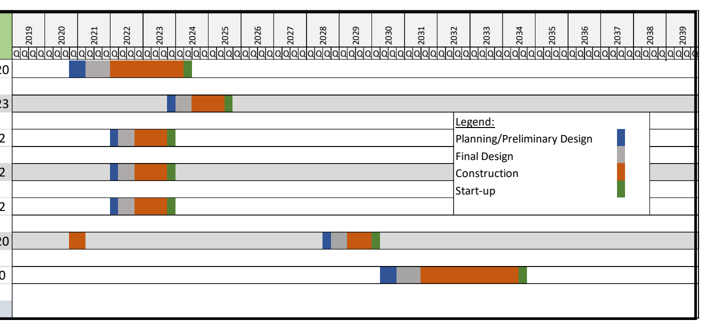

Figure 1 20-year CIP Summary

Section 2

INTRODUCTION

The Ojai Valley Sanitary District (OVSD/District) was formed in 1985 from four smaller sanitary districts and currently provides service to a population of about 24,000 people. The District maintains about 120 miles of mainlines ranging from 6-inch to 21-inch, 5 pump stations and a tertiary wastewater treatment plant (WWTP) that was last upgraded to a capacity of 3 mgd in 1996.

Untreated wastewater is collected from the City of Ojai; the unincorporated communities of Meiners Oaks, Mira Monte, Oak View, Casitas Springs, and Foster Park; and North Ventura Avenue area. Currently, the District collects and treats about 1.6 mgd that is discharged to the Ventura River under an National Pollutant Discharge Elimination System (NPDES) permit. During rain events the influent flow can increase to 3 mgd and during extreme events has exceeded 9 mgd.

2.1 Purpose

The purpose of this Facilities Plan it to present a 20-year plan for the WWTP, with the focus on the short term (less than five years) and medium term (five to ten years) periods. Longer term projects (beyond 2028) were also evaluated and discussed at a high level.

Section 3

BASIS OF PLANNING

The basis of planning for the Facilities Plan includes the short term and longer-term regulatory requirements, current and future flowrates for the plant, as well as influent wastewater characteristics.

3.1 Regulatory Drivers

The Regional Water Quality Control Board (RWQCB), Los Angeles Region, adopted the 2012 Total Maximum Daily Load (TMDL) for Algae, Eutrophic Conditions, and Nutrients in Ventura River; called the Ventura River Nutrient TMDL. In November 2013, the RWQCB adopted Order No. R4-2013-0173 that stipulated average monthly, maximum daily, and average seasonal limits for a range of water quality constituents, that include both concentration (mg/L) and mass loading (lb/day) limits for the monthly average. The concentration discharge limits for TN and TP were given an "interim" discharge limitation that applies to dry-weather conditions only.

Since then a new NPDES permit has been adopted; Order No. R4-2018-0170 with an effective date of February 1, 2019. Like the 2013 permit, the new Order stipulates average monthly, maximum daily, and average seasonal limits for a range of water quality constituents, that include both concentration (mg/L) and mass loading (lb/day) limits for the monthly average. TP was given a wet-weather concentration limit of 2.6 mg/L, and a dry-weather mass loading of

5,799 lbs, that is now in effect. TN was given winter seasonal concentration of 4.6 mg/L. An interim limit for TN was established at 7.6 mg/l monthly average.

The interim limits for TN are currently in place and last for 12-years following the effective date (June 28, 2013) of the Ventura River Nutrients TMDL, that is until June 29, 2025, after which the mass loading limits will apply. The mass loading limits for TN and TP are applied differently. For TN, the mass load of 8,044 lb/season applies for the summer season, from May 1 to September 30; a total of 153 days. For TP, the mass load of 5,799 lb/dry-weather season applies all year round, except during wet days (that receive more than 0.25 inches of precipitation).

At the current flowrate the WWTP will need to continuously meet TN and TP concentrations of around 4.0 mg/L and 1.3 mg/L, respectively, to meet the mass loading limits. The TP limit is heavily dependent on the number of "wet days" experienced during a given year.

3.2 Current and Future Flowrates

A critical part of establishing future treatment requirements is understanding future flows and loads to the plant. In 2018, the average annual daily flow to the plant was 1.6 mgd. The District decided that an annual growth rate of 0.3 percent per year should be used to establish future flows for the planning period. This results in an average baseline influent flowrate at the plant of 1.70 mgd in 2038, assuming 0.3 percent growth but no additional septic tank connections.

Figure 2 presents the projected flow rate for the 20-year planning horizon, and shows a baseline condition, lower curve, and two possible scenarios for the addition of flow from septic systems. Each scenario includes the calculated TN and TP concentration. The first scenario assumes the addition of 500 septic systems, or off-site wastewater treatment (OSWT) systems in about 5 years, while the second scenario assumes up to 1,000 septic systems being added to the collection system by 2028. For the purposes of the planning exercise it was assumed that for each septic system tied into the collection system, one capacity unit with an average flow of 133 gal/day would be added. For the first scenario, with 500 OSWT's added, the flow would reach 1.77 mgd in 2038, and 1.83 mgd for the second scenario that includes addition of 1,000 OSWT systems. The flow variations are important, because the flowrate impacts the effluent concentration limits to meet the TMDL.

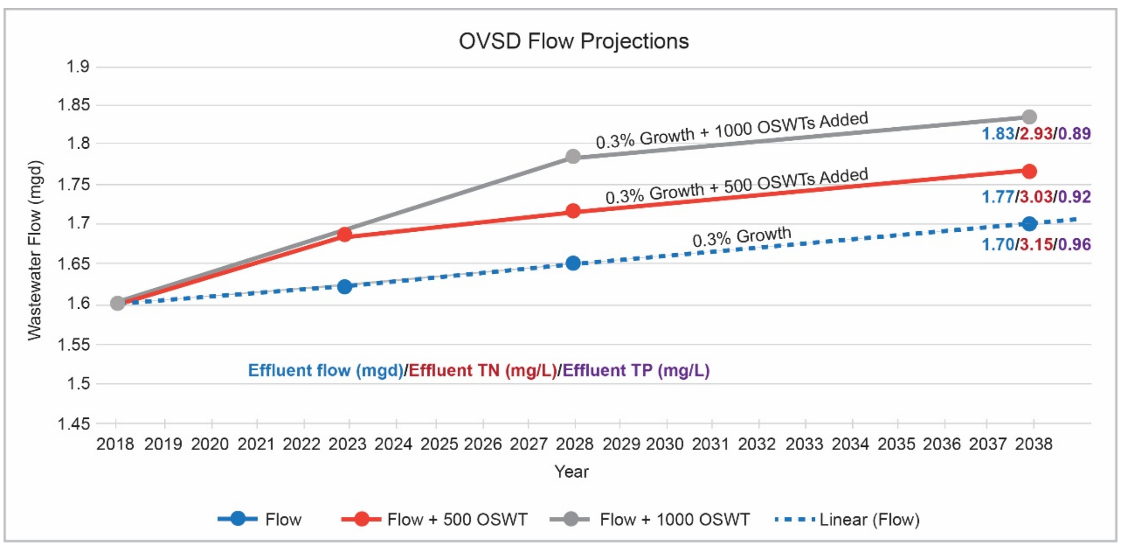

Figure 2 Flow Projection Curves for OVSD's WWTP

Figure 2 also shows the calculated effluent TN (in red) and TP (in purple) values that would be required to comply with the TMDL mass loading limits. Note that the calculated effluent limits include a 15 percent safety factor.

By 2038, the plant will have to meet TN and TP limits of about 3.0 mg/L (2.9 to 3.2 depending on the flow scenario) and <1.0 mg/L, respectively. When the TMDL comes into effect in 2025, plant flows will be lower and therefore the effluent limits will be higher, however the plant will have to be designed to meet the future TN and TP values at higher projected flows.

3.3 Influent Characteristics

Plant influent constituent concentrations and loading patterns were determined by analyzing plant influent data for the eight-year period of 2011 through 2018. As expected, recent influent constituent concentrations have increased relative to the 1996 design basis because of reduced water consumption within the service area due water conservation and the extended drought from 2012 to present. Reduced consumptive water use (i.e., the portion of potable water that enters the household sewer) provides less dilution of the organic and nutrient loads from residential and commercial sources and results in increased concentration of most wastewater constituents.

Table 1 summarizes the average and 90th percentile plant influent conditions determined for this project together with the values from the 1996 design data. With the exception of phosphorus, the average annual influent concentrations of the measured parameters increased by up to 38 percent compared to the 1996 design data, depending on the parameter concerned. The average annual data shown in Table 1 is the average data for January - November 2018.

BOD_5, TSS, and TKN showed the largest increases in concentration compared with the 1996 design values. Ammonia-N was the odd one out, showing only a 3 percent variation with the 1996 design value.

The reason for the significant reduction in phosphorus concentrations from the 1996 design data, for both soluble (42 percent) and total (20 percent), is not clear, but data showed that influent TP and influent soluble phosphorus (orthophosphate) values have been relatively stable for entire 2011 to 2018 period.

Table 1 Plant Influent Wastewater Characteristics

	1996 Design Data		2019 Facilities Plan(1)		% Change	
	Annual Average	90th Percentile	Annual Average	90th Percentile	Annual Average Data	Annual 90th Percentile Data
TSS, mg/L	320	520	401	514	25%	-1%
BOD_5, mg/L	240	294	332	442	38%	50%
TKN, mg N/L	40	55	51	78	28%	42%
NH_3-N, mg N/L	32	40	33	50	3%	25%
Total P, mg P/L	10	11	8	10	-20%	-9%
Soluble P, mg P/L	7	9	4	5	-42%	-44%
Alkalinity, mg/L as $CaCO_3$	325	340	337	360	4%	6%

Notes:
(1) Based on data for January to November 2018.

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3.4 Existing Facilities and Capacity

Figure 3 shows an aerial view of the existing plant, and Figure 4 shows a flow schematic. Raw wastewater flow enters the headworks through a 30-inch diameter trunk sewer. The headworks facility includes grinders and screenings removal. Downstream of the grinders, plant influent is directed to four intermediate pumps that lift the flow to a vortex grit removal system followed by a rotary drum screen. The screened influent is then routed to secondary treatment.

At secondary treatment, the influent flows through three anaerobic tanks in series. After the flow leaves the anaerobic tanks, it enters two identical parallel oxidation ditches that are sectioned into anoxic and aerobic zones. Flow from both oxidation ditches is combined in the mixed liquor splitter box and flows by gravity to two 85-foot diameter clarifiers. A portion of the clarifier underflow is sent to dewatering, and the remainder is routed back to the first anaerobic tank as the return activated sludge (RAS) flow. Secondary effluent flows to the filter influent pump station, where excess flow above the pump station flow set point flows by gravity to the three parallel equalization basins, and the rest is pumped to the tertiary facilities. During low flow conditions, stored secondary effluent in the EQ basins flows back to the pump station and is pumped to tertiary treatment. Processes downstream of the filter influent pump station all receive equalized flow.

At the tertiary facilities, secondary effluent flows through two flocculation basins in series before exiting through a channel. The channel feeds four deep-bed, continuous backwash sand filters before being routed to an ultraviolet (UV) system for disinfection. The UV system consists of one channel with five banks of UV lamps. As a backup, the flow can be routed through a chlorine contact tank downstream of UV.

After disinfection, flow is routed through a 28-inch diameter pipe, to a reaeration structure, and then into a 36-inch diameter pipe to the Ventura River outfall.

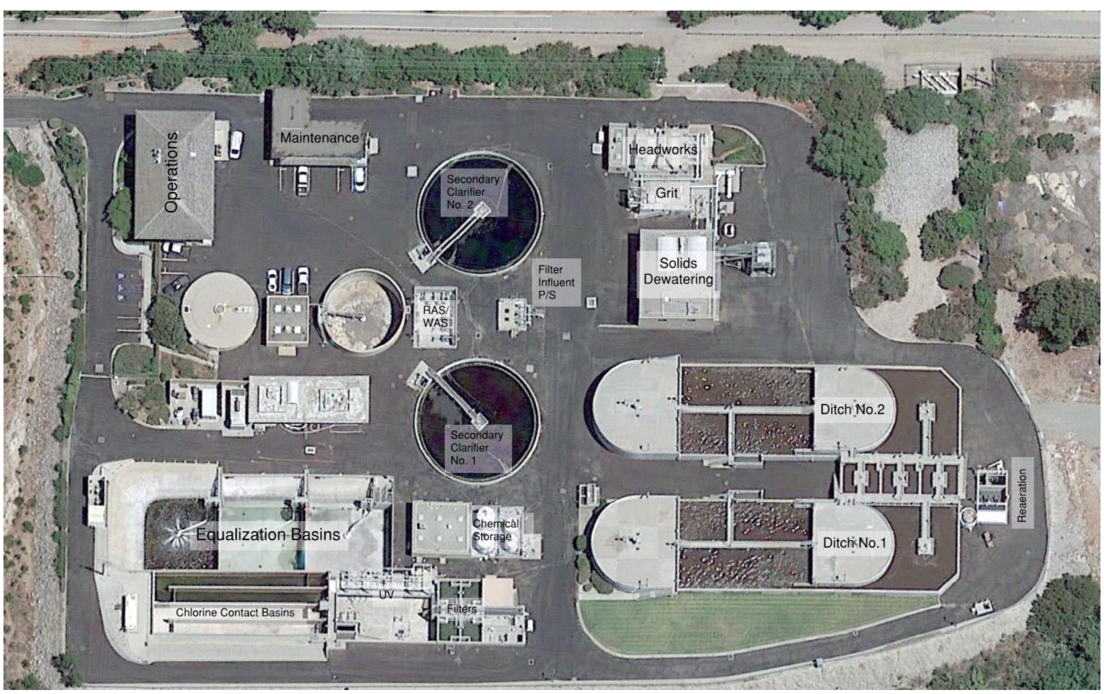

Figure 3 Aerial View of the OVSD Tertiary WWTP

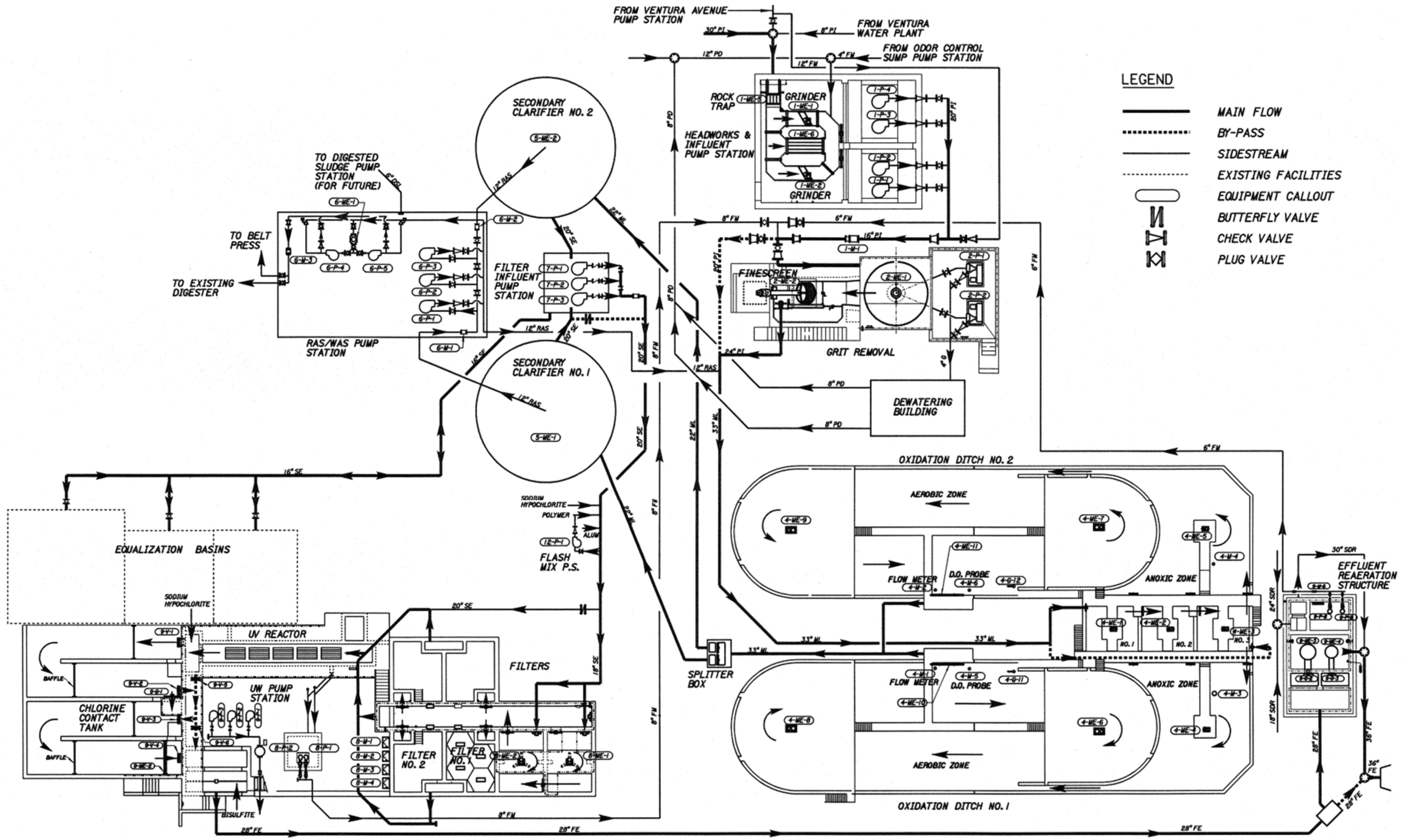

Figure 4 Liquid Process Flow Diagram of OVSD's WWTP

OVSD's WWTP influent flows were evaluated using influent historical flow data between January 2011 and December 2018 and are presented in TM 1, Existing Facilities Process Modeling in Appendix A. The design influent flows and peaking factors are summarized in Table 2.

Table 2 Design Flow Peaking Factors

Flows	Peaking Factor(1)	1996 Design Data Flows (mgd)
Annual Average Daily Flow (AADF)	1.0	3
Peak Month Average Flow (PMF)	1.43	4.3
Peak 2 Hour Flow (PTF)	2	6
Instantaneous PHF	3	9

Notes:
(1) Unless otherwise noted, peaking factors are relative to the design AADF of 3 mgd and are based on 1996 design data obtained from OVSD.

3.4.1 Process Model

A steady-state process model of the OVSD facility was developed using the activated sludge simulator, BioWin 5.3 (Flamborough, Ontario, Canada). The model was used to assess the capacity of the aeration basins for a range of operating scenarios. Modeling results were compared to historical operating data to confirm proper calibration of the model. The 3-month historical period from June 2018 to August 2018 showed recent stable plant performance and was used to calibrate the model. The steady-state process model was used to evaluate plant-wide process alternatives that are discussed later. Dynamic modeling is planned for the selected treatment alternative as part of the project design phase.

The modeling effort determined that the existing WWTP's secondary treatment capacity, assuming current discharge permit requirements, with updated loading parameters is 2.5 mgd under average annual dry weather flow (ADWF) conditions. This was assuming that both clarifiers will be in service during wet weather. The change in capacity from the design value (of 3 mgd) is due to the higher loading of BOD, TSS and nutrients compared with the design values used in 1996.

The modeling also determined that the limiting process component is the solids loading rate of the secondary clarifiers. This solids rate is based on a 90th percentile sludge volume index (SVI) during the period of 2011 - 2018 of 220 mL/g. To help control the SVI and prevent the large swings in SVI observed over the eight-year data period, it is recommended that the plant switch to solids retention time (SRT) control, from the current method of mixed liquor suspended solids (MLSS) control. This will help to stabilize the biomass population to match the influent loads and seasonal variations. As a result, the MLSS concentrations should vary during the year to reflect the changing conditions in the aeration basins. This will also help to maintain the phosphorus accumulating organisms (PAO) population and produce more consistent bio-P performance.

Calibrating the BioWin model was found to be somewhat challenging due to the differences between the influent wastewater quality characteristics and those for typical domestic sewage. This could be due to the presence of the dairy effluent and the long sewer system. Accordingly, it was recommended that a separate study be undertaken to assess the influent wastewater characterization to determine the fractions of the chemical oxygen demand (COD) such as soluble, biodegradable and non-biodegradable fractions, as well as the fractions of the nitrogen species.

3.4.2 Collection System Sampling Study

In September 2019, in an effort to better define the water quality for modeling, four composite samples were collected from five locations in the collection system that receive flow from four sewershed basins. A report on the preliminary assessment of the sampling study is included in TM 1 in Appendix A, together with a summary of the water quality data obtained from the study. The evaluation confirmed that the process modeling work conducted to evaluate various process alternatives to meet the TMDL resulted in reasonable conclusions. However, two recommendations were also made:

1. Further evaluation of the sewer system between sample point C01-C07 and the plant should be undertaken to identify any other sources of wastewater entering the system. This recommendation is based on the fact that the COD value of 1,300 mg/L for the September 12/13 sample at the plant cannot be explained.
2. The results from the September 2019 sampling study (as well as any additional data that may be collected) should be used to recalibrate the BioWin process model during the design phase of the TMDL project. At that point the model can be re-run to fine tune the process design parameters for the selected alternative.

Section 4

REGULATORY DRIVERS

There are several categories of existing or potential upcoming regulatory requirements that this facility plan addresses:

- Implementation of the Ventura River Nutrients Total Maximum Daily Load in 2025.
- Potential adoption of new and updated United States Environmental Protection Agency (USEPA) water quality criteria in the Los Angeles Region Basin Plan (Basin Plan).
- Requirements initiated at the State Board or Regional Board level.
- Future regulation of surface flows in the Ventura River.

4.1 Impacts of Reopening the TMDL

The 2012 TMDL for Algae, Eutrophic Conditions, and Nutrients in Ventura River, including the Estuary and its Tributaries (TMDL)[1] was based on a benthic algal biomass target for algae (150 milligram per square meter (mg/m^2) chlorophyll-a (chl.a)) which drove the quantification of the required load reductions. The sequence of steps used by RWQCB staff to derive load allocations resulted in required TN and TP load reductions of 50 percent for most dischargers. This benthic algal biomass target was not based on evidence linking levels of algal biomass to

[1] Los Angeles Regional Water Quality Control Board Resolution R12-011, adopted December 6, 2012, and becoming effective June 28, 2013.
Los Angeles Regional Water Quality Control Board Order R4-2018-0170, adopted December 13, 2018, and becoming effective February 1, 2019.

aquatic life beneficial use impairment (such as low dissolved oxygen (DO) or alteration of benthic invertebrate assemblages). Instead, it was based on subjective interpretations of how much stream algae is likely to impair recreational uses such as wading and trout fishing, and data sets that include few (usually none) southern California streams or streams from other Mediterranean climates.

The TMDL requirements are discussed in detail in TM 3 - Total Maximum Daily Load Requirements, included in Appendix A. A summary of some of the key findings are presented here, as regards potential future impacts to the regulations should the TMDL be reopened.

A variety of key assumptions of the TMDL are described that might or might not hold up if the Algae TMDL was reopened in the future. Among the vulnerable assumptions are 1) that nutrient loading during wet weather does not contribute to algal-related impairments, 2) that existing loading to the estuary is not high enough to cause impairments of beneficial uses, and 3) that nitrate contributions from daylighting groundwater were correctly characterized. In each case, if the assumption was discarded or revised during development of a future TMDL, estimates of assimilative capacity of TN and/or TP could be lower and more stringent load reductions might be required for dischargers. Other concerns related to the TMDL are highlighted below. The regulatory reaches of the Ventura River, and the location of OVSD's outfall, are shown on Figure 5.

- TMDL Monitoring Requirements and Results:
  - Exceedances of the algal biomass and DO targets in the TMDL have been frequent in the lower Ventura River and the Estuary since compliance monitoring began in early 2015. Diurnal variations in pH and DO are consistently observed in the river, above and below the discharge, and are strong evidence that submerged plants and algae are exerting an influence on DO. The fact that nutrient loads are lost in a non-conservative fashion in the lower river further supports a strong role of biological uptake in nutrient fate and transport.
  - It is currently unknown whether the river between Foster Park and the OVSD discharge is typically a gaining or losing reach. It will be important to correctly understand the nature of the flow subsidy from the OVSD discharge and the relationship between OVSD nutrient discharges and nutrient loads arriving from upstream.
  - <u>In most months</u>, surface flow decreases between the OVSD outfall and the estuary. The extent to which the loss of surface flow represents direct evaporative losses, uptake and evapotranspiration by aquatic and riparian vegetation, and/or groundwater recharge in the lower river is unknown. Understanding groundwater recharge may become important in the future if groundwater quality in the Lower Ventura River basin becomes an issue with the Regional Board.
  - There is evidence for periodic significant inputs of water and nutrients below the OVSD discharge that are unrelated to OVSD effluent. It will take many years of compliance monitoring to determine whether interannual hydrology (e.g., size and timing of winter storms, juxtaposition of wet and dry years) is responsible for different patterns of fate and transport of nutrients in the lower river. Compliance monitoring is not designed to elucidate which sources of nutrients unrelated to OVSD contribute to those patterns.

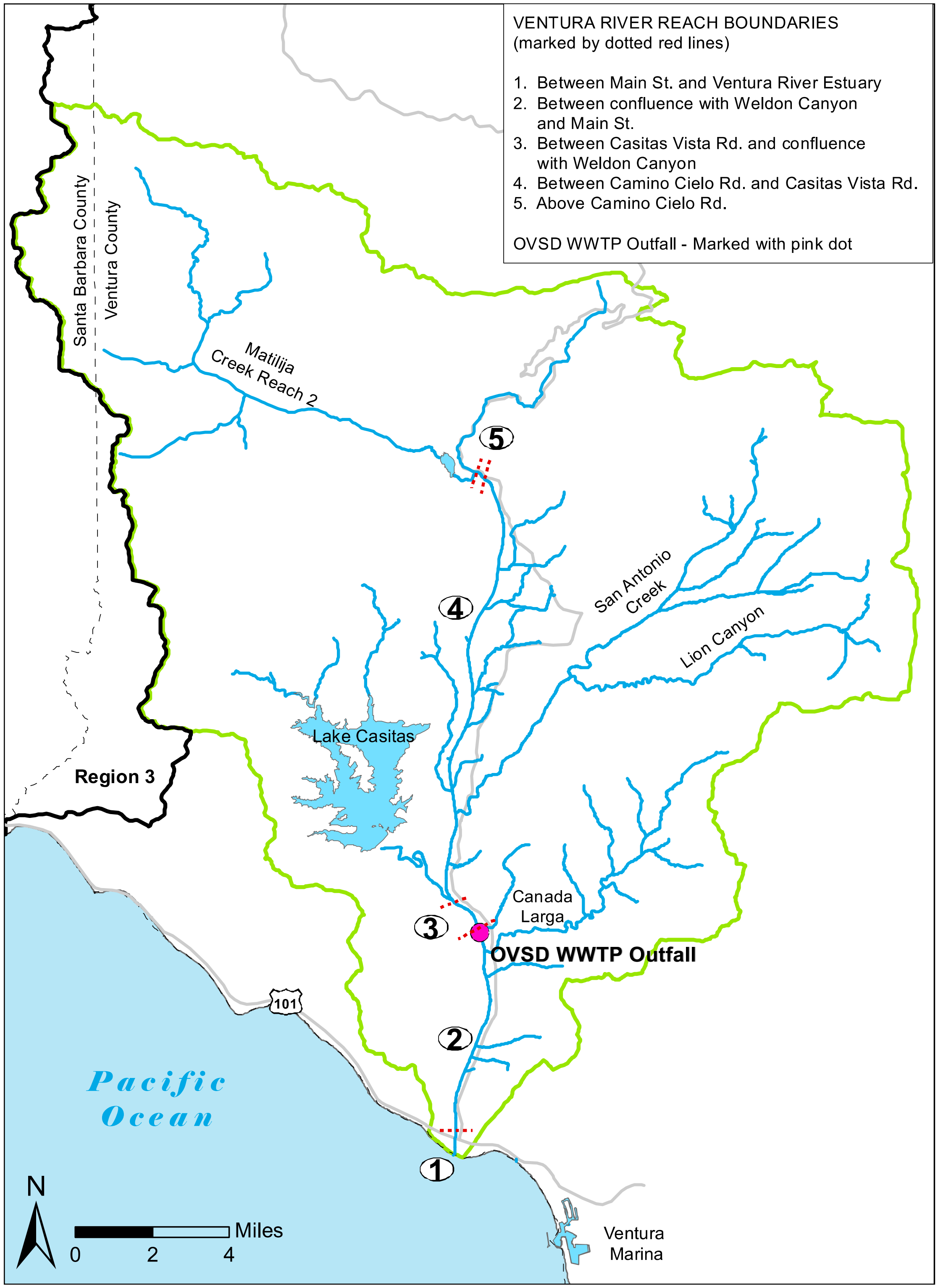

Figure 5 Regulatory Reach Designations for the Main Stem of the Ventura River

- Existing Flow Conditions in the Lower River:
  - An 89-year record of mean daily flows for United States Geological Survey (USGS) Gage 11118500, located at the Foster Park Bridge (top of Reach 3), was used to characterize long-term average patterns of flow for entire Water Year's (WYs) and calendar months.
  - Based on long-term median mean daily flows for calendar months, the OVSD "flow subsidy" ranges from 17 percent (in March) to 92 percent (in September) of total estimated flow at the outfall. However, because days with zero flow are statistically possible during any month at Foster Park, the flow subsidy can intermittently be much higher.
- Evidence for Non-Nutrient Related Contributions to Impairments Addressed by the Algae TMDL:
  - Temperature, conductivity, and flow could all influence DO in the river, but they are not responsible for the strong diurnal variations in DO and pH that are characteristic for the river. Data that would allow evaluation of the effects of canopy cover or other riparian habitat characteristics on algal-related impairments are not being reported by monitoring entities.
  - Empirical relationships between flow, DO, and algal biomass from the Ventura River suggest that mean daily flows $\geq$ 3 cubic feet per second (cfs) would prevent benthic algae at TMDL target levels (150 mg/m^2 chl.a) from driving pre-dawn DO below 5 milligrams per liter (mg/L).
  - There are several lines of evidence that non-algal factors are influencing daily and monthly patterns of DO in the estuary. The lunar tidal cycle, and particularly the spring/neap tidal cycle, may be driving the timing, frequency, and severity of DO impairments in the estuary. This phenomenon will be important to understand if DO impairments in the estuary are addressed in a new or reopened TMDL.
- Potential Impacts of Different Forms of Nitrogen on Benthic Algae in the Lower River:
  - The time frames and extent to which inputs of organic N or particulate N can participate in algal growth has not been studied in the Ventura River. Nutrient <u>spiraling</u> lengths for the Ventura River, are not known. Estimated nutrient <u>uptake</u> lengths cited by the Regional Board in the TMDL Staff Report are approximately half the distance between the OVSD outfall and the head of the estuary, however, the validity of the estimates is not known.
  - Receiving water data from OVSD's required monitoring program revealed high variability in the magnitude and percent organic N in stream water above the OVSD discharge. Frequent high percentages of organic N above the outfall suggest that much of the TN naturally in transport in the lower river would require microbial processing before being eligible to contribute to algal or macrophyte growth.

4.2 Impact of Future Regulations

TM 4 - Future Regulatory Requirements, presented a review of current and future regulations that might impact the operation of the treatment plant, which effluent quality parameters might be impacted and when such regulations may be implemented. TM 4 is included in Appendix A. A summary of the findings are presented here.

Based on current effluent and receiving water quality, changes to OVSD's permit limits during the Facilities Plan planning period are most likely to occur based on the following three factors:

1. Potential incorporation in the Region 4 Basin Plan of new USEPA human health criteria would trigger reasonable potential, and a need for numeric effluent limits, for seven constituents that are not currently assigned limits in OVSD's NPDES permit. The seven constituents are Dioxin, Benzo(a)pyrene, Benzo(b)flouranthene, Bis(2-ethylhexyl)-Phthalate, Dibenzo(a,h)Anthracene, Dichlorobromomethane, and Indeno(1,2,3-cd)Pyrene. It is not yet known what effluent concentration would be applied.
2. Adoption of new, more stringent, aquatic life criteria for ammonia and selenium into the Region 4 Basin Plan would result in revised permit limits for OVSD, but are unlikely to pose compliance problems.
3. A reopened Algae TMDL or a new Benthic Community Effects TMDL for reaches below OVSD's discharge could result in a reevaluation of OVSD's effluent limits for TN and TP. The potential arises from ongoing exceedances of the numeric targets in the Algae TMDL in the reaches below the OVSD discharge and potential new statewide impairment thresholds for TN and TP (lower than concentrations used for modelling in the Algae TMDL) that may be included in the State Board's upcoming Biostimulatory Substances Amendment to the Water Quality Control Plan for Inland Surface Waters, Enclosed Bays, and Estuaries of California (ISWEBE Plan). However, it is currently unknown how the amendment will be implemented for specific water bodies or publicly owned treatment works (POTWs) in general, and what regulatory off-ramps might be provided to dischargers.

Other more stringent new or updated water quality criteria promulgated by the USEPA or the State Board that could be adopted in Region 4 in the next few years for ammonia, cadmium and copper appear to be comfortably met at OVSD's receiving water monitoring stations and in OVSD effluent. Barring changes in effluent and receiving water quality, it is not likely that these other new standards will result in effluent limits for OVSD.

OVSD will need to track whether use of the new Test of Significant Toxicity (TST) test statistic for toxicity tests leads to future exceedances in effluent or receiving water, causing potential for 303(d) listings for toxicity in Reaches 1 or 2, and potentially expand the geographic scope of the expected toxicity TMDL for Reach 3.

OVSD comfortably meets its effluent limits for salt constituents (total dissolved salt (TDS), chloride, sulfate, boron), and receiving water below the outfall comfortably meets the Basin Plan surface water objectives for salt constituents that apply in Reach 2 (there are no surface water objectives for salts that apply to Reach 1). Based on current receiving water quality, 303(d) listings and a TMDL for salts in Reach 2 would not occur unless surface water objectives are changed through a Basin Plan Amendment. A reevaluation of OVSD's permit limits for salts that was based on protection of *groundwater* quality would likely be accomplished through the salt and nutrient planning process in the Recycled Water Policy, which would be preceded by studies and stakeholder processes, and would also require a Basin Plan Amendment.

Three parallel regulatory processes are underway that directly or indirectly address surface flows in the Ventura River. Regulation of surface flows could affect the ability of OVSD to divert effluent to reuse. Guesswork about whether OVSD's reuse prospects would be positively or negatively affected by these developments is extremely speculative at this time. A key study by California Department of Fish and Wildlife (CDFW), designed to provide guidance to the State Board on flows required to support Southern California Steelhead habitat and life cycles, may provide the earliest clues about the future status of surface flows in the lower Ventura River.

Section 5

CONDITION ASSESSMENT OF EXISTING FACILITIES

A process-by-process visual condition assessment of structural, mechanical and electrical components was not part of the original project scope. Instead, the operational and physical condition of the various processes at the WWTP was established based on the findings and discussion with operations and maintenance (O&M) staff during a Workshop Meeting on March 26, 2019. This information was used to identify potential modifications to the treatment plant that may or may not be incorporated into the short term TMDL project. The preliminary condition assessment of existing facilities is summarized in the TM 6, Condition Assessment of Existing Facilities, included in Appendix A.

The existing as-built drawings were reviewed, and a preliminary list of the processes and facilities was developed. A process-by-process review of the information took place and estimates were made of the remaining useful life for concrete structures and mechanical equipment. Some process areas have harsher conditions, and concrete life is lower in those areas. No assessment of the electrical components of the plant were made.

5.1 Structural

All concrete structures on the site have life expectancy that is greater than the 20-year planning horizon of this Facilities Plan, except for the oxidation ditches. The ditches were originally constructed as part of the 1997 project, and the concrete already shows visible signs of deterioration. Oxidation Ditch No. 1 (west side ditch) was constructed first. Oxidation Ditch No. 2 was constructed about 18 months later. Ditch No. 1 has the worst condition with numerous visible cracks, see Figure 6.

Due to the condition of the ditches, OVSD commissioned an investigation by Oakridge Geoscience Inc. and the CTL Group, who took core samples of the ditch walls. These samples confirmed the presence of alkali silica reactivity (ASR) in the concrete aggregate. Since both structures are unlined, it is anticipated that the concrete deterioration will get worse over time, and that the anticipated life of the structures is less than 20 years.

Figure 6 Examples of Cracks Visible in the Ditch No. 1 Structure

Given that the life expectancy of these structures is less than 20 years, the District feels that a major structural rehabilitation is needed. This might include removal and rebuilding of some or all of the decks that support the aerators.

For the purposes of the Facilities Plan it was assumed that structural rehabilitation of the ditches will be required. For Ditch No. 1 this would include removal and reconstruction of the two aerator decks, as well as lining of the entire structure with an epoxy or polyurethane liner. For Ditch No. 2, only lining of the ditch was assumed.

5.2 Mechanical Equipment

For all facilities evaluated, the condition of the mechanical equipment varied between poor and good. The District provides continuous maintenance, repairs and replacement of certain equipment, such as in the utility water pump station and the chemical dosing building, so these facilities would not need to be included in a CIP.

5.3 Electrical and Instrumentation Equipment

The electrical and instrumentation and control components of the plant have been kept current by the plant maintenance staff. Accordingly, these facilities were not included in the assessment.

5.4 Summary

Based on the condition assessment and feedback from the District staff, the following presents a summary of the modifications that need to be included in either a short term CIP, perhaps as part of the TMDL project, or a longer term CIP.

1. Headworks and Influent Pump Station:
    a. Remove the existing Rock Trap.
    b. Replace the existing channel grinders with new bar-screens and a new screenings washer/compactor located at grade.
    c. Make provisions to address grit settling during low-flow conditions.
    d. Replace all existing gates.
2. Grit Chamber:
    a. Make provisions to address grit settling in the channels during low-flow conditions.

   b. Replace the existing grit mixer.
   c. Evaluate and rehabilitate the structure as needed to address corrosion and cracks.
3. Sludge Dewatering:
   a. Replace existing sludge transfer pumps with new pumps. Consider alternative technology to minimize vibration issues.
   b. Replace the existing BFP with a new dewatering unit. Consider installing a redundant dewatering unit.
4. Oxidation Ditches:
   a. Replace all original mechanical equipment (anaerobic mixers, anoxic mixers, and aerators).
   b. Demolish and replace the aerator decks in Ditch No. 1.
   c. Line both ditches with an epoxy or a polyurethane liner to minimize further concrete deterioration due to ASR.
5. RAS/WAS Pump Station:
   a. Replace the existing RAS pumps.
6. Tertiary Filtration:
   a. Make provisions to address solids accumulation during low-flow conditions.
   b. As far as possible, line the filter structure with an epoxy or a polyurethane liner to minimize the potential for deterioration due to areas of ASR.
7. UV Disinfection:
   a. Incorporate the recommendations from Carollo Engineers, Inc. (Carollo)'s August 2018 report and an onsite meeting on August 30, 2019.

Findings from the condition assessment are presented in TM 6 Condition Assessment, located in Appendix A. The proposed CIP for the above is presented in the next section.

Section 6

CAPITAL IMPROVEMENT PROGRAM

The goal of the CIP is to present a framework to guide the District through future rehabilitation and replacement projects over the next 20-years. This will allow appropriate budgeting as well as planning and design to take place in a timely manner to allow the projects to be constructed.

Costs are presented in 2019 dollars and are not escalated to future years. Costs were prepared in accordance with the guidelines of Association for the Advancement of Cost Engineering International (AACE International) 18R-97 for a Class 5 estimate.

Construction cost estimates include direct and indirect costs. Direct costs include materials, labor, construction equipment required for installation, and subcontractor costs. Indirect costs include contractor general conditions, contractor overhead and profit, sales tax, and an estimating contingency of 25 percent.

Direct construction costs were estimated from various references. Where possible, the costs from design estimates or construction bid tabs were used and converted to current dollars. Other cost sources included Carollo's reference projects, the R.S. Means price catalog, Carollo's

Unit Price catalog, and vendor quotes for major pieces of equipment. The total project capital cost was estimated as the total construction cost plus an additional allowance of 25 percent for engineering, legal, administration, and permitting cost.

For this Facilities Plan, projects fall into two categories. Firstly, there those projects that meet the short terms needs over the next 5-years. Secondly, there are those that will position the District to meet the longer terms needs associated with facility rehabilitation and replacement, as well as addressing future needs such as meeting a higher effluent quality for recycled water production, for example. Each is dealt with in the sections that follow.

6.1 Addressing Short Term TMDL Needs

Short term objectives focus on the need to address modifications driven by OVSD's 2018 NPDES Permit. Although the WWTP routinely meets the numeric concentration limits in the NPDES Permit, the permit also includes a TMDL for nitrogen and phosphorus removal, which comes into effect in June 2025. Short term objectives also include improving the energy efficiency of the plant. Four process upgrade alternatives were developed and evaluated to address the short term objectives.

Five treatment alternatives were developed for meeting the short term 2025 TMDL, as listed below. Details are presented in TM 5, in Appendix A.

- Alternative 1 – 5-Stage Bardenpho in Combined Ditch Configuration.
- Alternative 2 – 3-Stage Bardenpho Ditches plus Denitrification Filters.
- Alternative 3 – 5-Stage Bardendo in modified Ditch Aeration Basin Configuration.
- Alternative 4 – New 5-Stage Bardenpho Aeration Basin Process.
- Alternative 1A – Combination of Alt 1 and Alt 2.

Alternatives 1, 2, 3 and 1A include use of one or both of the existing oxidation ditches. Due to the concerns with the condition of the concrete in the oxidation ditches and whether the structures will last another 20 years, a fifth alternative (Alternative 4) was developed, which would construct a new aeration basin designed to meet current and future operating conditions and the effluent limits required in the TMDL.

Each alternative was evaluated for its ability to meet the TMDL limits, as well as deal with higher flow and loads during high flow events in the winter. Each configuration was stressed to determine its performance for treating a flow of 6 mgd for six consecutive days, at loads 50-percent greater than average conditions. These conditions were expected to simulate a large winter storm event.

Alternative 1 (combining both existing ditches into a single 5-stage Bardenpho system) had the lowest estimated 20-year life-cycle costs, but lacked redundancy, making it impractical as a long-term solution. Accordingly, a hybrid of Alternatives 1 and 2, called Alternative 1A, was developed, to include the benefits of Alternative 1 and the redundancy features of Alternative 2. Because of its ability to makes use of all existing facilities and incorporate good process redundancy and operational flexibility, Alternative 1A, with an initial preliminary estimated construction cost of $16.1 million (2019 dollars), was selected as the preferred alternative. This cost estimate was later refined, as described in the next section.

6.1.1 Description of Preferred Alternative

Alternative 1A includes using both ditches in series during the winter months in a 5-stage Bardenpho configuration, with the flexibility to take one ditch out of service in the summer months. When one ditch is out of service, the required TMDL limits would be achieved via polishing in denitrification filters with a capacity of 2 mgd. This would provide a high level of flexibility to plant operations.

Alternative 1A would operate as Alterative 1 during the wet season. That is, the ditches would be operated in series as a combined 5-stage Bardenpho process to achieve the TMDL limits even during high flow events. Smaller capacity denitrification filters would be provided to allow one ditch to be taken out of service during the summer months. In this configuration, the operating ditch would become a 3-stage Bardenpho ditch, as it is today, and the denitrification filters would provide polishing to remove residual nitrate. Process modeling has shown that this configuration will achieve the TMDL limits during the summer months. Figure 7 shows a schematic of Alternative 1A. The dotted line indicates the flow configuration to the denitrification filters when one ditch is out of service. Additional details are presented in TM 5 TMDL Implementation and Facilities Upgrades, located in Appendix A.

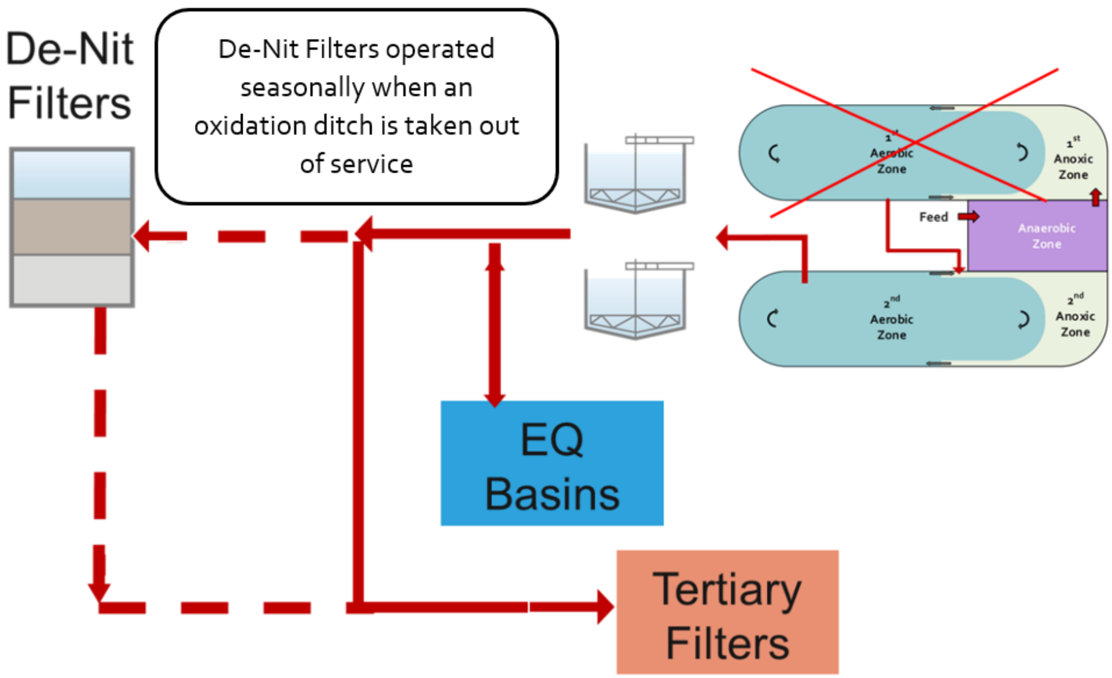

Figure 7 Schematic of Hybrid Alternative 1A

A preliminary site layout for Alternative 1A is shown in Figure 8. As indicated, the denitrification filters are located to the west of the westerly ditch on the existing grass area. The capacity of the filters would be 2 mgd, to meet the anticipated maximum daily flow conditions during summer. The layout also shows new chemical storage facilities to the south of the filters. These would replace the existing temporary Micro-C storage and dosing system located south of the anaerobic zone.

Figure 8 Preliminary Layout for Hybrid Alternative 1A

6.1.1.1 Cost Refinement of Alternative 1A

The initial capital cost estimate for Alternative 1A was evaluated to identify areas where costs could be cut or deferred to a future project. The major item considered was the proposed lining of the oxidation ditches to prevent further deterioration of the concrete due to the ASR conditions. Because portions of oxidation ditch No. 1 would be re-built as part of the project, the District felt that lining of the ditch may no longer be needed. For oxidation ditch No. 2, the extent of the concrete deterioration appears to be significantly less than for ditch No. 1. For these reasons it was decided to remove lining of both ditches from the project, which resulted in a significant cost saving.

A small cost reduction resulted from refinement of the size of the connection pipe between the ditches. The initial pipe size was reduced to 42-inches. The hydraulic impacts of this reduction in pipe size will need to be assessed during the design.

Additionally, the allowances included for electrical and instrumentation, mechanical, and site yard work were evaluated to identify more specific estimates for each. Finally, the allowance for engineering, management and legal was reduced from an initial amount of 35-percent to 25-percent, and the tax percentage was adjusted.

These refinements reduced the construction cost estimate from $16.1 M to $10.6 M (all 2019 dollars), resulting in a total project cost estimate of $13.3 M (2019 dollars), including engineering, management and legal costs. Details of the original and refined cost estimates are included in Appendix B.

6.1.2 Cost Estimate for Short Term CIP

The short-term CIP includes a single project that incorporates Alternative 1A described above. The estimated project cost for Alternative 1A is presented in Table 3, and includes the following project elements:

- Modifications to both oxidation ditches to allow them to operate in series.
- Replacement of the remaining two gates in the ditches, the radial turbine mixers in the anaerobic stage, the radial turbine mixers in the anoxic stages.
- Replacement of the mechanical aerators with new VFD driven units.
- Demolition and replacement of the aerator decks in Ditch No. 1.
- Replacement of the four RAS pumps with new non-clog horizontal centrifugal pumps.
- New denitrification filters with a capacity of 2 mgd.
- Upsizing of the filter feed pumps to meet the requirements of the new denitrification filters.
- New chemical storage and dosing system to support the new denitrification filters and provide a permanent area for storage and dosage of Micro-C for the oxidation ditches with two 3,000 gallon storage tanks, and associated yard piping, valve stations and controls.

The refined construction cost, excluding items identified in Section 6.1.1.1, is estimated to be $10.6 million (2019 dollars), which results in a project cost of around $13.3 million(2019 dollars) with the inclusion of a 25 percent allowance for engineering, legal and administration costs. Details of the costs are included in Appendix B. This project would come on line in the first half of 2024 to allow one full year of seasonal variations before the TMDL limits begin in June 2025.

Note that the TMDL project does not include any upgrades to the existing plant other than the items listed above. The District may choose to bring forward one or more of the projects listed in the next section to include with the TMDL project.

Table 3 Short Term CIP to 2025

CIP Project Number	Summary Description	Required On-Line Date	Project Cost(1)
01	Addressing Short Term TMDL: • 5-Stage Bardenpho Ditch Modifications to combine ditches per Alt 1A • Denitrification Filters • Chemical Storage	2024(2)	$13,278,000
	Total Short Term CIP (2019 - 2025)		**$13,278,000**

Notes:
(1) Project cost includes construction cost estimate plus a 25 percent allowance for engineering, legal, and administration costs. Costs are in 2019 dollars.
(2) Completing this project by June 2024, allows one full year of operation prior to the TMDL enforcement date.

6.2 Addressing Longer Term Needs

The projects that will position the District to meet the longer terms needs (beyond 2025) associated with facility rehabilitation and replacement, as well as addressing future regulatory or effluent quality needs such as for recycled water production, are discussed in this section.

Table 4 presents the longer-term CIP which is assumed to start after year 2025. Based on the list of projects presented in Table 4, the total long term CIP for the period 2025 to 2038 is expected to be about $34 million.

As shown in Table 4, some projects have "required on-line" dates that are earlier than the assumed start date for the long term CIP. This is because, based on the condition assessment

(TM 6) these facilities are expected to have reached their useful life prior to 2025. The District may wish to move some or all of these projects into the short term CIP.

Project Nos. 02 through 05 (headworks and influent pump station, grit chamber, sludge dewatering and tertiary filtration) were described earlier in Section 5. These projects mostly address rehabilitation and replacement of existing mechanical facilities that are reaching the end of their useful life.

6.2.1 UV Disinfection

Project No. 06 addresses the requirements for the UV Disinfection system. In August 2018, Carollo submitted a report to the District on the performance of the UV system. There had been some effluent bacterial count violations. The report is included in Appendix C. The plant has been operating the UV system with four of the five banks in service, to ensure that sufficient dose is applied to the water. Four banks of UV lamps in operation is significantly more than should be needed during average flows of around 1.6 mgd.

Carollo's 2018 report resulted in two main findings. Firstly, the water level in the channel exceeded the operating criteria. Secondly, there was the need to address the issue with seemingly poor UV intensity, which may well be related to the first issue. In August 2019 there was a follow up meeting on the site. Inspection of the level in the UV channel indicated that the level of water above the UV lamps was higher than it should be. This would lead to the potential for lower doses and bacterial count violations. The District has been working with the UV maintenance providers, Ironbrook, to get the downstream level control gate adjusted to produce the correct depth of water above the top lamps and avoid short circuiting. Final adjustments were made on November 14, 2019. It is not clear whether or not the level adjustment will solve the high effluent bacterial counts.

If the channel water level adjustment does not solve the issue, then there is potential to increase the UV intensity from the existing lamps by increasing the voltage feeding the rectifiers which will increase voltage that feeds the ballast which will thereby increase the lamp current and power to the lamp. According to Ironbrook, the UV system should be able to handle a voltage increase of 10 percent, which would increase the lamp power. The actual increase in power would have to be measured on site.

Because the current modifications to the UV system are relatively minor, a line item in the CIP to address the recent bacterial count violations, has not been included. However, the UV lamps, transformers, ballasts and UV racks were all replaced in 2013 and are expected to have a remaining life of around 10 years. Thus, for purposes of the CIP, a cost to replace the UV system in 2030 has been included.

6.2.2 Future Recycled Water Project

Project No. 07 (Recycled Water Project) was included for completeness. This project is not a requirement at this point, but is a placeholder to capture the anticipated cost of potential changes to the Regulatory framework that would require a significantly higher quality effluent from the plant; whether this effluent is discharged to the River or used as high quality recycled water. For purposes of the CIP, the project was assumed to be required in 2035.

6.3 CIP Schedule

A preliminary schedule for the proposed 20-year CIP is shown in Figure 9. In the figure, each year of the CIP is divided into quarters. Estimates have been made for planning/preliminary design, final design, construction and start up for each of the seven projects. For Project No. 02 (Headworks and Influent Pump Station), the schedule shows a later start time than that listed in Table 4. This is to illustrate the impact of starting Project No. 02 later, to avoid having two contractors on the site at the same time, as would be the case if Projects 02 - 04 all start in 2022 to achieve the 2023 completion date.

Table 4 Long Term CIP (2025 - 2038)

CIP Project Number	Summary Description	Required On-Line Date	Project Cost(1)
02	Headworks and Influent Pump Station: • Remove the existing Rock Trap. • Replace the existing channel grinders with new bar-screens and washer/compactor at grade. • Address grit settling during low-flow conditions. • Replace all existing gates.	2023(2)	$3,734,000
03	Grit Chamber: • Address grit settling in the channels during low-flow conditions. • Replace the grit mixer. • Evaluate and rehabilitate the structure as needed to address corrosion and cracks.	2023(2)	$797,000
04	Sludge Dewatering: • Replace sludge transfer pumps with new pumps. • Replace the existing Belt Filter Press (BFP). • Installing a redundant BFP dewatering unit.	2023(2)	$4,673,000
05	Tertiary Filtration: • Address solids accumulation during low-flow. • Line the filter structure with an epoxy or a polyurethane liner to minimize the potential for deterioration due to areas of ASR.	2023	$107,000
06	UV Disinfection: • Incorporate the recommendations for Carollo's August 2018 report and an onsite meeting on August 30, 2019.	2030(2)	$3,735,000
07	Recycled water program or Higher Quality Effluent: • MF/RO/UV-AOP treatment with EDR for brine concentration(3).	2035	$21,227,000
	Total Long Term CIP (2025 - 2038)		**$34,273,000**

Notes:
(1) Project cost includes construction cost estimate plus a 25 percent allowance for engineering, legal, and administration costs. All costs in 2019 dollars.
(2) Based on remaining life of existing equipment.
(3) Full advanced treatment cost estimate shown here considered as the conservative alternative for planning purposes.

Project No.	Project Name	Project Cost Estimate (2019)	Project Start
01	TMDL Project	$13,278,000	10/2/2020
02	Headworks and Influent Pump Station	$ 3,734,000	9/15/2023
03	Grit Chamber	$ 797,000	1/2/2022
04	Sludge Dewatering	$ 4,673,000	1/2/2022
05	Tertiary Filtration	$ 107,000	1/2/2022
06	UV Disinfection	$ 3,735,000	10/2/2020
07	Recycled Water Program/High Qlty Eff	$21,227,000	1/2/2030
	Total 20-Year CIP	**$47,551,000**	

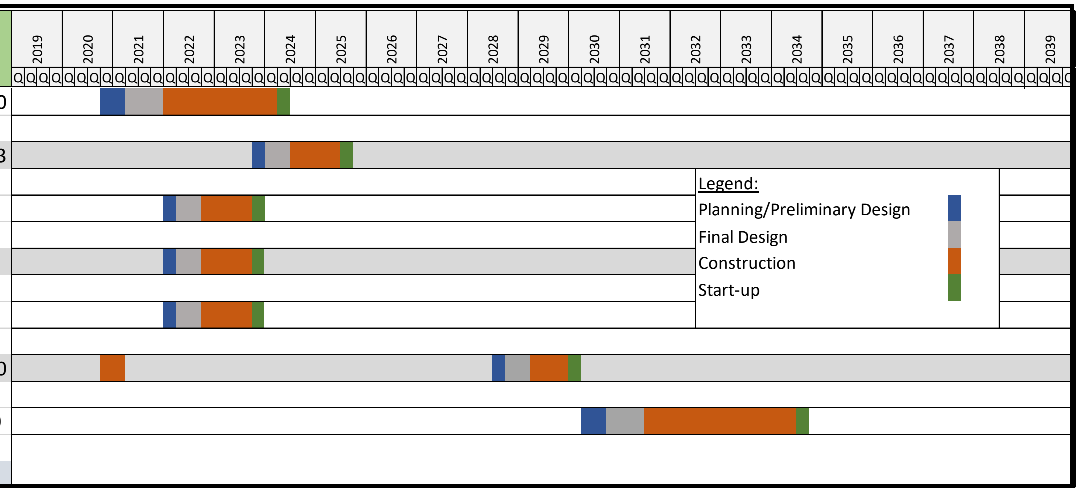

Figure 9 Preliminary 20-year CIP Schedule

Appendix A

TECHNICAL MEMORANDA

- TM 1: Existing Facilities Process Modelling
- TM 2: Existing Facilities Hydraulic Profile
- TM 3: Total Maximum Daily Load Requirements
- TM 4: Future Regulatory Requirements
- TM 5: TMDL Implementation and Facilities Upgrades
- TM 6: Condition Assessment

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 1
EXISTING FACILITIES PROCESS MODELING

REVISED FINAL | August 2020

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 1
EXISTING FACILITIES PROCESS MODELING

REVISED FINAL | August 2020

Carollo Project No. 11321A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2020.09.17 10:15:21-07'00'

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

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Technical Memorandum 1

EXISTING FACILITIES PROCESS MODELING

1.1 Introduction

This technical memorandum (TM) is the first in a series of six that will form the basis of the 20-year Facilities Plan for Ojai Valley Sanitary District (OVSD). TM 1 includes a review of historical plant influent data, and an evaluation of the existing facilities at the Wastewater Treatment Plant (WWTP). The WWTP evaluation includes review of historical performance, developing a plant process model, and determining the capacity of the major unit processes.

1.2 Key Findings and Recommendations

The key findings and recommendations are:

- Current plant influent constituent concentrations are higher than the 1996 design data. Corresponding average influent total suspended solids (TSS), biological oxygen demand (BOD), and ammonia concentrations are 25, 38, and 28 percent greater, respectively, than the 1996 design basis. This is due to severe drought conditions and state mandated water conservation efforts over the last decade that have reduced influent flow and significantly increased wastewater constituent concentrations.
- This study determined that the existing WWTP's Secondary Treatment capacity, assuming current discharge permit requirements, with updated loading parameters is 2.5 mgd under average annual dry weather flow (ADWF) conditions, with both clarifiers in service. It is assumed that both clarifiers will be in service during wet weather.
- The limiting process component is the solids loading rate of the clarifiers. This solids rate is based on a 90th percentile sludge volume index (SVI) obtained during the period of 2011 through 2018 of 220 milliliters per gram (mL/g).
- At lower SVI values, closer to 100 mL/g, the reliable plant capacity (with one clarifier out of service) with winter peak conditions would be closer to 2.0 mgd. This indicates the importance of controlling the SVI and keeping it as low as possible.
- To help control the SVI and prevent the large swings in SVI observed over the eight-year data period, it is recommended that the plant switch to solids retention time (SRT) control, from the current method of mixed liquor suspended solids (MLSS) control. This will help to stabilize the biomass population to match the influent loads and seasonal variations. As a result, the MLSS concentrations should vary during the year to reflect the changing conditions in the aeration basins. This will also help to maintain the phosphorus accumulating organisms (PAO) population and produce more consistent biological phosphorus removal (Bio-P) performance.
- Calibrating the BioWin model was somewhat challenging due to the differences between the influent wastewater quality characteristics and those for typical domestic sewage. This could be due to the presence of industrial effluent and the long sewer system. Accordingly, a separate study was undertaken by OVSD to assess the influent characterization. Specifically, the sampling study involved determining the fractions of

the chemical oxygen demand (COD) such as soluble, biodegradable and non-biodegradable fractions, as well as the fractions of the nitrogen species. The study was undertaken in September 2019 and a week of daily flow based composite samples were collected from the drainage basins and submitted for analysis. The results showed that the assumptions made for calibration of the BioWin model were reasonable. However, it was recommended that the updated COD fractionation information be used to re-calibrate the process model during the design phase of the project so that process design parameters can be fine-tuned. It was also recommended that further evaluation of the sewer system between sample point C01-C07 and the plant be undertaken to identify other sources of water that may be entering the system. This recommendation followed a high COD value that could not be explained. The results of the sampling exercise are included in Appendix A.

1.3 Influent Characteristics

Plant influent constituent concentrations and loading patterns were determined for this 2019 Facilities Plan by analyzing plant influent data for the eight-year period of 2011 through 2018. As expected, recent influent constituent concentrations have increased relative to the 1996 design basis because of reduced water consumption within the service area. Reduced consumptive water use (i.e., the portion of potable water that enters the household sewer) provides less dilution of the organic and nutrient loads from residential and commercial sources and results in increased concentration of most wastewater constituents.

Table 1.1 summarizes the average and 90th percentile plant influent conditions determined for this project together with the values from the 1996 design data. With the exception of phosphorus, the average annual influent concentrations of the measured parameters increased by up to 38 percent compared to the 1996 design data, depending on the parameter concerned. The average annual data shown in Table 1.1 for 2019 is the average data for January through November 2018.

The 5-day BOD test (BOD_5), TSS, and Total Kjeldahl Nitrogen (TKN) showed the largest increases in concentration compared with the 1996 design values. Ammonia nitrogen (ammonia-N or NH_3-N) was the odd one out, showing only a 3 percent variation with the 1996 design value.

The reason for the significant reduction in phosphorus concentrations from the 1996 design data, for both soluble (42 percent) and total (20 percent), is not clear. Figure 1.1 shows influent total phosphorus (TP) and influent soluble phosphorus (orthophosphate) and indicates that the values have been relatively stable for the last eight years.

Table 1.1 Plant Influent Conditions

	1996 Design Data		2019 Facilities Plan(1)		% Change With Respect to:	
	Annual Average	90th Percentile	Annual Average	90th Percentile	Annual Average Data	Annual 90th Percentile Data
TSS, mg/L	320	520	401	514	25%	-1%
BOD_5, mg/L	240	294	332	442	38%	50%
TKN, mgN/L	40	55	51	78	28%	42%
NH_3-N, mgN/L	32	40	33	50	3%	25%
Total P, mgP/L	10	11	8	10	-20%	-9%
Soluble P, mgP/L	7	9	4	5	-42%	-44%
Alkalinity, mg/L as $CaCO_3$	325	340	337	360	4%	6%

Notes:
Abbreviations: calcium carbonate=$CaCO_3$; mg nitrogen per liter=mgN/L; mg phosphorus per liter=mgP/L; phosphorus=P.
(1) Based on data for January to November 2018.

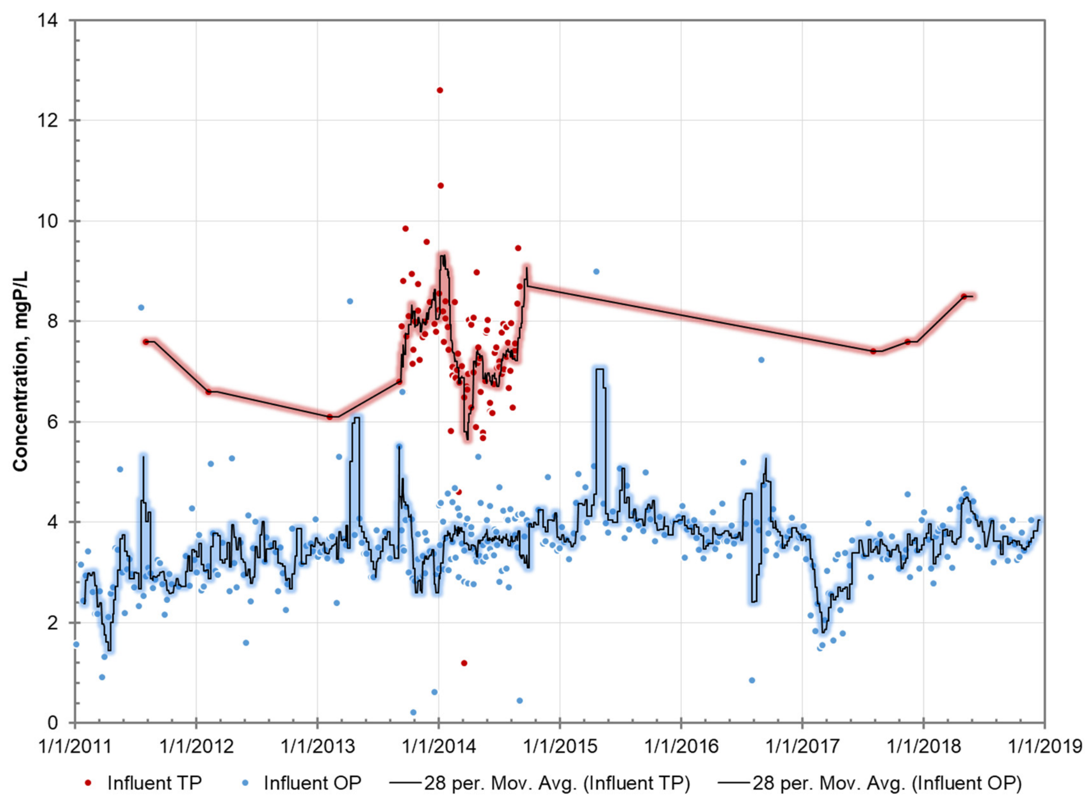

Figure 1.1 TP and Soluble P Concentrations to the WWTP

1.3.1 Comparison with Other Southern California Agencies

Figure 1.2 shows the influent wastewater flow and constituent concentration (TSS, BOD, and Ammonia) for OVSD and two plants located in Riverside County (Plant A and Plant B) for the period of January 2011 through January 2019.

As shown, there was a significant drop in daily influent flow in the aforementioned period for all three plants. Based on the trend lines, the flow to the OVSD's plant decreased by 36 percent, which was higher than Plants A and B (24 percent and 29 percent respectively), but not too different. The reduction of daily influent wastewater flow can be attributed to general increases in water conservation and a further reduction in water use mandated by the state of California that took effect in April 2015 during the most recent drought period.

Figure 1.2 shows a steady increase in TSS over the eight-year period for OVSD. However, a steady and slow decline in TSS over the same period can be seen for the other two plants. The 28-day moving average of the BOD and Ammonia-N concentrations show that both of these parameters increased during 2011-2019 for all three plants, although the rate of increase was different in each case. This pattern is in agreement with reduced water consumption within the service areas. Reduced consumptive water use (i.e., the portion of potable water that enters the household sewer) provides less dilution of the organic and nutrient loads from residential and commercial sources.

The purpose of this comparison was to determine whether the flow and influent quality trends observed for OVSD are in line with other Southern California agencies. The trends on Figure 1.2 seem to indicate that to be the case for all parameters presented, with the exception of TSS. TSS shows an increasing trend for OVSD and a flat or slightly declining trend for the other two plants. There is no clear reason for this difference. Overall, the trends at OVSD seem to be in line with those at the two other plants included in the comparison.

1.4 Existing Facilities

The WWTP is a tertiary plant with a dry weather design capacity (1996) of 3 million gallons per day (mgd) and an instantaneous peak flow capacity of 9 mgd. Untreated wastewater is collected from the City of Ojai, the unincorporated communities of Meiners Oaks, Mira Monte, Oak View, Casitas Springs, Foster Park, and North Ventura Avenue area through approximately 120 miles of sanitary sewer lines.

The WWTP provides a high level of treatment with nutrient removal, filtration, and disinfection. The treatment plant process includes influent grinding, grit removal and screening; activated sludge treatment using oxidation ditches with anaerobic, anoxic and aerobic zones for BOD, nitrogen, and phosphorus removal; secondary sedimentation, tertiary filtration, ultraviolet (UV) disinfection, and reaeration through static aerators prior to discharge. As a backup, the WWTP can use chlorination to disinfect the effluent. Equalization basins allow for evening out diurnal flows to the tertiary filters. A schematic of the treatment plant is shown on Figure 1.3. The tertiary facilities were designed for an average flow of 3 mgd and a peak flow of 4.3 mgd. Treated effluent is discharged at Discharge Point 001 to the Ventura River.

1.4.1 Secondary Treatment

The WWTP secondary treatment process achieves nutrient removal utilizing oxidation ditches that incorporate anaerobic, anoxic, and aerobic zones. The anaerobic zones provide Bio-P removal, the anoxic zones provide denitrification, and the aerobic zones provide soluble carbon removal and nitrification.

Figure 1.4 shows a more detailed schematic of the secondary treatment system. Influent flows reaching the WWTP go through screens and a grit chamber at the headworks before being routed to the secondary treatment. At secondary treatment, the influent flows through three anaerobic tanks in series. Micro-C, a commercially available external carbon source, is added to the third anaerobic tank to aide in the nitrification/denitrification process (NDN). This Micro-C addition was not part of the original design and was an Operations staff idea to test a second anoxic zone for lower effluent nitrogen. Testing by staff also included Micro-C addition to the dedicated anoxic zone. After the flow leaves the anaerobic tanks it enters two identical parallel oxidation ditches which are sectioned into anoxic and aerobic zones (Figure 1.4). The aerobic zone utilizes surface aerators to supply air to the biomass and support the nitrification process. Flow from both oxidation ditches are combined in the mixed liquor splitter box and flow via gravity to two 85-foot diameter clarifiers. A portion of the clarifier underflow is sent to dewatering and the remainder is routed back to the first anaerobic tank as the return activated sludge (RAS) flow.

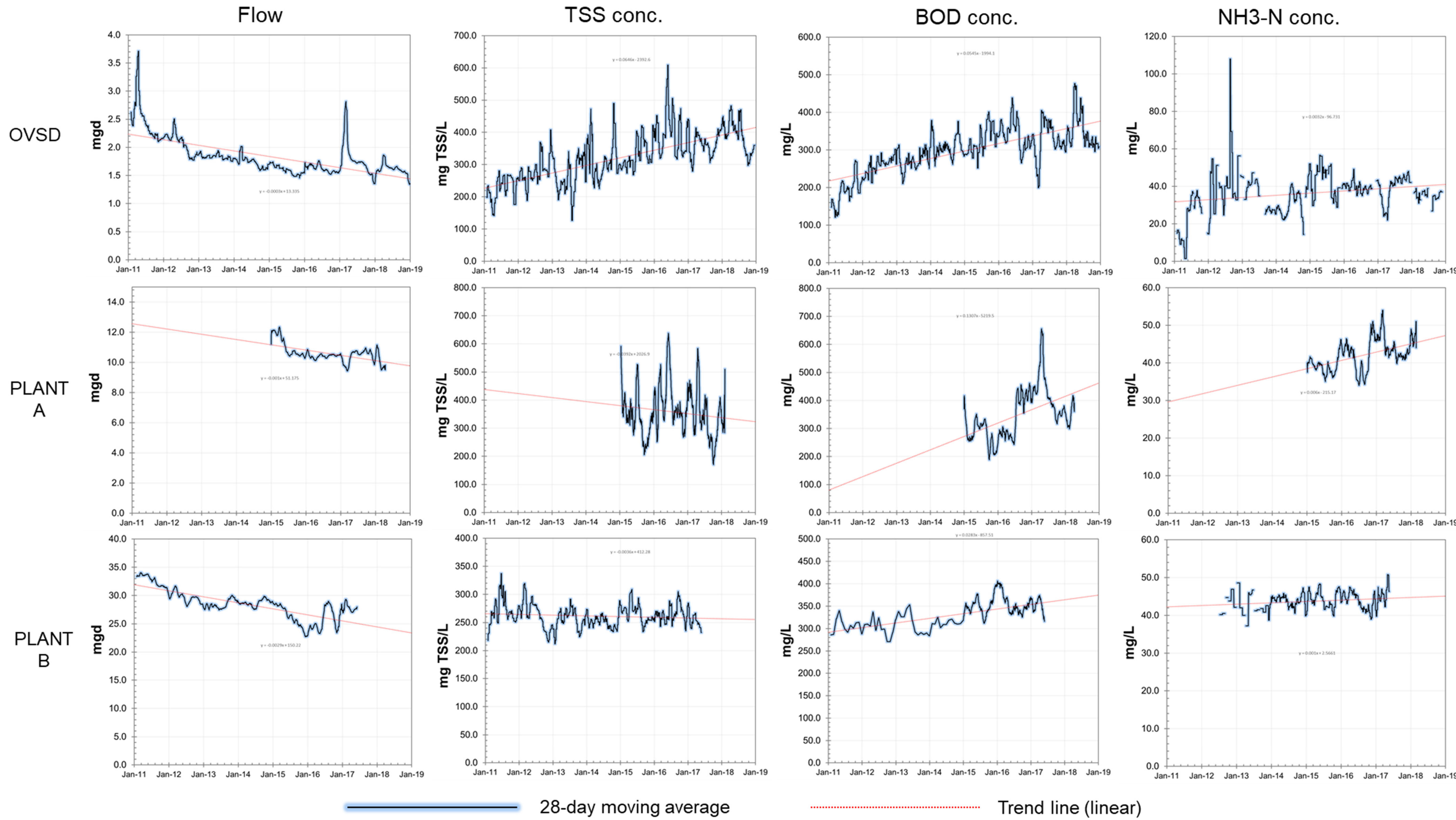

Figure 1.2 Influent Flow, BOD Concentration, TSS Concentration, and Ammonia Concentration for OVSD, Plant A, and Plant B

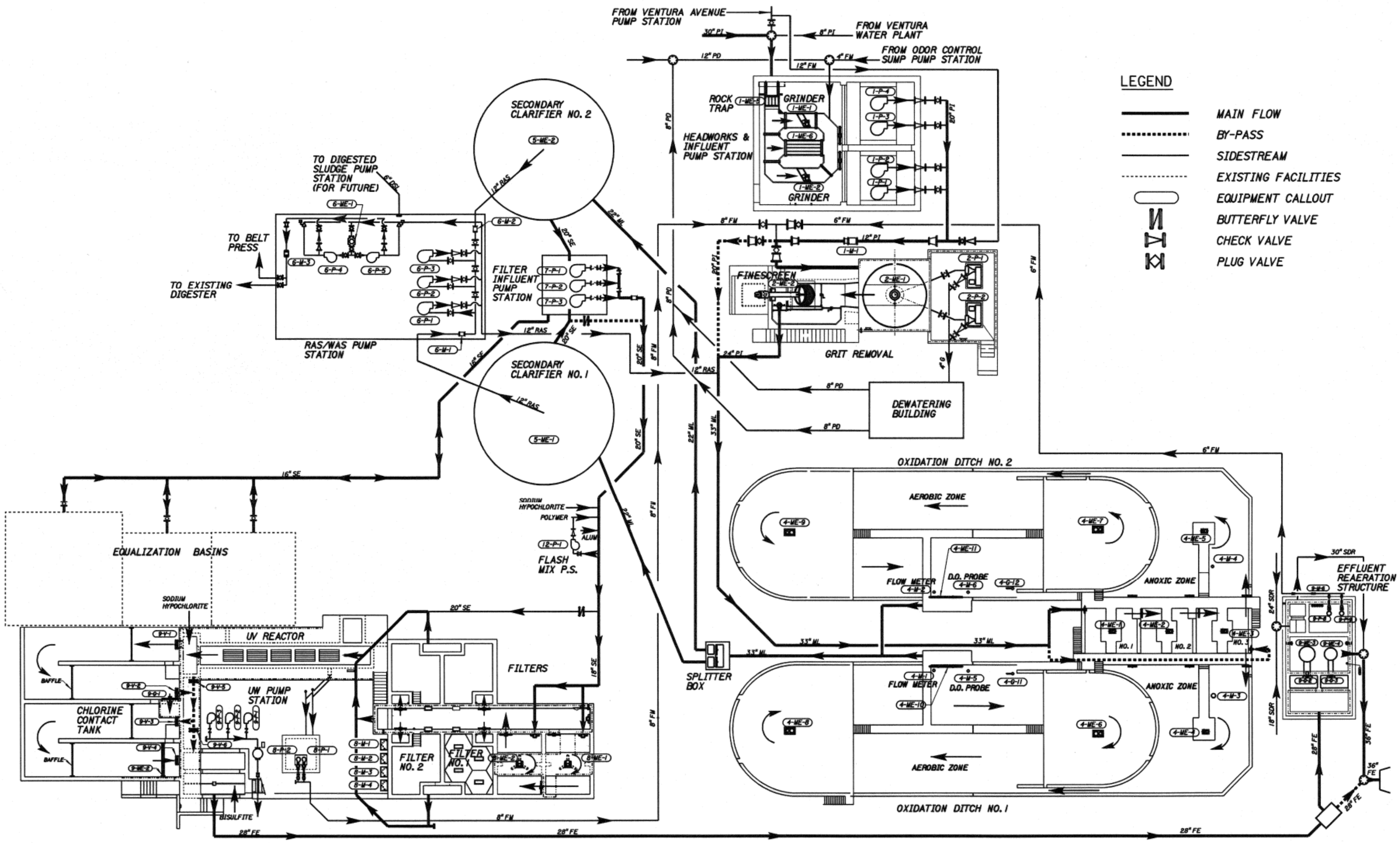

Figure 1.3 Liquid Process Flow Diagram for Ojai Valley Sanitary District

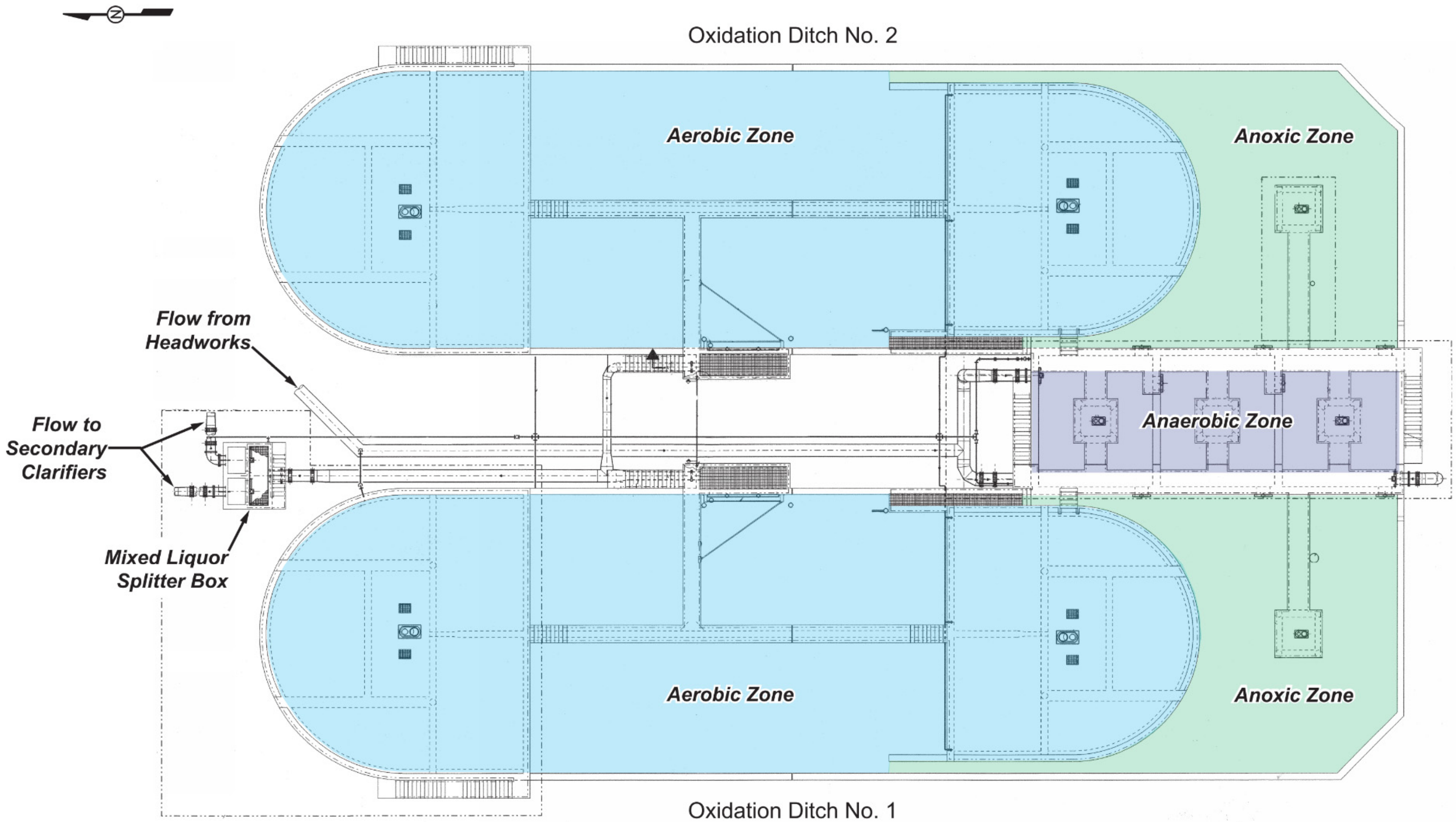

Figure 1.4 Schematic of Secondary Processes at Ojai Valley Sanitary District's WWTP

1.4.2 Performance Assessment

This section summarizes the historical treatment performance of the WWTP. Daily operating data from January 2011 through December 2018 were reviewed for the assessment. Discussions were also held with staff to identify operational issues.

The performance assessment is comprised of two main sections:

- Overall treatment performance of the WWTP with respect to meeting discharge limits and other effluent requirements.
- Historical load and performance of each of the major unit processes.

An understanding of the WWTP's current treatment performance is critical to determining the treatment capacity of the WWTP. Based on historical load and performance, recommended criteria for assessing capacity were developed for each major treatment process. The recommended criteria serve as the basis for the process capacity assessment.

1.4.2.1 Overall Performance

The WWTP currently receives higher concentrations of influent wastewater constituents compared to when the plant was built in 1996, but any resulting stress to nutrient removal is mitigated by a decreasing influent flow trend.

Although both BOD and TSS concentrations have increased, Figure 1.5 shows that TSS and BOD loads to the WWTP over the last eight years have stayed relatively flat at around 5,000 pounds per day (lb/d) for TSS and BOD.

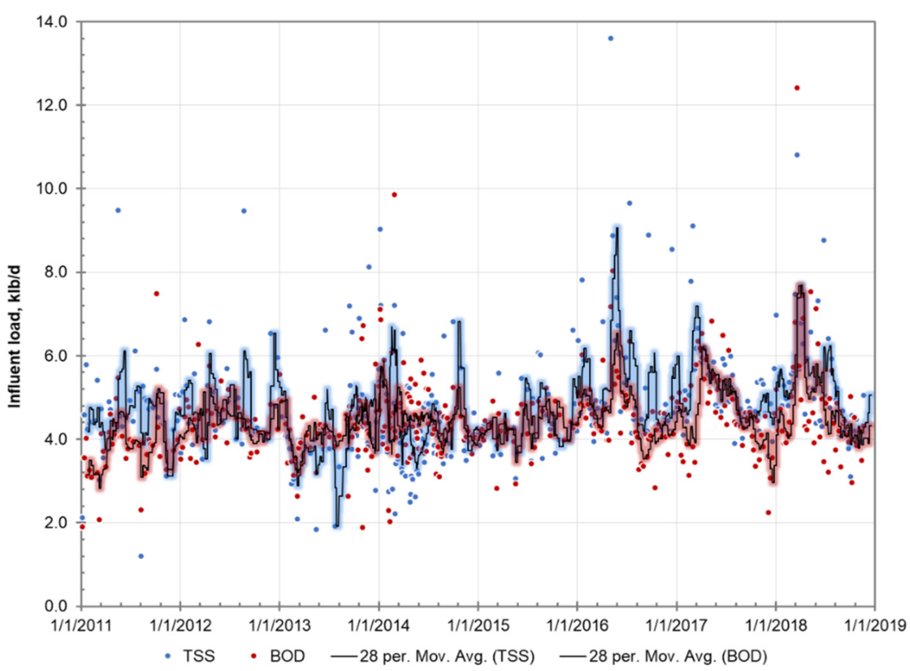

Figure 1.5 TSS and BOD Loading to the WWTP

Figure 1.6 shows TKN as N and ammonia-N loading rate for the plant. The ammonia-N data is somewhat variable and does not show a distinctive increasing or decreasing trend. The influent TKN data is sparse, but consistent sampling was done in 2014 and 2017, and there is not much variation in TKN load for these periods. A 30 percent increase in the Ammonia-N: TKN fraction was seen when comparing data from 2014 to the 2017 sampling period. Discussion with OVSD staff revealed that in 2015 they started receiving 100,000 gals/d from Dairy Farmers of America, an industrial user. That is approximately 6 percent of OVSD's current influent flow, and the addition of this industrial effluent could explain the increase in the inorganic nitrogen fraction observed in the 2017 data. Regular monitoring of TKN should be done.

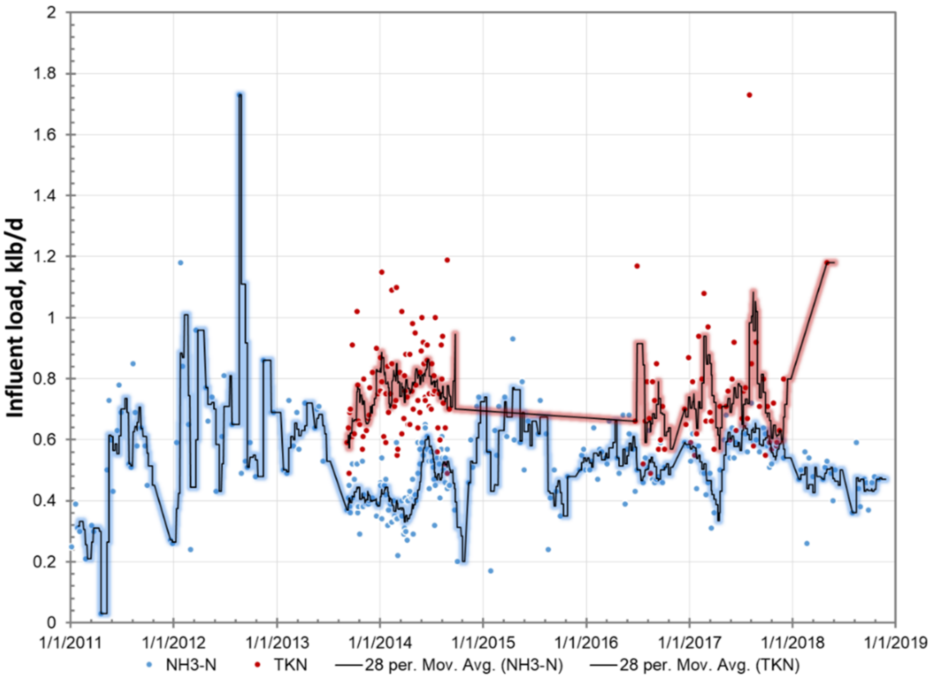

Figure 1.6 TKN as N and Ammonia as N Loading to the WWTP

Figure 1.7 shows that secondary treatment NDN performance has been good and stable, and most of the time the plant has been well below the interim discharge limit of 7.6 mg/L Total Nitrogen (TN).

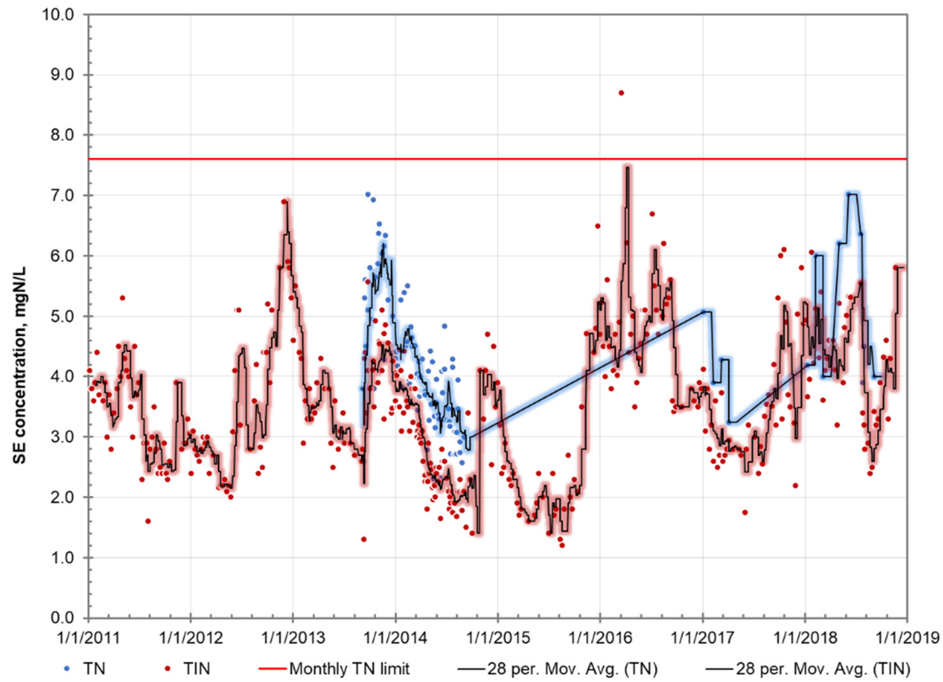

Figure 1.7 WWTP's Secondary Effluent TIN and TN, Plus Interim NPDES Limit

Figure 1.8 shows good Bio-P performance at the WWTP with secondary effluent soluble phosphorus values that remained below 2 mgP/L for a majority of the eight-year period. However, regular spikes in secondary effluent soluble phosphorus during the fall months can be seen on Figure 1.8.

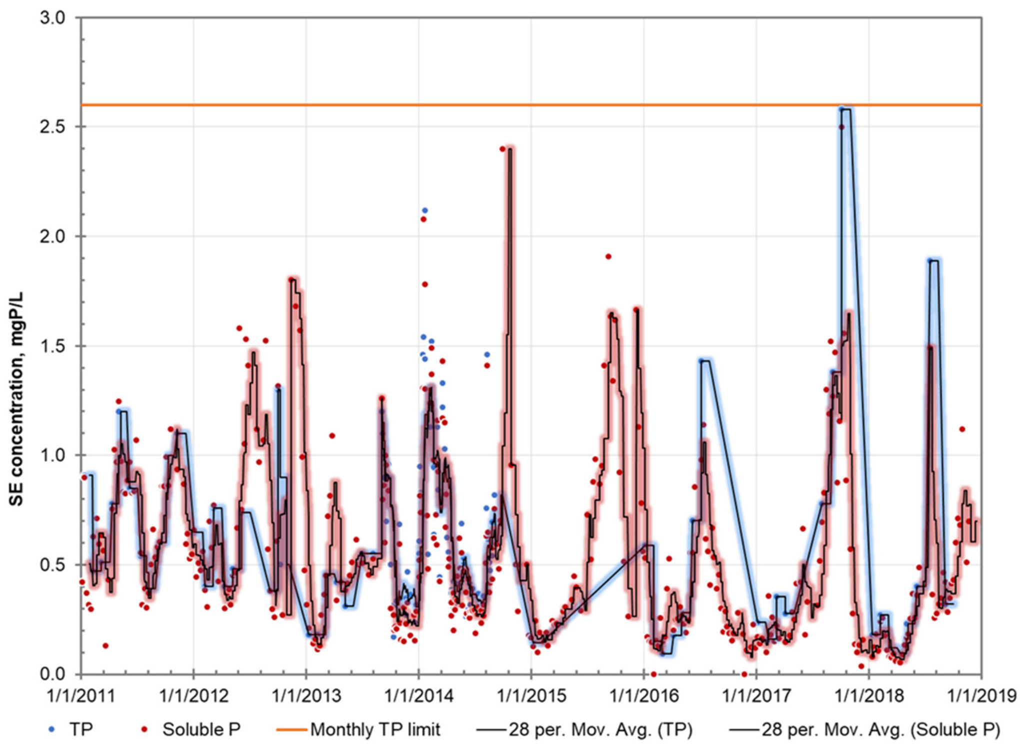

Figure 1.8 WWTP's Secondary Effluent Total and Soluble Phosphorus

These soluble phosphorus spikes trend with SVI values (Figure 1.9). Bio-P is well documented to promote good settling characteristics. PAO communities thrive in an ideal Bio-P environment and are denser than regular bacteria found in the activated sludge process. This helps form denser flocs that settle better and have lower SVI values. Figure 1.9 shows that the plant has operated at low SVI values (below 100 mL/g) for periods of time, but there are also periods when the SVI values increase to over 200 mL/g. For all of 2018 the SVI values were low (around 100 mL/g).

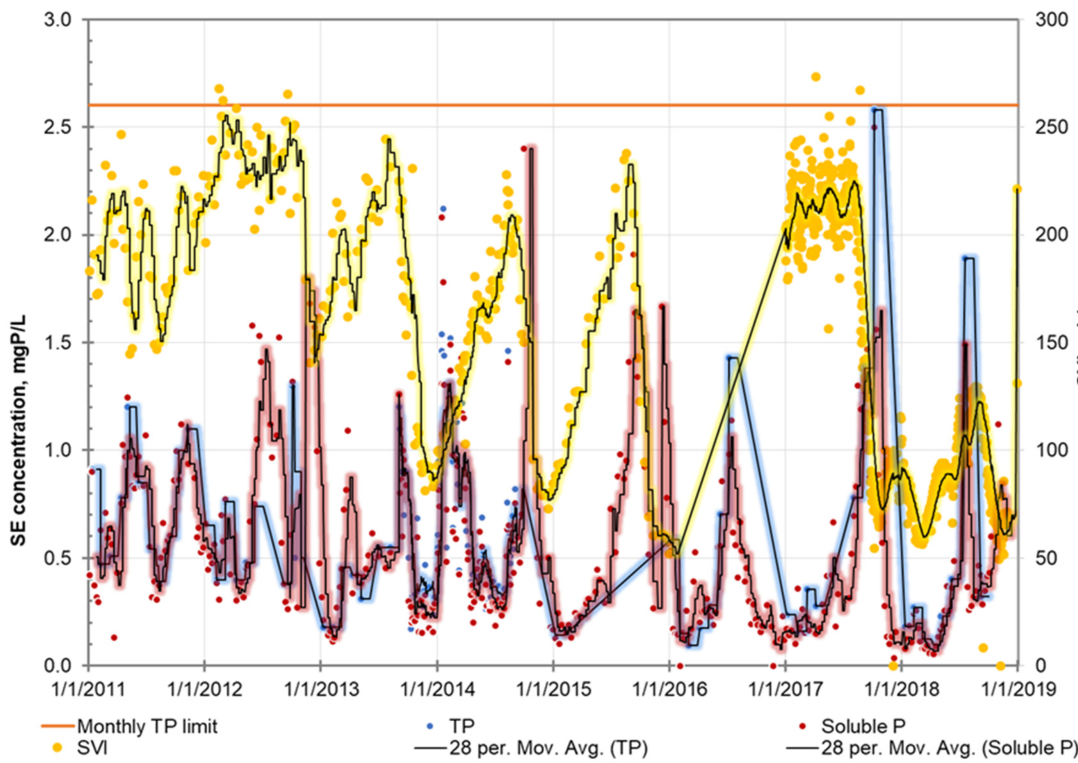

Figure 1.9 WWTP's Secondary Effluent Total and Soluble Phosphorus and SVI Values

The data suggests that the plant falls out of ideal Bio-P performance which results in both higher SVIs and higher secondary effluent soluble phosphorus concentrations. Discussions with WWTP staff on January 23, 2019, revealed that wasting rates are increased during the summer months to reduce the biomass inventory. The more aggressive wasting during warmer months should not upset the PAO community, as they require a minimum SRT of only two days. It is possible, and perhaps may be more likely, that the lower sewer flows that are being experienced increase the detention time in the sewers. A longer detention time combined with warmer temperatures in the summer provides ideal conditions for the sewer system to become a large bioreactor. This would result in consumption of available internal carbon sources before the wastewater reaches the WWTP, and this lack of internal carbon in the warmer months might be causing deteriorating Bio-P performance and resultant higher SVIs.

This is important to understand because, in the future when the TMDL limits are imposed, stable performance of the plant will be critical for meeting the more stringent effluent discharge limits.

1.4.2.2 Process Performance

This section summarizes the historical process load and treatment performance of all major processes at the WWTP. The historical load and performance of each unit process was compared to the original design criteria and industry accepted operating and performance criteria. The performance of each unit process provides a benchmark for the planning of new facilities and assessing capacity. In some cases, historical performance confirms that original design criteria are appropriate for assessing unit process capacity. In others, above or below average performance warrants using criteria different from the original design for assessing capacity. For each unit process through secondary treatment, recommended criteria are provided for use in the capacity assessment. Tertiary filters and UV disinfection were not included in the process analysis. Table 1.2 summarizes the key load and performance data as well as the recommended criteria for the capacity assessment.

Table 1.2 WWTP Unit Process Performance and Criteria Summary

Process/Design Parameter	Design Parameter	Units		2011-2018 Performance	MOP-8(1) or Typical Values(2)	Recommended Criteria for Capacity Analyses
Grit Removal (Non-Aerated)	Diameter of Grit Chamber	mgd at AA		-	-	3.7(2)
Drum Screen	Flow Rate at Peak Wet Weather Flow	mgd		5.83		6
Oxidation Ditch	SRT	d		12.5 – 27.6 Avg = 19.5	-	22
	MLSS	g/L		2.3 – 4.5 Avg = 3.2	2.0-4.0(1)	4.0
Secondary Clarifiers	Max Surface Over Flow Rate	gpd/sq ft		75 – 513 Avg = 151	400-700(1)	400
	Max Solids Loading Rate (at SVI 220 mL/g)	lbs/sq ft/d		3 – 15 Avg 5	30	21.4(3)
	Max Solids Loading Rate (at SVI 130 mL/g)	lbs/sq ft/d		3 – 15 Avg 5	30	35(2)(4)
Dewatering Belt Press	Pounds of Sludge per Hour	lbs/hr at 1%		587 - 1,229	-	1,800

Notes:
Abbreviations: average=Avg; annual average=AA; day=d; grams per liter=g/L; gallons per day=gpd; gallons per day per square foot=gpd/sq ft; hour=hr; pounds per hour=lbs/hr; pounds per square foot per day=lbs/sq ft/d.
(1) Design of Municipal Wastewater Treatment Plants Fifth Edition, Water Environment Federation/America Society of Civil Engineers, 2010.
(2) Typical values based on Carollo Experience.
(3) Based on SVI of 220 mL/g and MLSS concentration of 3,400 mg/L.
(4) Based on SVI of 130 mL/g and MLSS concentration of 3,700 mg/L.

1.4.3 Process Model Development

This section summarizes the secondary process model development. A steady-state process model was developed to assess the capacity of the aeration basins for a range of operating scenarios. Modeling results were compared to historical operating data to confirm proper calibration of the model. The three-month historical period from June 2018 to August 2018 showed recent stable plant performance and was used to calibrate the model. In subsequent deliverables, the steady-state process model will be used to evaluate plant-wide process alternatives.

The activated sludge simulator, BioWin 5.3 (Flamborough, Ontario, Canada), was used to model the WWTP under steady-state conditions. A schematic of the model configuration is shown on Figure 1.10.

In BioWin, the wastewater is divided into particulate and soluble fractions. Each fraction is further divided into biodegradable and non-biodegradable portions. When performing a simulation, the values for COD, TP, TKN, and inorganic suspended solids (ISS) are inputted. NH_3-N, BOD_5, and TSS are derived from the TKN, COD, and ISS based on inputted fractions.

F_{bs} = fraction of COD that is readily biodegradable.

F_{xsp} = fraction of COD that is slowly biodegradable.

F_{us} = fraction of COD that is unbiodegradable soluble.

F_{na}= fraction of TKN that is NH_3-N = $NH_3\text{-}N_{inf}/TKN_{inf}$.

F_{nus} = fraction of TKN that is soluble unbiodegradable = $TKN_{eff\ sol}/TKN_{inf}$.

F_{up}, the COD fraction that is particulate unbiodegradable COD, is another important fraction, however, it cannot be directly calculated from the wastewater characterization data and is adjusted to match the influent wastewater characteristics and secondary effluent as part of the calibration process.

As a first step in calibrating the steady-state plant wide model, a simulation of June 2018 to August 2018 performance was carried out using BioWin default values for particulate, soluble, biodegradable, and non-biodegradable fractions for influent COD.

This calibration period was selected because it was the most recent period (within one year of the commencement of this study) that displayed the least variation in influent flows and loads, solids inventory, Bio-P and NDN performance. Furthermore, it was classified as a dry weather period with minor variability (±3.4 percent) in influent flows (no rainfall).

Table 1.3 shows how the calibrated model COD fractions differed from the BioWin default COD values. The F_{bs} and F_{up} fractions showed the greatest variation from the BioWin default values and can be attributed to the impacts of two external factors: industrial flows (Dairy Farmers of America) and long detention times in the sewers caused by low flows.

Table 1.3 Default and Calibrated BioWin COD Fractions for Raw Wastewater

Parameter	Default BioWin COD Fractions	Calibrated(1) Model Fractions	Remarks on Calibration
Fbs(2)	0.1600	0.0600	Low flows and long detention times in the sewer might be the reason for a lower fraction.
Fxsp	0.7500	0.8000	A higher slowly biodegradable fraction might be the byproducts of fermentation in the sewers.
Fus(3)	0.0500	0.0500	Standard fraction.
Fup(4)	0.1300	0.2200	Very high, this reflects the high TSS in the influent wastewater, maybe caused by growing biomass due to fermentation in the sewers.
Fna(5)	0.6600	0.6448	Fraction was uncharacteristically low(7) for calibration period. Overall there is unusually high variability in this fraction, and this might be due to industrial clients.
Fnus(6)	0.0200	0.0200	Standard fraction

Notes:
Abbreviations: fraction of readily biodegradable COD=Fbs; fraction of COD that is slowly biodegradable=Fxsp; fraction of COD that is unbiodegradable soluble =Fus; fraction of COD that is particulate unbiodegradable=Fup; fraction of TKN that is NH3-N = NH3-Ninf/TKNinf =Fna.
(1) Calibration was done using historical plant data from June 2018 to August 2018.
(2) Typically ranges from 0.05 to 0.25.
(3) Typically ranges from 0.04 to 0.16.
(4) Typically ranges from 0.07 to 0.22.
(5) Typically ranges from 0.50 to 0.75.
(6) Typically ranges from 0.00 to 0.07.
(7) The ratio averaged 0.7785 for the 2017-2018 period.

The F_{bs} fraction (fraction of readily biodegradable COD) had to be adjusted to less than half of the default value. This adjustment seems to fit with the long detention times in the sewers and resultant consumption of readily biodegradable COD present in the wastewater before it gets to the plant. The F_{up} fraction (fraction of COD that is particulate unbiodegradable) had to be increased to the maximum as part of the model calibration process. This may be the result of biomass growth in the sewers due to the consumption of the readily biodegradable COD fraction mentioned above.

The adjustments that were needed during the model calibration process indicate that the wastewater characteristics entering the plant are not "typical" of domestic wastewater. Because of this, it is recommended that a study be implemented to characterize the plant influent more closely and measure the COD fractions mentioned above. This will provide more confidence in the process modeling exercise. In addition, the study should investigate sewer dischargers and flows, particularly from non-domestic sources.

The impact of COD consumption in the sewers and the resulting seasonal variations in available carbon for Bio-P and NDN operation need to be understood and will become more important when the more stringent effluent discharge limits are applied.

External factors were not the only issue that had to be resolved when calibrating the model as there were also internal factors. These internal factors are classified as plant operational parameters that are easier to control and monitor than external factors but require knowledge of the WWTP's operational philosophy.

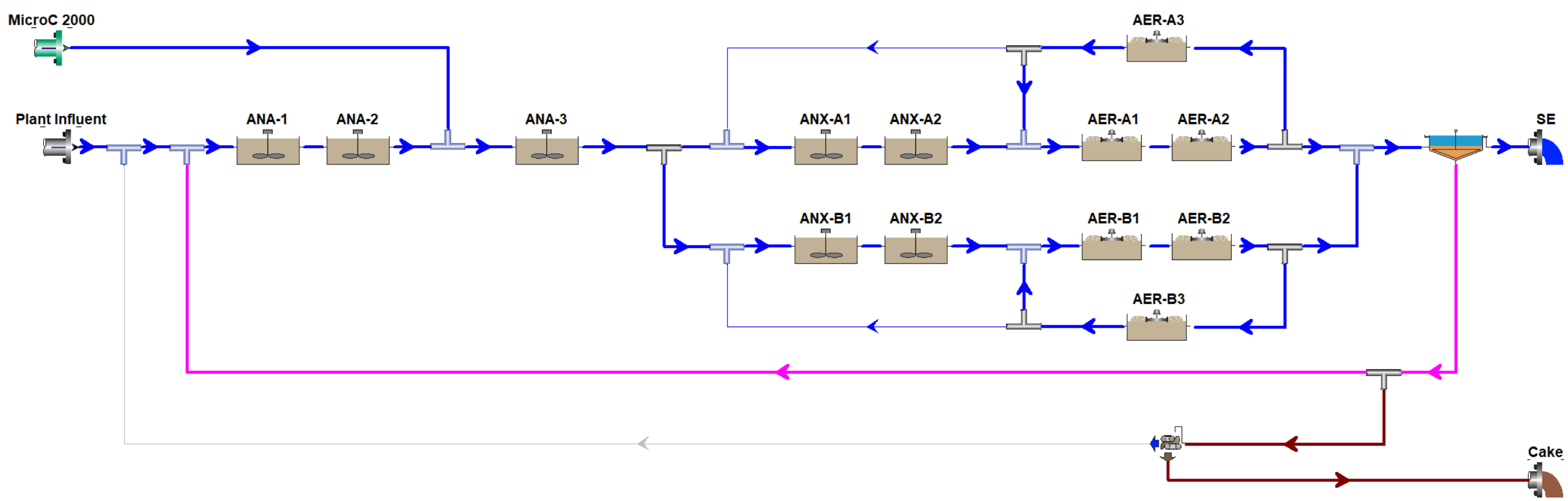

Figure 1.10 BioWin Schematic of the OVSD WWTP

When the calibration period was selected, it became apparent that the SRT of the plant varied significantly (Figure 1.11). Discussions with WWTP staff on January 23, 2019, revealed that Operators control the plant based on MLSS which results in SRT fluctuations that were noticed in the historical data. Figure 1.12 shows that this is the case and, over the past three years, the MLSS varied less than 10 percent. Maintaining a constant MLSS concentration will result in seasonal variation of the SRT.

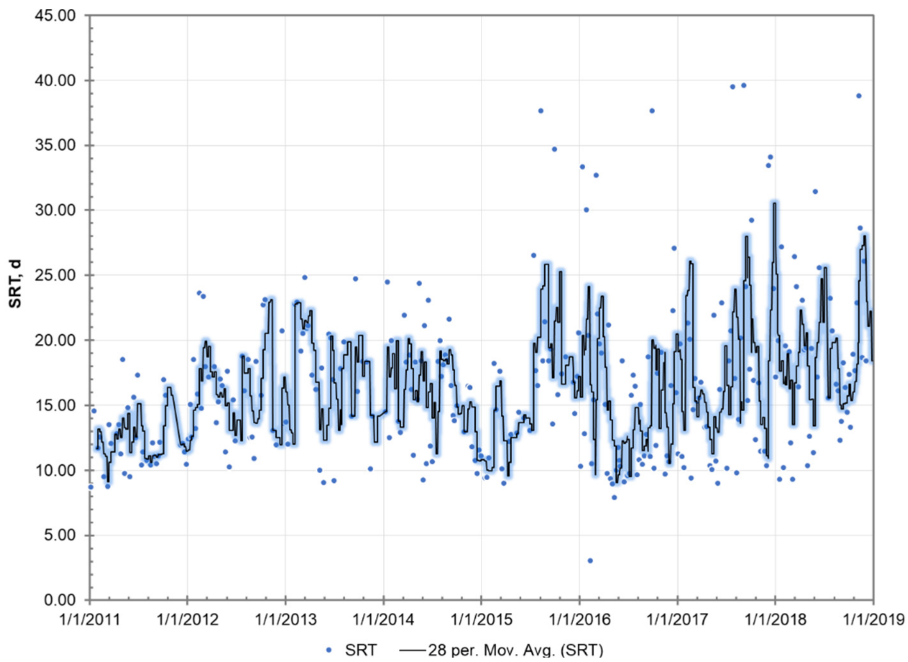

Figure 1.11 WWTP's SRT Values for the Eight-Year Period of 2011 Through 2019

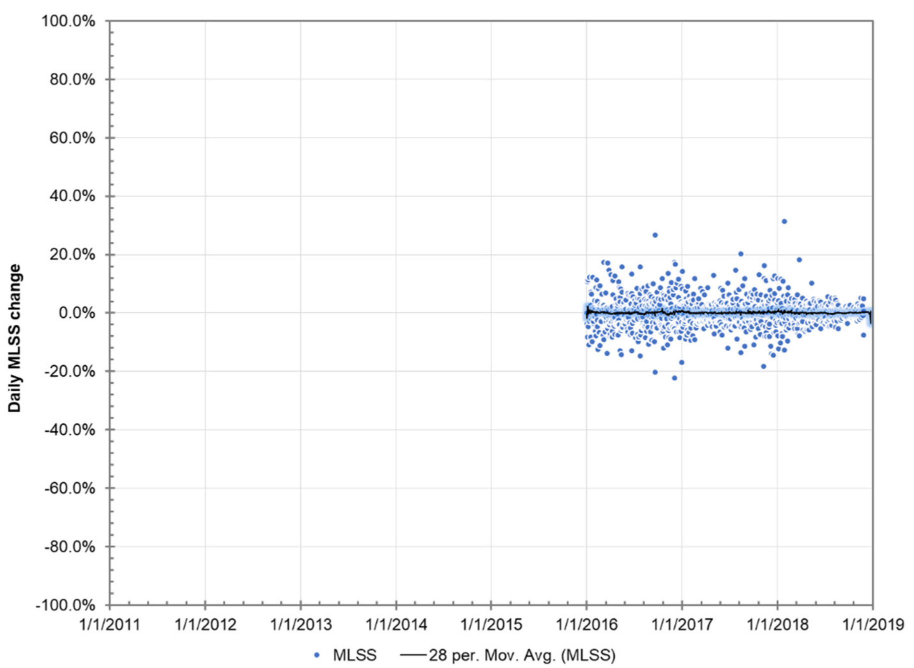

Figure 1.12 WWTP's Daily MLSS Change Values

Differences in a calibrated model to an average plant operation will be larger for a particular parameter if it varied more during the calibration period. This was the case with SRT, as it varied by over 30 percent, ranging from 40 days to as low as 10 days.

The calibrated model had a 17 percent difference to the plant average SRT for that period. This is considerable as the SRT is important when establishing potential optimization for Bio-P performance, NDN performance, and capacity. However, this difference is within the large variation witnessed when analyzing the plant data.

Based on these findings, it is recommended that the WWTP operators implement SRT control. There are several benefits that will result from SRT control, such as more stable SVI values and more stable Bio-P performance. The former will increase capacity significantly, and the latter will make it easier for the plant to meet future nutrient limits.

Additionally, Micro-C was added in the anoxic zone to promote nutrient removal. Table 1.4 summarizes the BioWin input parameters used for the calibration, the calibration output, and comparison to historical data.

Table 1.4 BioWin Calibration Summary

Parameters	Units	OVSD Data(1)	BioWin Calibration	Difference
Plant Influent				
Flow rate	mgd	1.63	1.63	-(2)
TSS	mg/L	401	398	-1%
VSS	mg/L	-	367	-
ISS(3)	mg/L	-	30	-
COD	mg/L	-	770	-
BOD	mg/L	332	324	-2%
Ammonia	mgN/L	33.2	31.9	-4%
TSS load	klb/d	5.4	5.4	1%
BOD load	klb/d	4.5	4.7	5%
NH_3 load	klbN/d	0.44	0.45	2%
Temperature	°C	24.8	24.8	-(2)
Plant Recycles				
Flow rate	mgd	-	0.02	-
Oxidation Ditch				
Micro C	gal/d	40	40	-
MLSS	mg/L	3,142	3,051	-3%
MLVSS	mg/L	2,545	2,569	1%
MLVSS fraction	Percent	81.0	84.2	4%
R(4)	-		0.18	-(2)
Total SRT	d	21.0	24.50	17%
Aerobic SRT(5)	d		18.50	-

Parameters	Units	OVSD Data(1)	BioWin Calibration	Difference
Secondary Effluent				
TSS	mg/L	0.9	0.8	-11%
BOD	mg/L	1.1	1.1	-6%
NH_3	mgN/L	0.02	0.22	-(2)
NO_2	mgN/L	0.01	0.06	-
NO_3	mgN/L	3.71	3.32	-10%
TP	mgP/L	1.15	1.20	5%
Waste Activated Sludge				
TS load	klb/d	4,380.0	3,400.0	-22%

Notes:
Abbreviations: degrees Celsius=°C; gallons per day=gal/d; thousand pounds per day=klb/d; thousand pounds of nitrogen per day=klbN/d; total solids=TS.
(1) Arithmetic average of reported operations and performance data from 6/1 through 8/31/2018.
(2) User-specified value equal to reported operations and performance data
(3) Inorganic suspended solids (i.e., TSS - VSS).
(4) RAS flow fraction, R, is the ratio of the RAS flow rate to the wastewater flow rate.
(5) Calculated using aerated fraction of aeration tank only.

All plant influent parameters in the calibrated model were a close fit to average plant data and did not exceed 5 percent in difference.

Oxidation ditch solids inventory was a close fit as well as the mixed liquor volatile suspended solids (MLVSS). This confirms that both the model and the plant data are in agreement over the active biomass responsible for NDN and Bio-P.

When modeling an oxidation ditch it is important to be aware that there are two recycles present. Flow circulates around the aerobic portion of the ditch and from the aerobic section to the anoxic portion of the ditch.

Secondary effluent parameters were a close fit to plant data with variation between plant and modeled TN and TP not exceeding 10 percent. Overall, the calibration was a success and there is enough confidence in the model to determine the secondary treatment capacity.

1.4.4 Capacity Assessment

This section summarizes the results of the capacity analysis. Capacities were estimated for each major unit process and are dependent on a range of parameters including flow, influent wastewater characteristics, treatment objectives (i.e., BOD_5 or ammonia removal, etc.), process configurations and limitations, and desired redundancy.

1.4.4.1 Average Daily Flow Capacity

The Average Daily flow capacity was estimated for facilities where sizing is established by influent BOD_5 and TSS load to the plant. Facilities that are sized based on influent BOD_5 and TSS load to the plant include the oxidation ditches and solids handling facilities. To determine the capacity for these facilities, the calibrated plant-wide process model was used to simulate maximum month conditions with Micro-C addition. The influent flow was increased until the recommended design criteria (as established in Table 1.2) were exceeded for each particular unit process. This influent flow was taken as the maximum month capacity limit for that particular unit process with all units in service. The maximum month capacity was converted to an equivalent Average Daily based on the historical peaking factors observed. The peaking factors were based on the 90^{th} percentile monthly average and were 1.13, 1.33, 1.25, and 1.15 for flow, TSS, BOD, and Ammonia, respectively. When evaluating the capacity in this scenario Micro-C addition was not varied, but kept at the rate reported at by OVSD staff (40 gallons per day).

Figure 1.13 summarizes the total average daily flow capacity for each process with all units in service. Under these average conditions, the SVI was assumed to be 130 mL/g based on average values measured at the plant over the period of 2011 through 2019. As shown in Figure 1.13, the plant capacity is calculated to be 2.5 mgd, controlled by the solids loading rate of the secondary clarifiers. Appendix 1B presents the State Point Analysis used to determine the clarifier capacity.

Although the plant is not subject to Title 22 reliability requirements currently, it could be in the future. Additionally, it is good practice to determine the plant's "reliable" capacity with one of the largest process units out of service, which in this case is a secondary clarifier. For regular maintenance, it was assumed, however, that process units would be taken out of service during dry-weather conditions only. Assuming no wet weather peak flow and one clarifier out of service, the reliable plant capacity would be around 2.8 mgd. But, because the peak flow capacity with all units in service (2.5 mgd) is the controlling value in this case, this represents the plant capacity.

If the 90th percentile SVI value is used (220 mL/g), the wet weather peak capacity (with all units in service) would decrease to 1.4 mgd as shown on Figure 1.14. At lower SVI values, closer to 100 mL/g, the reliable plant capacity (with one clarifier out of service) with winter peak conditions would be closer to 2.0 mgd. Note that these capacity limits are controlled by the sludge settleability, not hydraulics. From a hydraulic perspective the secondary clarifiers are not the limiting process. Further hydraulic analysis is presented in TM 2.

The above analysis confirms the importance of controlling the sludge SVI within the range similar to what has been achieved during 2018. This will become more important in the future when lower effluent limits need to be achieved to meet the TMDL.

1.4.4.2 Micro-C Addition

Process model scenarios were run without the addition of Micro-C. The capacity of the plant did not change significantly (decrease by 0.1 mgd). However, without Micro-C, the denitrification performance dropped, resulting in an increase in effluent nitrate by 1 mg/L to 4.3 mg/L. The plant data analyzed does not clearly indicate a carbon limitation but seasonal variation in performance in terms of NDN and Bio-P can be seen. Therefore, conducting the separate recommended study assessing influent characterization could bring to light periods of the year when higher or lower Micro-C dosage is necessary. Again, this will become more important when the more stringent TMDL effluent discharge limits are in force.

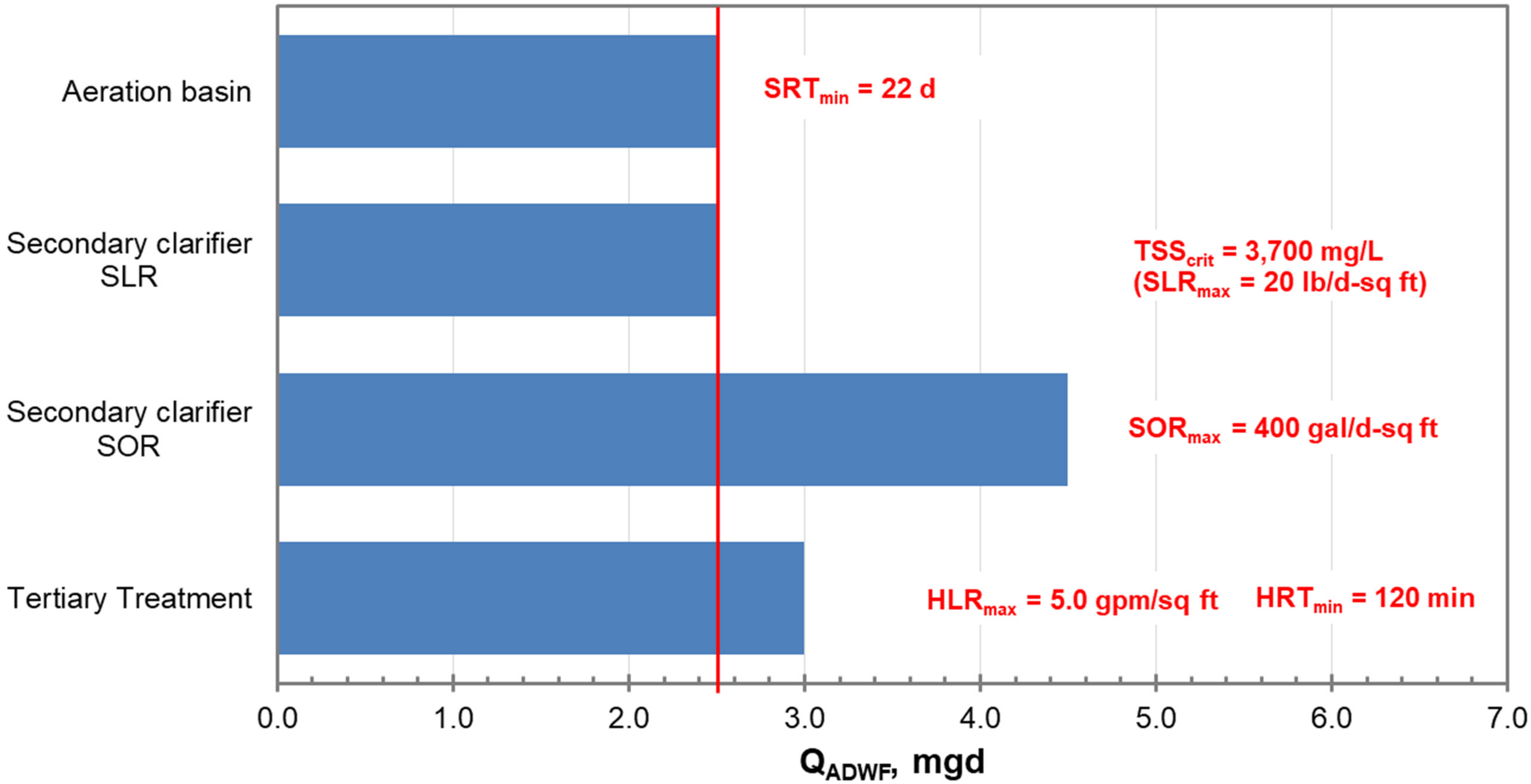

Figure 1.13 WWTP Process Capacity Using the Average SVI (130 mL/g)

1.5 Conclusions and Recommendations

From the analysis presented in this TM the following conclusions and recommendations can be made:

1. The current plant influent constituent concentrations are higher than the 1996 design data. Corresponding average influent TSS, BOD, and ammonia concentrations are 25, 38, and 28 percent greater, respectively, than the 1996 design basis. This is consistent with what other southern California agencies have experienced as a result of increased water conservation, prolonged drought conditions and additional state mandated water conservation efforts that have reduced influent flow and significantly increased wastewater constituent concentrations.
2. The existing WWTP's Secondary Treatment capacity, assuming current interim discharge permit requirements, with updated loading parameters is 2.5 mgd ADWF. This is based on an SVI of 130 mL/g which is the average of the eight-year analysis period and above the average obtained during 2018 and reflected in Figure 1.13. However, it is best practice to use the 90th percentile SVI of 220 mL/g. Figure 1.14 shows that if this approach is taken the capacity of the plant decreases to 1.4 mgd.
3. At lower SVI values, closer to 100 mL/g, the reliable plant capacity (with one clarifier out of service) with winter peak conditions would be closer to 2.0 mgd. This indicates the importance of controlling the SVI and keeping it as low as possible.
4. To help control the SVI and prevent the large swings in SVI observed over the eight-year data period, it is recommended that the plant switch to SRT control, from the current method of MLSS control. This method will help to stabilize the biomass population to match the influent loads and seasonal variations. As a result, the MLSS concentrations should vary during the year to reflect the changing conditions in the aeration basins. This will also help to maintain the PAO population and produce more consistent Bio-P performance.

Calibrating the BioWin model was somewhat challenging due to the differences between the influent wastewater quality characteristics and those for typical domestic sewage. This could have been due to the presence of the industrial effluent and the long sewer system. Accordingly, it was recommended that a separate sampling study be undertaken to assess the influent characterization. Specifically, the sampling study involved determining the fractions of the COD such as soluble, biodegradable and non-biodegradable fractions, as well as the fractions of the nitrogen species. The study was undertaken in September 2019 and a week of daily flow based composite samples were collected from the drainage basins and submitted for analysis. The results showed that the assumptions made for calibration of the BioWin model were reasonable. However, it was recommended that the updated COD fractionation information be used to re-calibrate the process model during the design phase of the project so that process design parameters can be fine-tuned. It was also recommended that further evaluation of the sewer system between sample point C01-C07 and the plant be undertaken to identify other sources of water that may be entering the system. This recommendation followed a high COD value that could not be explained. The results of the sampling exercise are included in Appendix A.

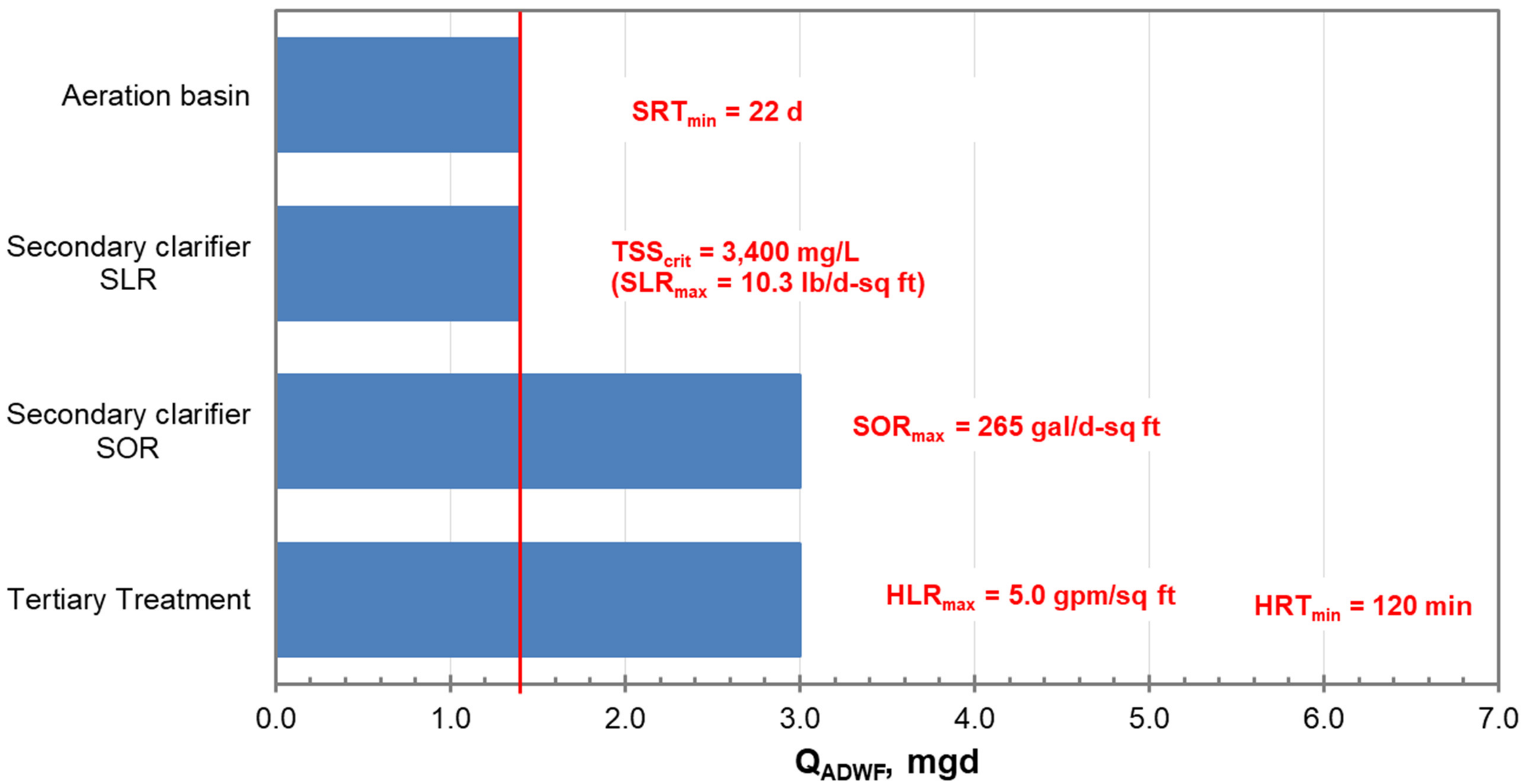

Figure 1.14 WWTP Process Capacity Using the 90th Percentile SVI (220 mL/g)

Appendix 1A

WASTEWATER CHARACTERIZATION SAMPLING PLAN AND RESULTS

Ojai Valley Sanitary District
Facilities Master Plan

Appendix 1A

WASTEWATER CHARACTERIZATION SAMPLING PLAN AND RESULTS

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Appendix 1A

WASTEWATER CHARACTERIZATION SAMPLING PLAN AND RESULTS

1A.1 Introduction and Background

Technical Memorandum (TM) 1, Existing Facilities Process Modeling concluded that the calibration of the BioWin model was somewhat challenging due to the differences between the influent wastewater quality characteristics and those for typical domestic sewage. This implied that the typical domestic wastewater characteristics were being modified either by the presence of an industrial effluent and/or the impacts of a long detention time in the sewer system. Accordingly, TM 1 recommended that a separate study to assess the influent characterization be undertaken.

This appendix outlines the recommended wastewater characterization sampling plan for the Ojai Valley Sanitary District (OVSD) Wastewater Treatment Plant (WWTP). The wastewater characterization sampling plan is designed to gather data to refine a steady-state process model that has been developed for the Master Plan. The calibrated model will be used to confirm the existing plant capacity, identify process requirements at future flows and loads, and evaluate treatment configurations to meet more stringent final effluent discharge criteria. In addition to this, this sampling plan is designed to better characterize wastewater from the Valley as well as from the Dairy Farmers.

The wastewater characterization sampling plan is designed to take place over four days. It is recommended that two of the days be week days, and the other two be a Saturday and Sunday. It is intended that all samples will be collected by plant staff and analyzed by the plant laboratory or by an outside laboratory where appropriate. In addition, OVSD will provide any automated samplers required to conduct the sampling. If any sampling and analysis is already conducted by OVSD, that effort does not need to be duplicated, but rather should be considered part of the execution of the sampling plan.

1A.2 Details

Determining the influent wastewater characterization would involve determining the fractions of the chemical oxygen demand (COD) such as soluble, biodegradable and non-biodegradable fractions, as well as the fractions of the nitrogen species. This information will help to make the process model as accurate as possible to improve the reliability of the model findings during the design to meet the anticipated performance with future flow and load to meet the new total maximum daily load (TMDL) discharge limits. This information will also allow OVSD to assess how current industrial clients are altering influent wastewater characteristics and determine impacts to the treatment of the influent flows if these industries were to halt sending flows to the WWTP.

The process model is designed to use chemical oxygen demand (COD) data, rather than biochemical oxygen demand (BOD) data, to quantify and define the organic strength of the wastewater. As a result, sampling data will be used to determine the composition of the influent COD (e.g., soluble or particulate, biodegradable or unbiodegradable). Identifying these different fractions is important so that sludge production rates can be accurately predicted. It is also important because the kinetics and rate of degradation of different COD fractions varies greatly. The rate of COD degradation affects the diurnal aeration oxygen demands within the basins and affects the final effluent quality.

Total Kjeldahl nitrogen (TKN), filtered TKN, ammonia nitrogen, and total oxidized nitrogen (TON) (i.e., nitrate and nitrite) are included in the sampling plan as this data is required to accurately model mixed liquor growth kinetics and nutrient concentrations. Alkalinity and pH are included in the sampling plan to account for pH effects on biological activity within the activated sludge system.

Four daily composite samples will be collected and analyzed to calculate average values of the various COD and nutrient fractions.

1A.3 Sampling Plan/Approach

It is recommended that on four days, composite samples be collected over a one-week dry weather period, with two days being during the week as well as on Saturday and Sunday. Dry weather in this instance means when the influent flow is close to the annual average value. At the time of this report's development, this value would be around 1.7 million gallons a day. Figures 1A.1 through 1A.3 of Appendix 1A summarize proposed sampling locations, sample preparation, and constituents to be analyzed for daily composite and daily grab samples.

All composite samples should be collected using an automated composite sampler (refrigerated). Composite sampling should be flow-paced where possible and where flow-paced sampling is not possible, the composite sampler should be programmed with a non-uniform, time-weighted frequency to simulate the approximate flow characteristics.

All grab samples should be immediately refrigerated to below 5 degrees Celsius after they are collected.

1A.3.1 Daily Sampling

Daily composite sampling conducted on four days (two weekdays and two weekend days) during a one-week period will cover two systems: Collection system and WWTP.

1A.3.1.1 Collection System Sampling

Daily composite samples are to be collected from six metering stations and confluence sites labeled in Figure 1A.1. Details on the analyses to be implemented are located in Figure 1A.2. These samples are necessary to characterize the influent wastewater, determine major sources of nutrient loadings, and the effects of detention times on the degradation of nutrients through biological or/and chemical processes.

1A.3.1.2 WWTP Sampling

Daily composite samples are necessary from two locations: Plant Influent (before recycle and plant drain) and Secondary Effluent. The information regarding these two samples is located in Figure 1A.2.

1A.3.1.3 Special Sampling

Part of the Facilities Plan considers potential future advanced treatment systems that may include reverse osmosis (RO) to treat some or all of the flow. In order to assist with that analysis it is proposed that one sample of Secondary Effluent be collected for additional analysis. Parameters to be included in the analysis would be general mineral parameters as well as silica. Details are shown on Figure 1A.2.

1A.4 Methods

Descriptions of the analytical methods required for the wastewater sampling plan can be found in Standard Methods for the Examination of Water and Wastewater, 21st edition (APHA et al., 2005) or in Methods for Chemical Analysis.

On-line temperature and pH meters should be used where available. Otherwise, temperature and pH should be measured on grab samples obtained when the composite or grab sample is collected.

Sample preparation (filtration) should occur immediately after collection before samples are analyzed in-house or shipped to an outside laboratory. Some tests are performed on both unfiltered and filtered samples. Two types of filters are used. For soluble COD, soluble BOD, and soluble TKN, 1.2- to 1.5-micron glass fiber filters are used (these are the same filters used for TSS/VSS analysis in the laboratory). For soluble ammonia, nitrite, and TON, 0.45-micron filters are used.

The flocculated/filtered COD (ffCOD) sample preparation should be performed in accordance with the procedure outlined in Mamais et al. (1993). A summary of the procedure is as follows:

> *Add 1 ml of a 100 g/L zinc sulfate solution to a 100 ml sample and mix vigorously with a magnetic stirrer for about one minute. The pH of the sample should then be adjusted to 10.5 with 6M sodium hydroxide (NaOH) solution while mixing gently, then allowed to settle quiescently for a few minutes. Clear supernatant should then be withdrawn with a pipette and passed through a 0.45-µm Millipore filter. The COD of the filtrate should then be determined to quantify the ffCOD of the sample.*

The Millipore filter should be triple rinsed with DI water before sample filtration to remove any starch binder that could bias the measured filtrate COD concentration. This sample preparation procedure is designed to flocculate any colloidal material so that the ffCOD concentration represents the "true" soluble COD concentration.

1A.5 Analysis of Results

Laboratory results for all four daily composite samples should be collected and then analyzed/evaluated. Statistical analysis will be carried out to determine the realistic parameter(s) that should be used in the model for confirmation of the process sizing and configuration at the start of preliminary design.

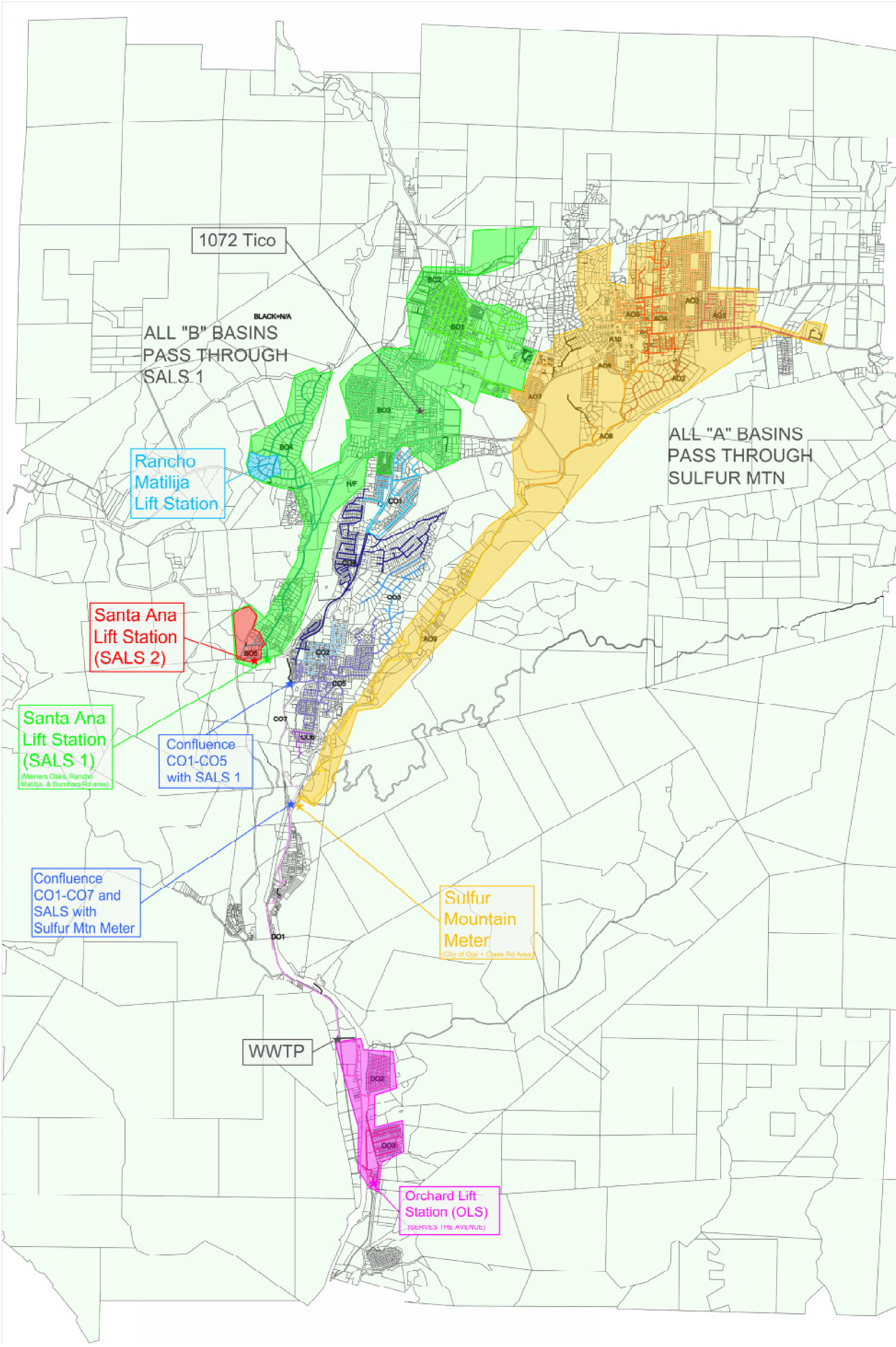

Figure 1A.1 OVSD's Overview of Collection System and Sampling Locations

Daily Composite Sampling Matrix

	Flow-Weighted 24-Hour Composite[5]	Q[0]	TSS[1]	VSS[1]	COD	sCOD[1]	ffCOD[2]	VFA[3]	BOD_5	$cBOD_5$	$scBOD_5$[1]	TKN	sTKN[1]	NH3-N	NO2-N	NO3-N	TP	OP[4]	pH[6]	Alk	Temp[6]
WWTP (before recycle and plant drain)	X	●	●	●	●	●	●	●	●	●	●	●	●	●			●	●	●	●	●
Santa Ana Lift Station 1 (SALS 1)	X	●	●	●	●				●	●		●	●	●			●	●	●	●	●
Sulfur Mountain Meter	X	●	●	●	●				●	●		●	●	●			●	●	●	●	●
Confulence C01-CO5 with SALS	X	●	●	●	●				●	●		●	●	●			●	●	●	●	●
Confulence C01-CO7 with SALS and Sulfur Mtn Meter	X	●	●	●	●				●	●		●	●	●			●	●	●	●	●
Orchard Lift Station	X	●	●	●	●				●	●		●	●	●			●	●	●	●	●
Secondary Effluent[7]	X	●	●	●	●	●	●	●	●	●	●	●	●	●			●	●	●	●	●

Notes
0 Record flow if measured only
1 Sample filtered through 1.2µ glass-fiber filter
2 Flocculated/Filtered COD
3 Volatile fatty acids (acetic, proponic, butyric, valeric)
4 Soluble reactive phosphorus ("orthophosphorus")
5 Manual composite OK if sampler not available
6 Grab sample OK
7 Special Sampling (Filtered sample). Parameters: Alkalinity, Ca, Mg, Na, SO_4, Cl^-, SiO_2, TDS, Conductivity

Figure 1A.2 Sampling Plan for Collection System

1A.6 Sampling Results

This section summarizes the preliminary findings of the sampling effort and its impact on any of the previous conclusions based on the BioWin model results.

1A.6.1 Collection Sample Analysis

Figure 1A.3 shows a map of the collection system and the sampling locations used in the sampling study. The District hired Weck Laboratories to conduct the sampling and do the analyzes. Sampling took place between September 12 and 21, 2019.

The collection system conveys flow from four basins (Basin A-D). There were 6 collection system locations sampled that reflect the contributions from the four basin areas:

1. Sulfur Mountain collection point receives flow from Basin A.
2. Santa Ana (SALs) collection point receives flow from Basin B.
3. C01-C05 collection point receives combined flow from Basins B and C.
4. C01 – C07 collection point receives combined flow from Basins A, B and C.
5. Orchid collection point receives flow from Basin D.
6. Plant Influent collection point receives combined flow from all four Basins.

Four days of daily flow proportioned composite samples were collected from each sampling point during a dry weather period, with two days being during the week as well as on Saturday and Sunday. Dry weather in this instance means when the influent flow is close to the annual average value, which would be around 1.7 million gallons a day. Flows during the sample period averaged 1.5 million gallons a day. This was an important consideration, since it minimizes the impact of external flows on the quality of water in the sewer collection system. Flow monitoring data was provided by the District.

In other words, sampling under average dry weather conditions provides the best opportunity to capture the representative quality from the domestic and industrial customers.

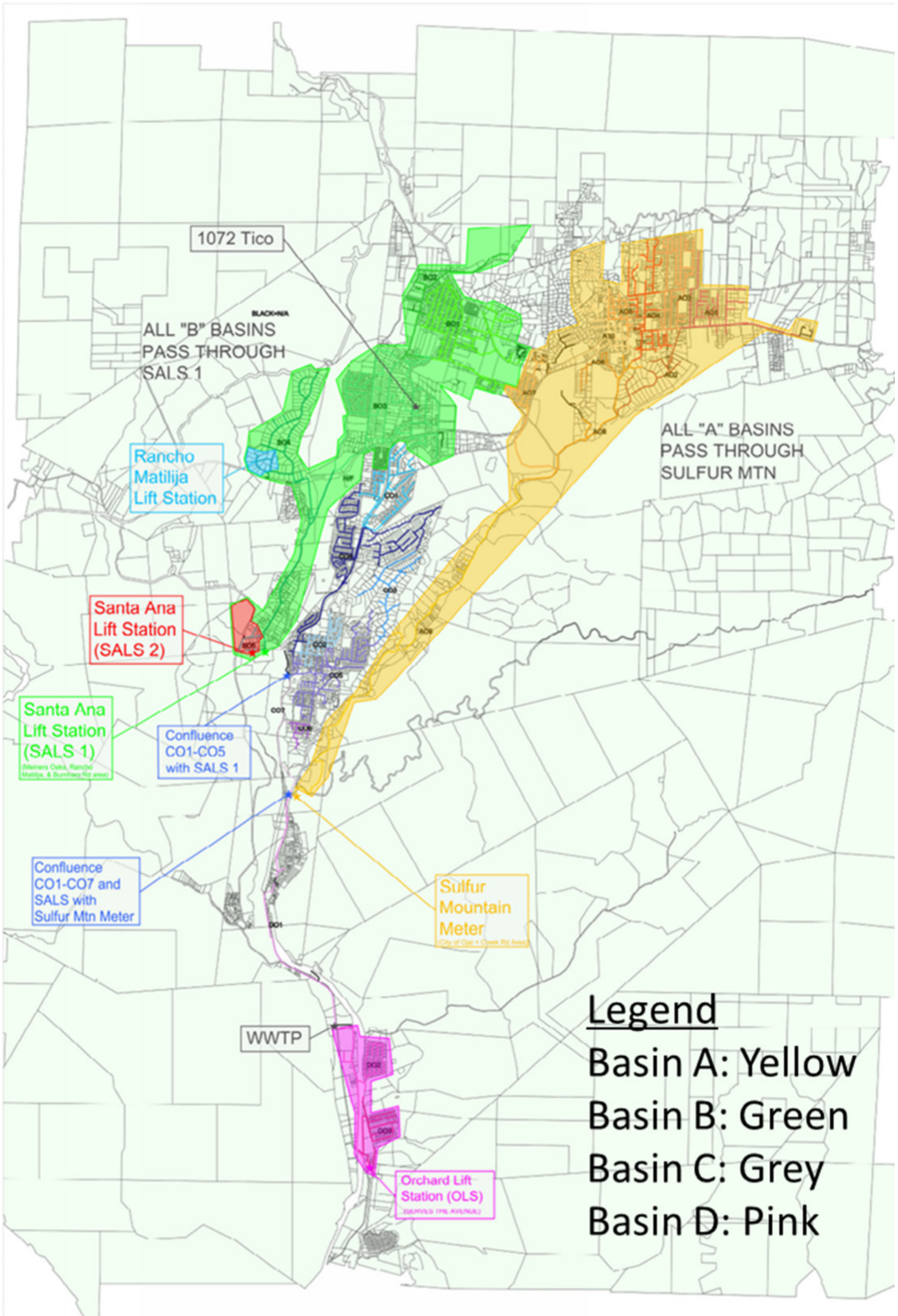

Figure 1A.3 OVSD's Overview of Collection System and Sampling Locations

Figure 1A.4 presents a schematic arrangement of the collection system and indicates the percentage of flow from the four Basins. As shown, Basins A, B and C contribute the greatest percentage of the flow, with Basin A having the largest percentage (34.8 percent), and Basins B and C having similar values of 24.7 and 27.7 percent, respectively. Basin D only contributes 12.8 percent of the total flow.

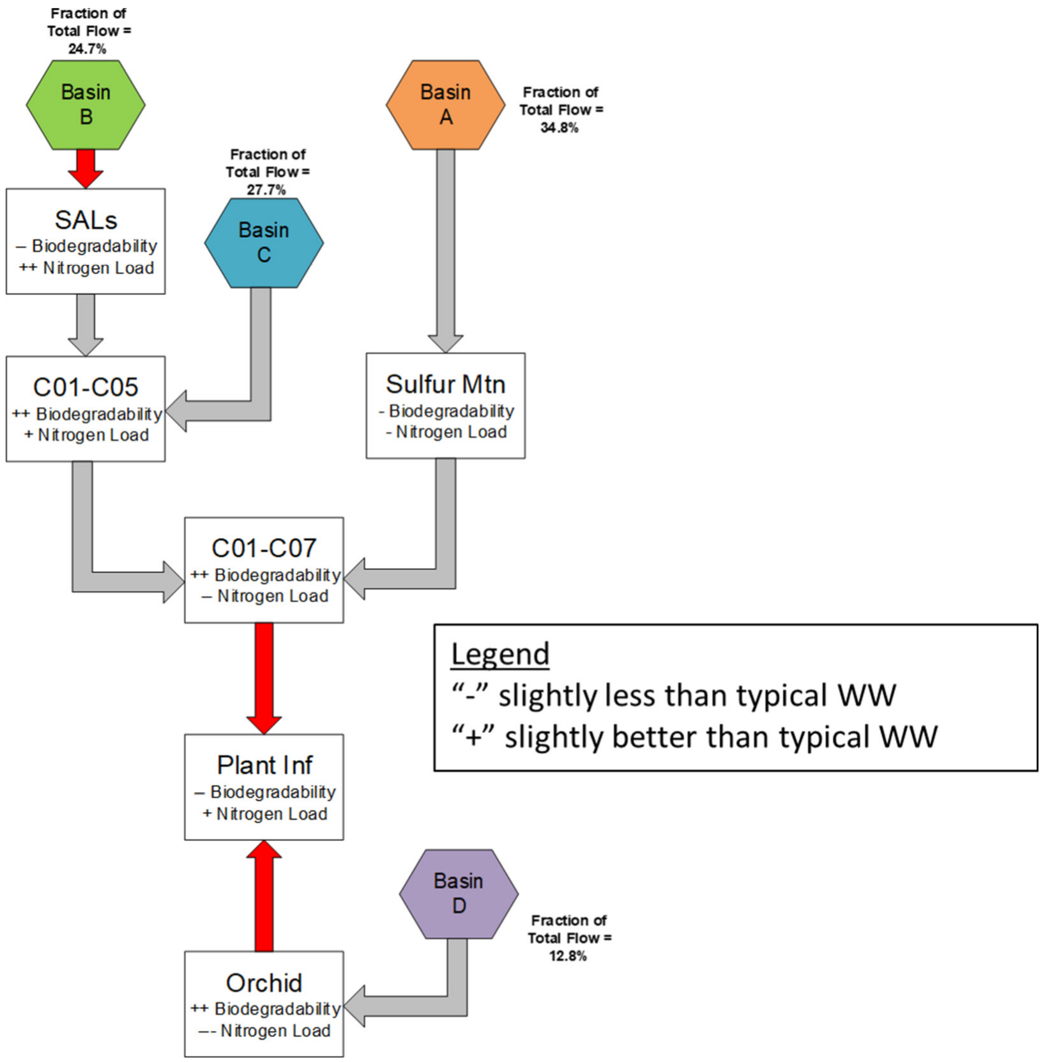

Figure 1A.4 Qualitative Summary of Collection System Sampling Results

Table 1A.1 presents the average results of four composite samples for COD, BOD, TSS, TKN, ammonia-N, and total phosphorus for the Basins and the sample locations. For Basin A, the BOD and TSS concentration are fairly typical for domestic wastewater, and if anything, could be considered on the low side based on current water conservation practices, with BOD of less than 200 mg/L. The COD to BOD ratio, see Table 1A.2 is slightly higher than a typical range of 2.2 to 2.4, indicating slightly poorer biodegradability. This may be due to the size of the sewer shed and length of time flow takes to get to the sample location. The average total phosphorus value is relatively low.

For Basin B, the average BOD value was lower than for Basin A, and the COD was higher, indicating both a "weak" wastewater and one with poorer biodegradability; COD/BOD ratio of 3.3 (Table 1A.2). Basin B shows high ammonia compared to the other sampling points, with an average value above 50 mg/L. Actual composite samples ranged between 40 and 63 mg/L. The total phosphorus value is in the typical range.

Combining Basins B and C (at sample point C01-C05) in roughly a 50:50 blend increased the average BOD, lowered the average COD and resulted in a lower COD/BOD ratio of 1.9 (Table 1A.2). This suggests that the Basin C flow is "fresher" and more biodegradable.

Table 1A.1 Summary of Average Collection System Sampling Results for Typical Parameters

Sampling Location and Basin	COD mg/L	BOD mg/L	TSS mg/L	TKN mgN/L	Ammonia-N mgN/L	Total Phosphorus mg/L
Sulfur Mountain – Basin A	520	193	200	41.8	33.5	5.5
SALs – Basin B	573	173	185	63.3	53	10.5
C01-C05 – Basins B and C	393	205	167	54.0	43.5	5.7
C01-C07 – Basins A, B and C	513	308	303	62.0	49.5	8.3
Orchid – Basin D(1)	803	435	248	37.5	22.8	8.1
Plant Influent – Blend of all Basins(2)	753	233	360	64.8	33.2	9.2

Notes:
(1) The Orchid sampling location was sampled between September 14 to 17, and from September 20 to 21. This was different to all the other sampling locations.
(2) Blend based on District-reported flow by basin.

When Basins A, B, and C are combined (sample point C01-C07), the average BOD and TSS values both increase, and the COD to BOD ratio of 1.7 indicates good biodegradability. Ammonia-N is high, nearly 50 mg/L. This is the last sampling point in the upper sewer shed before flow gets to the plant.

Table 1A.2 Summary of VSS and Sample Results Ratios

Sampling Location	%VSS -	COD/BOD -	BOD/TKN -
Sulfur Mountain – Basin A	0.95	2.7	4.6
SALs – Basin B	0.92	3.3	2.7
C01-05 – Basins B and C	0.91	1.9	3.8
C01-07 – Basins A, B and C	0.88	1.7	5.0
Orchid – Basin D	0.93	1.8	6.6
Plant Influent – Blend of all Basins	0.89	3.2	3.6

Notes:
(1) Blend based on District-reported flow by basin.

Basin D showed the highest average COD of 803 mg/L, and a high BOD of 435 mg/L. This results in a COD to BOD ratio of 1.8, indicating good biodegradability. The nitrogen load in this source is relatively low due to the average ammonia-N concentration of 22.8 mg/L, but the total phosphorus concentration is typical. This source includes flow from an industrial source.

Combining all flows at the plant results in average BOD and TSS values that are in the typical wastewater range, but the resulting average COD to BOD ratio of 3.2 is high. This indicates that on average the combined influent flow has a lower biodegradability than typical wastewater. This result is unexpected, since the average COD to BOD ratio for the two upstream components are both less than 2. This is discussed further in the section below. Included in Attachment 1 are more detailed results for each of the sampling locations.

1A.6.2 Discussion of Individual Sample Dates

Table 1A.3 shows when samples were collected for COD analysis in the two sewers that combine at the treatment plant, and the results for the composite samples taken at the plant. As shown, samples at the plant were taken on consecutive days between September 12 and 16, which matched the dates on which the C01-C07 samples and all other northerly samples were collected. However, the samples for Basin D were collected on the consecutive days between September 14 and 17 (three samples) and the fourth was collected for the period September 20 to 21. The significantly higher than normal COD at the plant for the September 12 to 13 period (1,300 mg/L) is assumed to have resulted from a very high value in from Basin D, because the combined flow from Basins A, B and C (which represents more than 85 percent of the flow) had a COD of 560 mg/L. However, as shown in Table 1A.3, there was no sample from Basin D for the period in question in order to confirm this. The BOD at the plant was only 150 mg/L when the high COD value was measured, implying a very low biodegradability wastewater.

Using the data for the periods of September 14 to 15 and 15 to 16, for which there is data for all three sample points, shows that a mass balance on COD based on a flow split of 12.8 percent from Basin D and the remainder from Basins A, B and C, gives values which are close to those reported at the treatment plant.

Back calculating, based on a mass balance, indicates that the COD in the Orchid sample would have to have been around 6,200 mg/L during September 12 to 13, to result in the value of 1,300 mg/L measured at the plant. While not impossible, the relatively stable COD values for the four sampled days for Basin D, do not support such a high variance in quality.

Table 1A.3 Variation of COD on Sampling Dates and Locations

Sampling Date	Treatment Plant	Orchid-Basin D	C01-07-Basins A, B and C
	Composite COD Values (mg/L)		
9/12 – 9/13	1300	-	560
9/13 – 9/14	690	-	560
9/14 – 9/15	580	800	540
9/15 – 9/16	440	820	390
9/16 – 9/17	-	840	-
9/17 – 9/18	-	-	-
9/18 – 9/19	-	-	-
9/20 – 9/21	-	750	-
Average COD	**753**	**803**	**513**

Table 1A.4 presents the same data from Table 1A.3, but now includes the measured BOD values and the calculated COD to BOD ratios. For the Orchid – Basin D samples, the COD to BOD ratio is consistently in a good biodegradable range. Again, there is no Basin D sample data for the period of September 12 through September 14, but as shown both COD and BOD results for the four days for which samples were taken, show little variation. The largest flow contributor to the plant influent, sample point C01-C07, shows a good COD to BOD ratio for two of the four days, but a low BOD for the September 12 to 13 sample, and corresponding high COD to BOD ratio of 3.1. The higher BOD than COD value for September 15/16 cannot be explained.

On examination of the COD to BOD ratio of the combined stream at the plant, the data shows very difficult- to-treat wastewater for September 12/13, when the high COD occurred, and then average to good quality thereafter. On average, the combined water quality from Basins A, B, and C shown in Table 1A.4, seems good. The Basin D water quality, for the days sampled, was also good. Yet, the quality of the combined streams at the plant has some outlier data, particularly for the September 12/13 sample.

From this data one can only speculate as to the cause of the change in quality between sample point C01- C07 and the plant. Flow from Basin D certainly contributes to the change in quality, but flow from Basin D is only 13 percent of the total, and the data collected shows consistent quality. It may be that some other sources are entering the sewer between sampling point C01-C07 and the plant, but these appear to be intermittent. This might be an area that requires additional investigation.

Table 1A.4 Variation of COD/BOD Ratio for Sampling Locations and Dates

Sampling Date	Treatment Plant			Orchid-Basin D			C01-07-Basins A, B and C		
	COD	BOD	COD/BOD	COD	BOD	COD/BOD	COD	BOD	COD/BOD
9/12 – 9/13	1300	150	8.7	-	-	-	560	180	3.1
9/13 – 9/14	690	300	2.3	-	-	-	560	290	1.9
9/14 – 9/15	580	250	2.3	800	400	2.0	540	280	1.9
9/15 – 9/16	440	230	1.9	820	460	1.8	390	480	0.8
9/16 – 9/17	-	-	-	840	420	1.8	-	-	-
9/17 – 9/18	-	-	-	-	-	-	-	-	-
9/18 – 9/19	-	-	-	-	-	-	-	-	-
9/20 – 9/21	-	-	-	750	400	1.9	-	-	-
Average	**753**	**233**		**803**	**435**		**513**	**308**	

1A.6.3 Sampling Results Summary

- Basin A drains to the Sulfur Mountain sampling point, and the data shows wastewater that has slightly poor biodegradability.
- Basin B drains to the SALs and was shown to have the lowest biodegradability compared with all other sample locations, with a COD to BOD ratio of 3.3. The poor biodegradability might be from long detention time in the sewers as that region does not have any major industrial clients.
- Basins B and C combine at sample location C01-C05. Biodegradability increased at this point and closely resembled standard domestic wastewater, implying that Basin C has no collection conveyance issues and/or is a highly biodegradable wastewater.
- Basin B drains to the Orchid lift station and sampling point and showed a relatively strong wastewater, but one that is highly biodegradable, and has the lowest ammonia-N load; both positive outcomes. All samples from Basin D were fairly consistent in quality.
- The data was not able to explain the very high COD of 1,300 mg/L measured at the plant between September 12/13, mainly because a sample from Orchid (Basin D) was not collected that day. Although it is possible that an exceptionally high COD from Basin D (around 6,200 mg/L) could have occurred that day and resulted in the high COD value at the plant, this seems unlikely. This conclusion is drawn because that data that was

collected for Basin D was relatively stable, showing little variation. This indicates that further evaluation of the sewer system between sample point C01-C07 and the plant is needed to identify any other sources of wastewater.

1A.6.4 Sampling Results Impacts on TMDL Project

It was mentioned earlier that a reason that the sampling exercise was recommended was to assess the influent characteristics and confirm the assumptions that had been made during the process modeling effort.

In the BioWin process model, the wastewater is divided into particulate and soluble fractions. Each fraction is further divided into biodegradable and non-biodegradable portions. When performing a simulation, the values for chemical oxygen demand (COD), total phosphorus (TP), TKN, and inorganic suspended solids (ISS) are inputted. NH_3-N, BOD_5, and TSS are derived from the TKN, COD, and ISS based on inputted fractions.

F_{bs} = fraction of COD that is readily biodegradable = $(ffCOD_{inf}-fCOD_{eff})/COD_{inf}$

F_{xsp} = fraction of COD that is slowly biodegradable

F_{us} = fraction of COD that is unbiodegradable soluble = $ffCOD_{eff}/COD_{inf}$

F_{na} = fraction of TKN that is NH_3-N = NH_3-N_{inf}/TKN_{inf}

F_{nus} = fraction of TKN that is soluble unbiodegradable = $TKN_{eff\ sol}/TKN_{inf}$

F_{up}, the COD fraction that is particulate unbiodegradable COD, is another important fraction, however, it cannot be directly calculated from the wastewater characterization data and is adjusted to match the influent wastewater characteristics and secondary effluent as part of the calibration process.

Table 1A.5 shows how the calibrated model COD fractions compare with the measured values obtained from the sampling data, and the BioWin default COD values. Each parameter is discussed briefly in the remarks column in the table.

Table 1A.5 Comparison of COD Fractions

Parameter	Default BioWin COD Fractions	Calibrated(1) Model Fractions	Sampling Study Fractions	Remarks on Calibration and Study
F_{bs}(2) Readily Biodegradable Fraction	0.1600	0.0600	0.0748	Differences in model assumptions and sampling analysis are minimal. Therefore, it is reasonable to conclude that low flows and long detention times in the sewer might be the reason for a lower fraction.
F_{xsp} Slowly Biodegradable Fraction	0.7500	0.8000	Conformed to Include Addenda	Process modeling is required to get an accurate number. The particulate fraction and biodegradability is approximately 30% higher and 20% lower, respectively, than standard domestic water. This implies that the BioWin assumption might be correct.

Parameter	Default BioWin COD Fractions	Calibrated(1) Model Fractions	Sampling Study Fractions	Remarks on Calibration and Study
F_{us}(3) Soluble Unbiodegradable Fraction	0.0500	0.0500	0.0100	Results from the study show that this fraction is considerably lower than typical values. The standard fraction was used during model development due to limited data on secondary effluent at the time.
F_{up}(4) Particulate Unbiodegradable Fraction	0.1300	0.2200	Process modeling required	Process modeling is required to get an accurate number. The particulate fraction and biodegradability is approximately 30% higher and 20% lower, respectively, than standard.
F_{na}(5)	0.6600	0.6448	--	Data was unavailable.
F_{nus}(6)	0.0200	0.0200	0.0125	Lower fraction than the default and reflects the higher biodegradable load produced by industrial clients.

Notes:
(1) Calibration was done using historical plant data from June 2018 to August 2018.
(2) Typically ranges from 0.05 to 0.25.
(3) Typically ranges from 0.04 to 0.16. (4) Typically ranges from 0.07 to 0.22.
(4) Typically ranges from 0.50 to 0.75.
(5) Typically ranges from 0.00 to 0.07.
(6) The ratio averaged 0.7785 for the 2017-2018 period.

1A.6.5 Summary of Impacts to Modeling Results

The preliminary analysis of the sampling study data indicates that the assumptions made for the fractionation of COD during the development of the process model calibration used to assess alternatives for achieving the TMDL requirements seem reasonable. Accordingly, the conclusions reached for the analyses presented in TM 5 for various process alternatives to achieve the TMDL appear to be valid.

However, it is recommended that the results from this study be used to re-calibrate the process model during the design phase of the project. At that point the model can be re-run to fine tune the process design parameters for the selected alternative.

1A.7 Recommendations

Based on this preliminary analysis of the collection system sampling and analysis, the following recommendations are made:

1. Further evaluation of the sewer system between sample point C01-C07 and the plant be undertaken to identify any other sources of wastewater entering the system. This recommendation is based on the fact that the COD value of 1,300 mg/L for the September 12/13 sample at the plant cannot be explained.
2. The results from the sampling study should be used to re-calibrate the BioWin process model during the design phase of the project. At that point the model can be re-run to fine tune the process design parameters for the selected alternative.

1A.8 References

APHA, AWWA and WEF. Standard Methods for the Examination of Water and Wastewater, 21st edition. American Public Health Association, American Water Works Association, and Water Environment Federation, Washington, D.C. 2005.

Mamais, D., Jenkins, D., and Pitt, P. "A Rapid Physical-Chemical Method for the Determination of Readily Biodegradable Soluble COD in Municipal Wastewater:" Water Research 27(1): 195 197. 1993.

Attachment 1

DETAILED RESULTS FOR SAMPLING LOCATIONS

WWTP Inf

Date Analyte	Sep 13, 2019 10:00 AM	Sep 14, 2019 11:00 AM	Sep 15, 2019 10:40 AM	Sep 16, 2019 10:30 AM	Average
Acetic acid	ND	ND	ND	ND	
Alkalinity as $CaCO_3$	650	610	260	450	493
Biochemical Oxygen Demand	150	300	250	230	233
BOD, Carbonaceous	110	380	220	240	238
BOD, Carbonaceous, Dissolved	62	57	41	88	62
Butyric acid	ND	ND	ND	ND	
Chemical Oxygen Demand	1300	690	580	440	753
COD, Flocculated and Filtered	59	69	73	80	70
COD, Soluble	60	83	78	87	77
Isovaleric acid	ND	ND	ND	ND	
NO_2+NO_3 as N	ND	ND	84	ND	84
o-Phosphate as P	3.5	3.1	3.7	3.4	3.4
pH	7.8	7.8	7.8	7.7	7.8
Phosphorus, Total	13	8.4	9.1	6.4	9.2
Propionic acid	ND	ND	ND	ND	
Temperature, Degrees F	78.8	78.4	76.4	72	76
TKN	85	52	68	54	65
TKN, Soluble	40	37	52	41	43
Total Suspended Solids	570	370	310	190	360
Volatile Suspended Solids	520	330	280	150	320

SE

Date Analyte	Sep 13, 2019 10:10 AM	Sep 14, 2019 11:20 AM	Sep 15, 2019 11:00 AM	Sep 16, 2019 10:45 AM	Average
Acetic acid	ND	ND	ND	ND	
Alkalinity as $CaCO_3$	260	250	260	250	255
Biochemical Oxygen Demand	ND	ND	ND	ND	
BOD, Carbonaceous	ND	ND	ND	ND	
BOD, Carbonaceous, Dissolved	ND	ND	ND	ND	
Butyric acid	ND	ND	ND	ND	
Calcium, Dissolved	103	99.4	101	102	101
Chemical Oxygen Demand	20	19	23	18	20
Chloride, Total	160	160	160	160	160
COD, Flocculated and Filtered	16	16	11	13	14
COD, Soluble	16	12	14	5.6	11.9
Isovaleric acid	ND	ND	ND	ND	
Magnesium, Dissolved	30.6	29.5	30	30.6	30.2
NO_2+NO_3 as N	3500	3300	3900	4000	3675
o-Phosphate as P	0.63	0.66	0.77	0.82	0.72
pH	7.6		7.8	7.8	7.7
Phosphorus, Total	0.76	0.84	0.92	0.94	0.87
Propionic acid	ND	ND	ND	ND	
Silica as SiO_2, Dissolved	20	20	20	19	20
Sodium, Dissolved	140	140	140	140	140
Specific Conductance (EC)	1500	1600	1500	1500	1525
Sulfate as SO_4	240	240	240	240	240
Temperature, Degrees F	33		84.7	0	39.2
TKN	1.1	0.77	1.1	1.1	1.0
TKN, Soluble	0.85	0.64	0.55	1.2	0.81
Total Dissolved Solids	880	900	890	880	888
Total Suspended Solids	5	3	1	5	4
Volatile Suspended Solids	ND	ND	ND	ND	

C01-05

Date / Analyte	Sep 13, 2019 08:25 AM	Sep 14, 2019 08:45 AM	Sep 15, 2019 08:50 AM	Sep 16, 2019 08:50 AM	Average
Alkalinity as CaCO3	300	490	410	400	400
Ammonia as N	44	44	42	44	44
Biochemical Oxygen Demand	190	210	220	200	205
BOD, Carbonaceous	150	230	150	190	180
Chemical Oxygen Demand	390	480	350	350	393
NO2+NO3 as N	ND	ND	ND	ND	
o-Phosphate as P	3.7	3.9	3.5	3.7	3.7
pH	8.2	8.1	8.2	7.9	8.1
Phosphorus, Total	6	6.2	5.1	5.3	5.7
Temperature, Degrees F	69.8	70.7	71.4	68	70.0
TKN	46	60	50	60	54
TKN, Soluble	42	43	39	48	43
Total Suspended Solids	260	250	71	87	167
Volatile Suspended Solids	240	230	69	67	152

C01-07

Date / Analyte	Sep 13, 2019 09:05 AM	Sep 14, 2019 09:30 AM	Sep 15, 2019 09:30 AM	Sep 16, 2019 09:25 AM	Average
Alkalinity as CaCO3	540	510	430	450	483
Ammonia as N	42	62	49	45	50
Biochemical Oxygen Demand	180	290	280	480	308
BOD, Carbonaceous	150	270	190	360	243
Chemical Oxygen Demand	560	560	540	390	513
NO2+NO3 as N	ND	ND	ND	ND	
o-Phosphate as P	3.5	4.5	3.9	3.9	3.95
pH	7.9	7.7	7.8	7.8	7.8
Phosphorus, Total	7.5	9.3	8.3	7.9	8.3
Temperature, Degrees F	78.6	76.6	71.7	77.4	76.1
TKN	54	78	63	53	62
TKN, Soluble	43	70	47	48	52
Total Suspended Solids	300	300	230	380	303
Volatile Suspended Solids	280	260	210	320	268

Orchid

Date / Analyte	Sep 15, 2019 10:15 AM	Sep 16, 2019 10:05 AM	Sep 17, 2019 10:00 AM	Sep 21, 2019 07:30 AM	Average
Alkalinity as $CaCO_3$	390	440	380	330	385
Ammonia as N	26	26	18	21	23
Biochemical Oxygen Demand	400	460	480	400	435
BOD, Carbonaceous	300	490	460	390	410
Chemical Oxygen Demand	800	820	840	750	803
NO_2+NO_3 as N	ND	ND	ND	200	200
o-Phosphate as P	4.4	4.2	3.9	4.5	4.25
pH	7.1	7.5	6.9	7.5	7.25
Phosphorus, Total	9	7.7	7.4	8.3	8.1
Temperature, Degrees F	74.4	72.5	75.9	64.1	71.7
TKN	44	41	24	41	38
TKN, Soluble	34	37	26	36	33
Total Suspended Solids	150	300	350	190	248
Volatile Suspended Solids	140	260	340	180	230

SALs

Date / Analyte	Sep 13, 2019 08:00 AM	Sep 14, 2019 08:20 AM	Sep 15, 2019 08:25 AM	Sep 16, 2019 08:30 AM	Average
Alkalinity as $CaCO_3$	520	480	350	540	473
Ammonia as N	40	54	53	63	53
Biochemical Oxygen Demand	160	120	170	240	173
BOD, Carbonaceous	130	130	130	220	153
Chemical Oxygen Demand	1000	270	350	670	573
NO_2+NO_3 as N	ND	150	ND	ND	150
o-Phosphate as P	3.7	3.9	4.3	4.8	4.2
pH	8.3	8.1	8.2	7.9	8.1
Phosphorus, Total	11	13	6.8	11	10.5
Temperature, Degrees F	65.6	66.2	64.5	62.1	64.6
TKN	57	57	59	80	63.3
TKN, Soluble	40	51	49	64	51
Total Suspended Solids	190	130	170	250	185
Volatile Suspended Solids	170	140	170	200	170

Sulfur Mtn

Date / Analyte	Sep 13, 2019 07:40 AM	Sep 14, 2019 07:50 AM	Sep 15, 2019 07:55 AM	Sep 16, 2019 07:55 AM	Average
Alkalinity as $CaCO_3$	500	550	480	460	498
Ammonia as N	31	37	37	29	34
Biochemical Oxygen Demand	150	230	200	190	193
BOD, Carbonaceous	110	190	200	190	173
Chemical Oxygen Demand	520	500	690	370	520
NO_2+NO_3 as N	ND	ND	ND	ND	
o-Phosphate as P	2.2	2.7	4	1.7	2.7
pH	8.2	7.8	8.3	7.9	8.1
Phosphorus, Total	5.5	2.9	9	4.5	5.5
Temperature, Degrees F	62.2	64.4	62.4	66.7	63.9
TKN	45	47	41	34	41.8
TKN, Soluble	31	37	36	32	34
Total Suspended Solids	210	230	180	180	200
Volatile Suspended Solids	200	220	160	180	190

Appendix 1B

PLANT PROCESS CAPACITY CALCULATIONS

Ojai Valley Sanitary District
Facilities Master Plan

Appendix 1B

PLANT PROCESS CAPACITY CALCULATIONS

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Appendix 1B
PLANT PROCESS CAPACITY CALCULATIONS

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Appendix 1B

PLANT PROCESS CAPACITY CALCULATIONS

1B.1 Introduction and Background

Technical Memorandum 1 (TM 1) - Existing Facilities Process Modeling, concluded that the existing wastewater treatment plant's (WWTP's) secondary treatment capacity, assuming current discharge permit requirements and including updated loading parameters, is 2.5 mgd under average annual dry weather flow (ADWF) conditions with both secondary clarifiers in service.

This appendix provides a summary of the process calculations that arrived at the 2.5-mgd process capacity value.

1B.2 Secondary Treatment Capacity Assessment

The method that Carollo Engineers, Inc. (Carollo) uses to determine the capacity of the secondary treatment process is to assess the combined capacity of both the aeration basins and the secondary clarifiers. The first step in determining the capacity of the secondary clarifiers is the development of a solids flux curve.

1B.3 Solids Flux Curve Development

Solids flux is the movement of solids through a clarifier, and it is defined as shown below.

$$Solids\ Flux = mass\ of\ solids\ per\ unit\ clarifier\ area\ per\ unit\ time$$

$$with\ metric\ units\ of\ kg/m^2.hr$$

Table 1B.1 lists key parameters needed for state point analysis.

Table 1B.1 State Point Analysis Key Parameters

Parameter	Symbol
Influent flow rate	Q
Return activated sludge	RAS
Mixed liquor concentration	X_{MLSS}
Sludge settling characteristics	V_0, k
Clarifier surface area	A

A graph with solids flux on the y-axis and solids on the x-axis is created. Then three elements are plotted on the graph, which are:

- Surface overflow rate line (Calculated from the flow into the clarifier, Q and the surface area of the clarifier, A: $Slope = SOR = Q/A$
- Surface underflow rate line (calculated from the return activated sludge (RAS) flowrate and the surface area of the clarifier, A: $Slope = SUR = RAS/A$
- Settling flux curve: $G_S = X \cdot V_S$, where: X = Solids concentration, V_S = Settling velocity at that concentration

For most activated sludge, there exists the following relationship:

$$V_s = V_0 e^{-kX}$$

Substitution for V_S in the settling flux curve, gives:

$$G_s = X \cdot V_0 e^{-kX}$$

Furthermore, V_0 and k can be calculated using the sludge volume index (SVI). The sludge volume index value generated using a 2 liter settleometer without stirring (SVISN) correlation was used to determine V_0 and k. This method is used when the SVI test used is not stirred.

The average SVI value of 130 milliliters per gram (mL/g), based on the plant data, was used to determine V_0, and k:

$$V_0 = 9.21 \text{ m/h}$$

$$k = 0.48 \text{ L/g}$$

The above formulas were then used to create the solids flux curve. Figures 1B.1 and 1B.2 show a typical solids flux curve and the actual curve developed for Ojai Valley Sanitary District (OVSD), respectively. The blue line on Figure 1B.1 shows the shape of a typical settling flux curve and plots solids flux (on the y-axis) against mixed liquor suspended solids (MLSS) (on the x-axis). The area under the blue settling flux curve depicts the area within which the clarifier can operate without failure. The green line shows the surface overflow rate, and the yellow line shows the solids underflow rate. Where these lines cross is the so-called state point and indicates the operating condition of the clarifier under that set of conditions. If the state point is withing the area beneath the blue curve, then the clarifier is within its design capabilities. If the state point is outside the area of the blue curve, then the clarifier is in failure mode. The right-hand end of the yellow line should also fall within the area of the blue curve, otherwise the clarifier will experience solids failure.

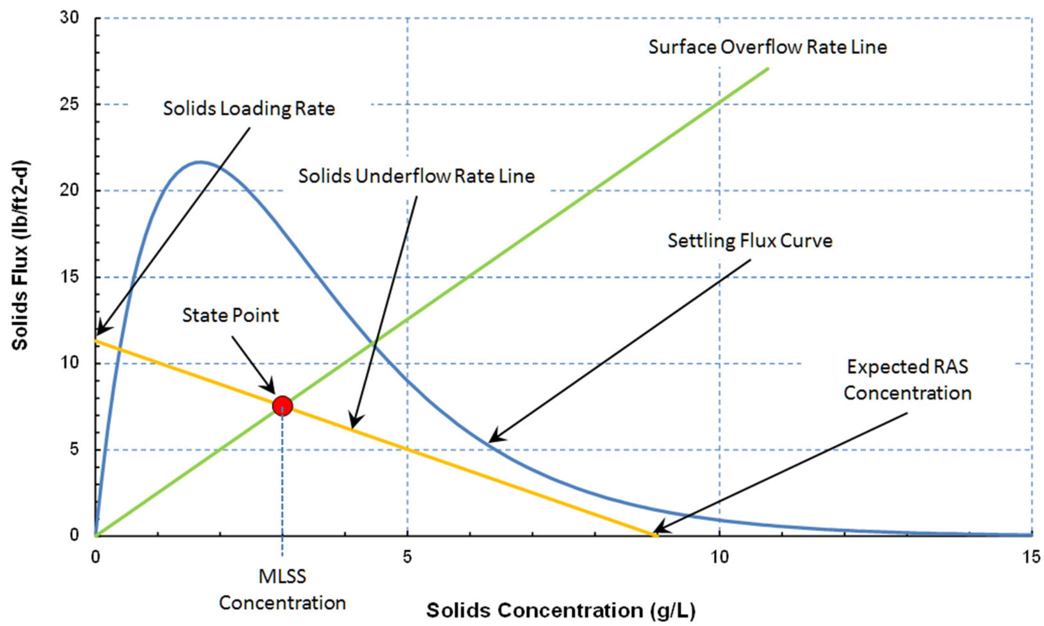

Figure 1B.1 Concept of the Flux Curve Graph

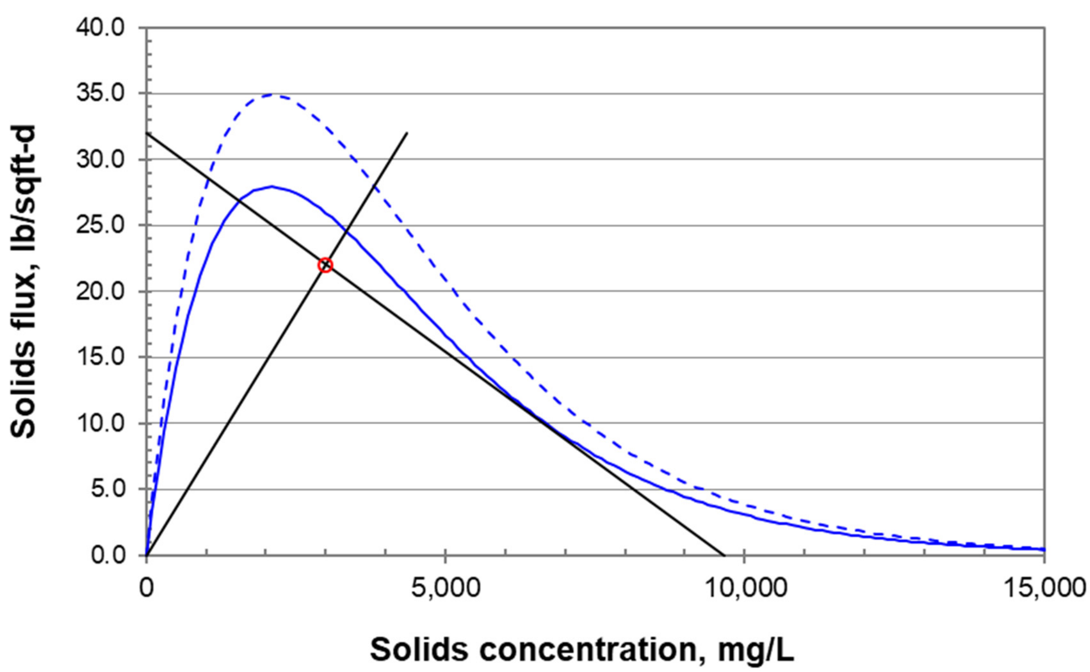

Figure 1B.2 OVSD Settling Flux Curve Graph With an MLSS of 3,000 mg/L and a Q_{ADWF} of 3.33 mgd

Figure 1B.2 shows the state point analysis for the OVSD treatment plant. The state point is within the area of the settling flux curve, which is good, and the solids underflow rate line is touching the settling flux curve, which indicates this is the limit of the solids underflow rate. The conditions shown on Figure 1B.2 are for an MLSS of 3,000 mg/L and a flow rate of 3.33 mgd.

1B.4 State Point Analysis

Since the information presented on Figure 1B.2 only represents one set of conditions, the next step in the state point analysis involves running multiple flow rates and determining the maximum MLSS that each flow rate can handle without failure in a clarifier. Failure, as mentioned, is characterized by:

- Thickening failure (sludge blanket buildup): This is when the solids underflow line intercepts or crosses the flux curve to the right-hand side of the state point.
- Clarification failure (washout): This is when the state point is above the settling flux curve.

Using the state point analysis tool, a table was generated, as shown in Table 1B.2.

Table 1B.2 State Point Analysis Determination of Maximum Q_{ADWF} for Different MLSS Concentrations

MLSS (mg/L)	Q_{ADWF} (mgd)
3,000	3.33
3,100	3.17
3,200	3.03
3,300	2.90
3,400	2.77
3,500	2.63
3,600	2.53
3,700	2.43
3,800	2.30
3,900	2.20
4,000	2.13

1B.5 Process Model Simulation

The next step is to consider the performance of the secondary biological treatment step. This involves determining the expected MLSS in the biological reactor at different influent flow conditions, Q_{ADWF}. To accomplish this task, a calibrated BioWin process model was used. The detailed review of the development of the model is located in TM 1, Existing Facilities Process Modeling. The model is run at each influent flow rate to determine what the mixed liquor concentration would need to be in order to achieve the desired treatment goals.

The calibrated model was simulated at five different flow rates to achieve an MLSS range between 3,600 mg/L and 4,000 mg/L. Table 1B.3 shows the results of this effort.

Table 1B.3 BioWin Simulated MLSS at Different Q_{ADWF}

MLSS (mg/L)	Q_{ADWF} (MGD)
3,600	2.17
3,700	2.52
3,800	2.96
3,900	3.57
3,400	3.91

1B.6 Process Capacity

Values from Tables 1B.2 and 1B.3 were then plotted on the same graph, as shown on Figure 1B.3. Values in Table 1B.2 depict the clarifier capacity, shown in blue, and values in Table 1B.3 depict the biological process reactor capacity, shown in orange. As expected, as the influent flow drops, the clarifiers will be able to handle greater and greater MLSS concentrations. And, for the oxidation ditches, as the MLSS concentration increases, the ditches can treat more flow. For these two systems to operate together, their individual capacities must match, and that occurs at the intersection of the two curves, which indicates the secondary process capacity. As shown on Figure 1B.3, the intercept occurs at 2.5 mgd. At this flow rate, an MLSS of 3,690 mg/L is expected, and, through state point analysis, it was determined that the resulting solids load will be handled by the secondary clarifiers without clarifier failure.

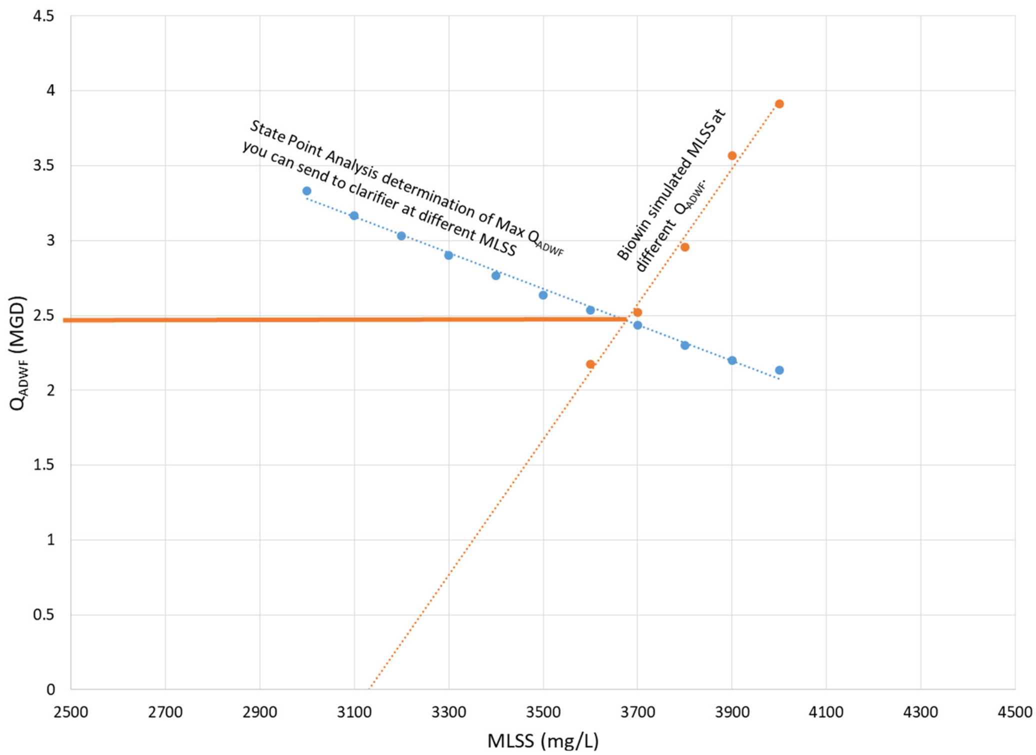

Figure 1B.3 Process Capacity Determination

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 2
EXISTING FACILITIES HYDRAULIC PROFILE

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Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 2
EXISTING FACILITIES HYDRAULIC PROFILE

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Carollo Project No. 11321A00

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EXISTING FACILITIES HYDRAULIC PROFILE

2.1 Introduction

This technical memorandum (TM) is the second in a series of five that will form the basis of the 20-year Facilities Plan for Ojai Valley Sanitary District (OVSD). This TM includes development of a hydraulic model using record drawings of the wastewater treatment plant (WWTP) and an evaluation of the hydraulic model. The evaluation comprises of identifying bottlenecks and deficiencies based on the current flow conditions, and comparing findings with the original design conditions.

2.2 Key Findings and Recommendations

The key findings and recommendations are:

1. To improve the accuracy of the hydraulic model developed in this study, it is recommended that equipment specifications for channel grinders, the rotary drum screen, and tertiary filters be provided. The information will reduce assumptions made on head loss calculations across the unit process and improve the accuracy of the hydraulic model.
2. OVSD's WWTP preliminary and secondary treatment facilities can handle a peak hour flow (PHF) of 9 mgd if the head loss across the rotary drum screen is as reported in the 1996 design data. The tertiary treatment plant can handle an equalized peak flow of 4.3 mgd.
3. Replace the 24-inch pipe, which conveys plant influent from the grit chamber and screen and recycles, with a 33-inch pipe to mitigate any hydraulic limitation at the headworks. This could provide up to 2 feet of additional freeboard at the grit chambers.
4. It is recommended that a hydraulic calibration be implemented at OVSD's WWTP as part of detailed design. A hydraulic calibration can be used to confirm calculated head losses to what is measured in the field. A hydraulic calibration can identify flow split issues and clogging of particular equipment or pathways, which can optimize maintenance and maximize the hydraulic capacity of the WWTP.
5. If only one channel in the Headworks is in operation, the low flow velocity is 0.7 fps, if both channels are in operation, the low flow velocity drops to 0.3 fps. It is recommended that only one channel be placed in operation during low flow conditions.
6. The low flow velocity in the filter influent channel is 0.1 fps. It is recommended that the channel be modified by installing an insert to increase the velocity and reduce the detention time.

2.3 Background

The WWTP is a tertiary plant with a dry-weather design capacity (1996) of 3 million gallons per day (mgd) and an instantaneous peak flow capacity of 9 mgd. Untreated wastewater is collected from the City of Ojai; the unincorporated communities of Meiners Oaks, Mira Monte, Oak View, Casitas Springs, and Foster Park; and North Ventura Avenue area through approximately 120 miles of sanitary sewer lines.

Figure 2.1 shows a flow schematic for the existing plant. Raw wastewater flow enters the headworks through a 30-inch diameter trunk sewer. The headworks facility includes channels with grinders.. Downstream of the grinders, plant influent is directed to four intermediate pumps that lift the flow to a vortex grit removal system steered by a rotary drum screen. The screened influent is then routed to secondary treatment.

At secondary treatment, the influent flows through three anaerobic tanks in series. After the flow leaves the anaerobic tanks, it enters two identical parallel oxidation ditches that are sectioned into anoxic and aerobic zones. Flow from both oxidation ditches is combined in the mixed liquor splitter box and flow via gravity to two 85-foot diameter clarifiers. A portion of the clarifier underflow is sent to dewatering, and the remainder is routed back to the first anaerobic tank as the return activated sludge (RAS) flow. Secondary effluent flows to the filter influent pump station, where up to 4.3 mgd is pumped to the Tertiary Filters and the remaining flow is diverted to Equalization Basins.

At the tertiary facilities, secondary effluent flows through two flocculation basins in series before exiting through a channel. The channel feeds four deep-bed, continuous backwash sand filters before being routed to an ultraviolet (UV) system for disinfection. The UV system consists of one channel with five banks of UV lamps. As a backup, the flow can be routed through a chlorine contact tank downstream of UV.

After disinfection and dechlorination, flow is routed through a 28-inch diameter pipe, to a reaeration structure, and then into a 36-inch diameter pipe to the outfall.

2.4 Basis of Design

The preliminary design criteria and assumptions for this project are described in the following sections.

2.4.1 Datum

The vertical datum for this study was obtained through record drawings provided by OVSD. The hydraulic calculations and all elevations in this TM are based on the elevations as shown in all plant record drawings.

2.4.2 Hydraulic Constraints and Limitations

The existing facilities are designed to meet the hydraulic constraints of the plant, upstream of the equalization and downstream of the equalization basin based on specified hydraulic control points. These water surface elevations (WSEs) are presented in Table 2.1. Additionally, free-discharging weirs identified in 1996 design data are presented in Table 2.2.

Table 2.1 Design Hydraulic Control Points

Hydraulic Constraints	Units	Value
Influent Pump Station Wet Well maximum operating level	ft	181.89
Filter Influent Pump Station Wet Well overflow weir elevation	ft	199.05

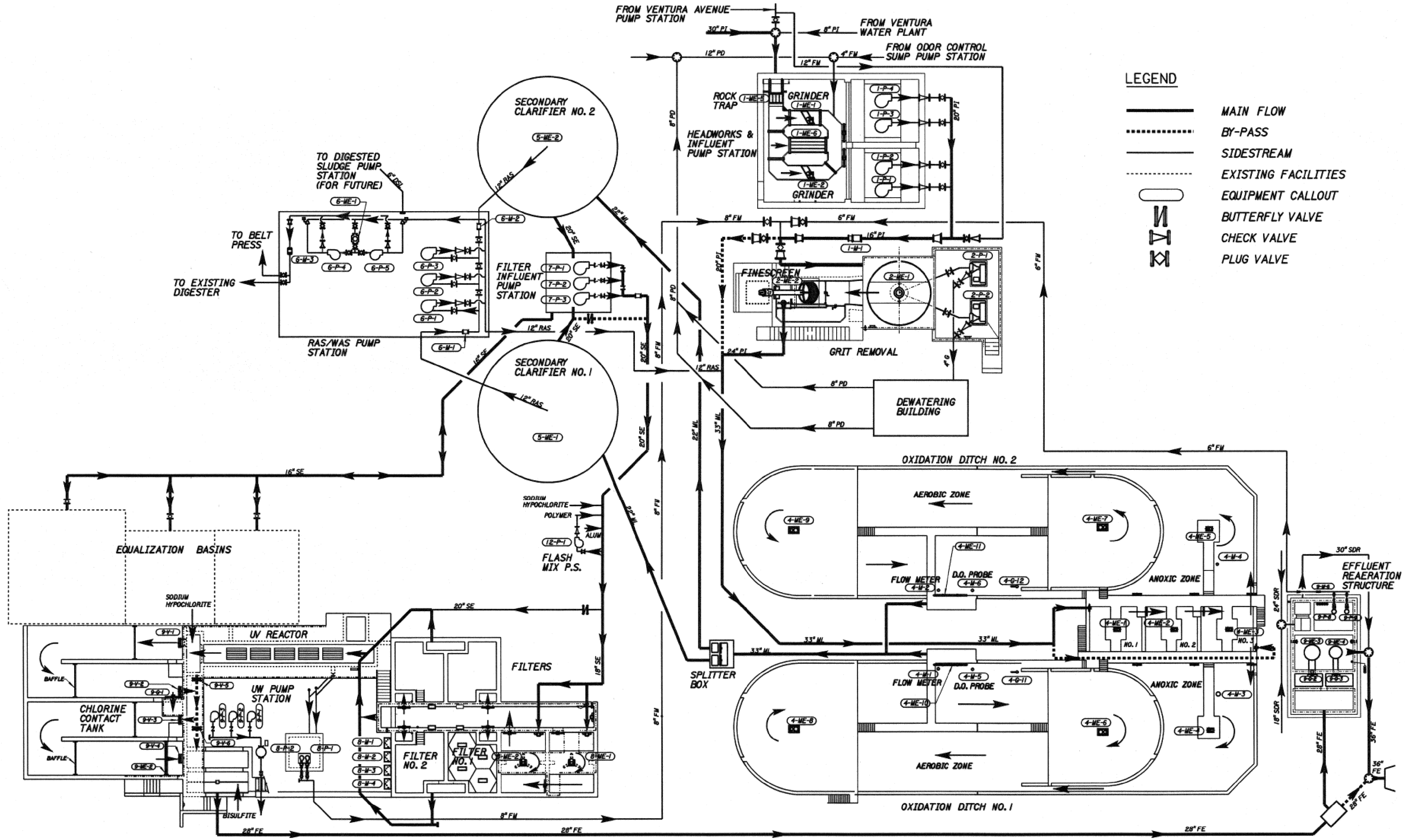

Figure 2.1 Liquid Process Flow Diagram of OVSD's WWTP

Table 2.2 Free-Discharging Weirs

Free-Discharging Weirs	Weir Elevation
Upstream Weirs	
Anoxic Tank	206.71
Oxidation Ditch	206.03
Mixed Liquor Splitter Box	201.97
Secondary Clarifiers	200.42
Filter Influent Pump Station	199.05
Downstream Weirs	
Flocculation Basin	203.21
Filters	200.00
UV Reactor	199.00(Varies to maintain 24” depth in Channel)
Utility Water Pump Station	195.85
Effluent Metering Structure	194.81

It should be noted that confirmation of the operational parameters such as wet well elevations and weir positions be implemented. Most of the weirs at OVSD’s WWTP are downward opening type and can be operated at different positions than what is shown on the 1996 design data.

2.4.3 Design Flows

OVSD’s WWTP design influent flows, which were obtained from the design influent historical flow data between January 2011 and December 2018, were analyzed and presented in TM 1, Existing Facilities Process Modeling. The design influent flows and peaking factors are repeated in Table 2.3.

Table 2.3 Design Flow Peaking Factors

Flows	Peaking Factor(1)	1996 Design Data Flows (mgd)
Annual Average Daily Flow (AADF)	1.0	3
Peak Month Average Flow (PMF)	1.43	4.3
Peak 2 Hour Flow (PTF)	2	6
Instantaneous PHF	3	9
Minimum Flow	0.47(1)	1.4(2)

Notes:
(1) Unless otherwise noted, peaking factors are relative to the design AADF of 3 mgd and are based on 1996 design data obtained from OVSD.
(2) Minimum flows are based on input from operations staff during progress meeting on February 14, 2019

2.4.4 Process Reliability and Design Standards

The assumed process reliability criteria and other design standards for this project are stated in Table 2.4.

Table 2.4 OVSD WWTP Process Reliability Criteria

Process	Units	PTF and PHF (Duty+Standby)	AADF and PMF (Duty+Standby)
Grinders	2	1+1	1+1
Grit Removal	1	1+0	1+0
Drum Screen (Fine Screens)	1	1+0	1+0
Anoxic Tank	3	3+0	3+0
Oxidation Ditch	2	2+0	2+0
Secondary Clarifiers	2	2+0	2+0
Equalization Basins	3	3+0	3+0
Flocculation Basin	2	2+0	2+0
UV Reactor	1	1+0	1+0
Chlorine Contact Tanks	1	0+1	0+1
Design Standards			
Weirs		Free discharge 6 inches preferred; submerged acceptable if needed	Free discharge minimum 6 inches
Flow Splitting		Not necessary, but preferred	Proper flow split
Freeboard		12 inches preferred; no spilling is critical	18+ inches (existing)

2.5 Hydraulic Modeling Evaluation

The hydraulic model was run at design flow conditions, which are listed in Table 2.2. The hydraulic model evaluation is presented in three sections: results from the model, hydraulic limitations, and comparisons of this study's hydraulic model with the 1996 hydraulic model.

2.5.1 Hydraulic Model Results

Figure 2.2 shows the hydraulic model of the existing facilities at OVSD's WWTP. The model makes certain assumptions listed in Table 2.5 based on the information obtained from OVSD.

Table 2.5 Hydraulic Model Assumptions

Assumptions	Remarks
Manual bar rack is used as a backup for the channel grinders.	A single channel grinder has the capacity to handle the instantaneous peak flow.
Head loss across the channel grinder was assumed to be the same as the 1996 record drawings (5 inches).	Product information about the equipment obtained from OVSD.
Head loss across rotary drum screen was assumed to be the same as the 1996 record drawings.	Product information about the equipment obtained from OVSD..
The rotary drum screen will handle up to 6 mgd, and any remaining flow will go through the Bypass Channel.	The 1996 record drawings listed the capacity as 6 mgd.

Assumptions	Remarks
The equalization basin can handle an instantaneous peak flow of 9 mgd and allows facilities downstream to receive a maximum flow of 4.3 mgd.	The tertiary treatment facilities are rated for 4.3 mgd based on the 1996 record drawings.
The flocculation basins are operated in series.	This is based on the process flow diagram in the 1996 record drawings.
Chlorine contact basins are not operated under any of the flow conditions.	This is based on personal communication with OVSD staff.
Head loss across tertiary filters was assumed to be the same as the 1996 record drawings.	Product information about the equipment obtained from OVSD.

The hydraulic model developed in this study showed that OVSD's WWTP is able to handle PHF conditions. This was based on the 1996 record drawings and the assumptions listed in Table 2.5. Model results met the following reliability criteria (Table 2.3):

- The free discharge weirs had less than 6 inches of free discharge or were fully submerged.
- There was less than 12 inches of freeboard.

It should be noted that equipment (specifically, the channel grinders, the rotary drum screen, and tertiary filters) information is needed to improve the accuracy of the model

Additionally, it is recommended that OVSD implement a hydraulic calibration on the WWTP to confirm calculated head losses watch what is measured in the field. A hydraulic calibration can identify flow split issues and clogging of particular equipment or pathways, which can optimize maintenance and maximize the hydraulic capacity of the WWTP.

2.5.2 Hydraulic Limitations

The hydraulic model indicated potential bottlenecks at OVSD's WWTP. This section discusses these limitations and lists potential strategies to mitigate them.

2.5.2.1 Grit Chamber and Screen

The hydraulic model identified that if the assumed head loss across the grit chamber varies by more than 1 ft the grit chamber will have less than 12 inches of freeboard. The 1996 record drawings show that there is a 20-inch bypass pipe that can be used to bypass the grit basins and rotary drum screens in high wet weather flow conditions. According to Carollo's experience, the head loss across the rotary drum screen is influenced by the size of the perforations.

It is recommended that the head loss across the rotary drum screen and the headworks facilities be investigated as part of the New Headworks design project. Additionally, a hydraulic calibration could be implemented to understand the effects of fouling on the head loss across the rotary drum screen.

2.5.2.2 Primary Influent Conveyance

The 24-inch primary influent pipeline highlighted in Figure 2.3 was identified in the model to have the most significant head loss upstream of the equalization basin at PHF conditions. The section of pipe highlighted in Figure 2.3 conveys plant influent and plant recycles, begins at the discharge end of the drum screen, and ends at the tee where RAS is introduced. The pipeline after the tee is 33-inch diameter and has significantly lower head loss with the additional flows.

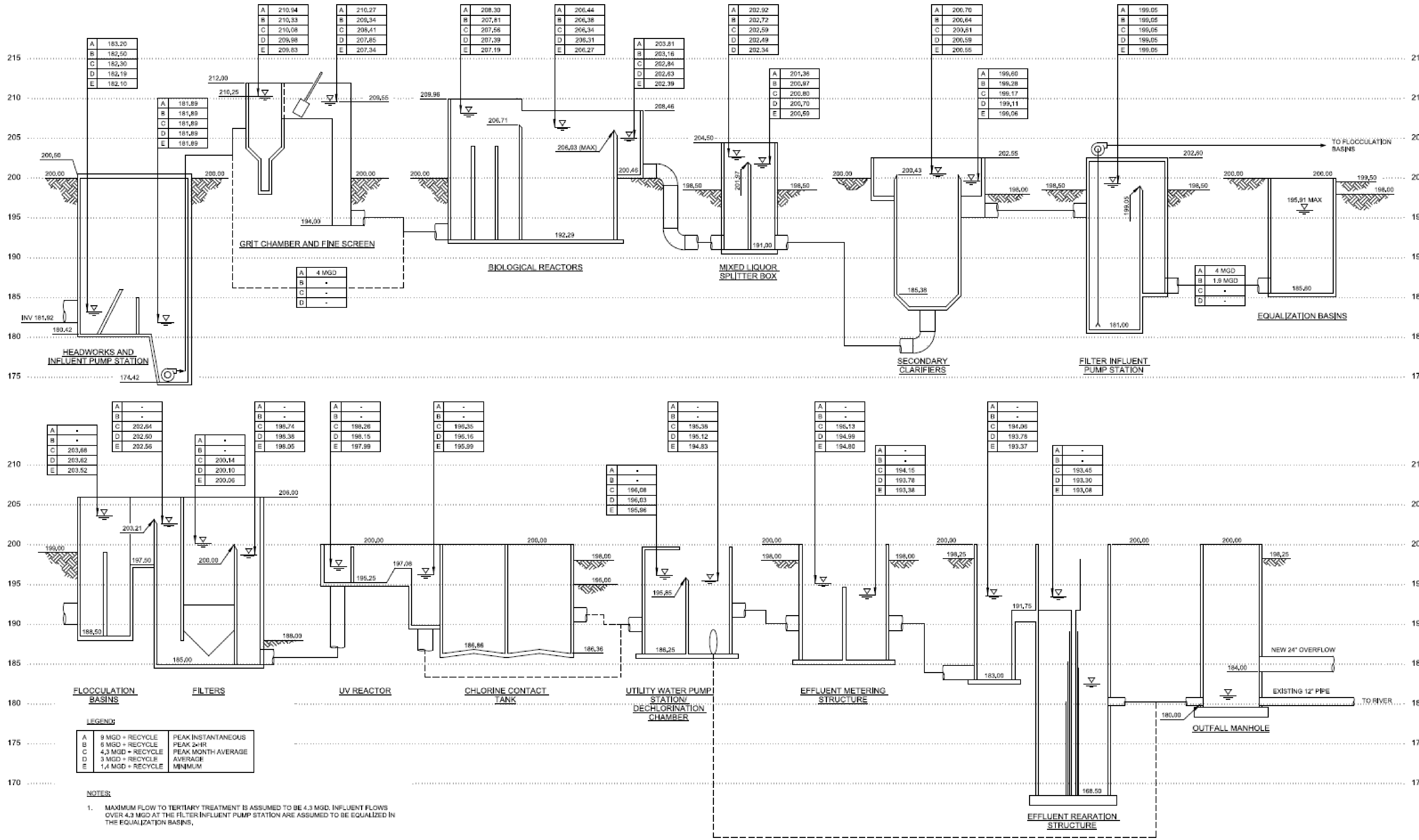

Figure 2.2 Hydraulic Model of OVSD's WWTP

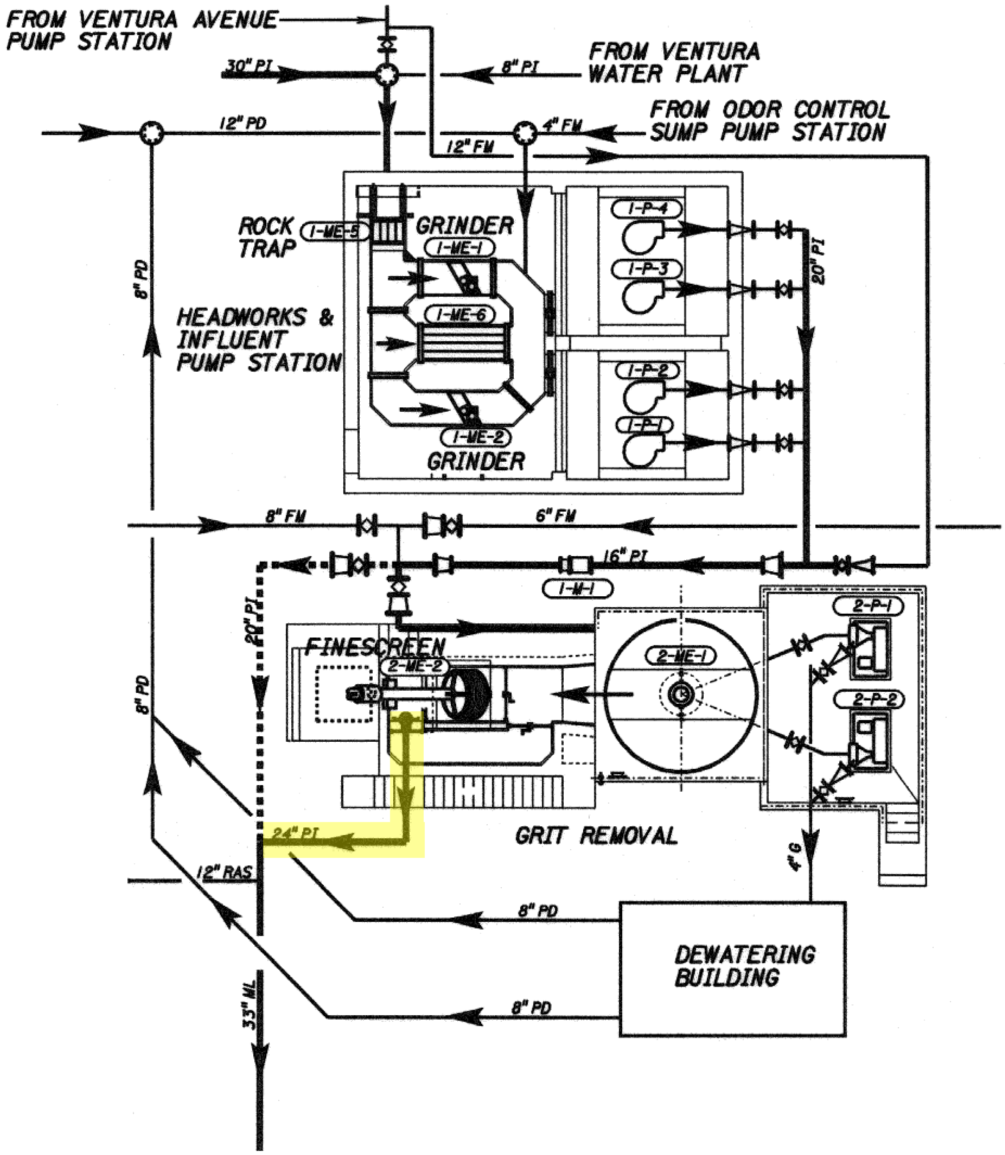

Figure 2.3 Primary Influent Conveyance Limitation

The 24-inch pipeline had a head loss of approximately 1.5 feet at PHF conditions. This was not enough to cause upstream to have less than 12 inches of freeboard with the assumed head loss from the 1996 record drawings. However, if the head loss across the rotary drum screen exceeds 1 ft, there will be less than 12 inches of freeboard at the grit chamber. The 20-inch Bypass pipe can be used to bypass the grit basins and rotary drum screens in case of high level in the grit basins during PHF conditions..

It is recommended that the highlighted 24-inch pipe be replaced with a 33-inch pipe, matching the section of pipe after the tee that conveys plant influent, recycles, and RAS. The hydraulic model identified that the replacement could provide up to 2 feet of freeboard at PHF conditions (see Appendix 2A) and mitigate any potential hydraulic issues at the headworks previously reported. Furthermore, this will minimize the duration when the 20-inch bypass pipe will be active, therefore minimizing unscreened and degritted influent from entering the downstream processes..

2.5.2.3 Rock Traps

Rock traps have been recorded to act as weirs if not routinely cleaned. If the rock traps at the headworks were to become clogged and act as weirs, the hydraulic model showed that there would be a spill at the headworks at PHF conditions. OVSD has commented that the rock traps have been taken out of service due to excessive maintenance.

2.5.3 Comparison to 1996 Hydraulic Model

This section compares this study's hydraulic model to the 1996 hydraulic model. It should be noted that the 1996 hydraulic model had two flow conditions but only labeled one on the hydraulic profile—the 100-year flood condition. It was assumed in this analysis that the other flow condition was PHF (9 mgd) for the upstream of the equalization basin and PMF (4.3 mgd) for downstream of the equalization basin.

2.5.3.1 Upstream of the Equalization Basin

This study's hydraulic model had similar water elevations to the 1996 hydraulic model with the following exceptions:

- The grit chamber bypass channel cannot be used to bypass excess flow under PHF conditions. The 24-inch plant influent identified in Section 2.5.2.2 needs to be replaced with a 33-inch pipe for there to be enough freeboard in the grit chamber to handle PHF conditions.
- The water elevation at the end of the biological reactors before conveyance to the mixed liquor splitter box was approximately 1 foot lower than the 1996 hydraulic model. It is unknown why there is a significant difference. However, a hydraulic calibration might reconcile the disagreement.

2.5.3.2 Downstream of the Equalization Basin

There were no notable differences between this study's hydraulic model and the 1996 hydraulic model.

2.5.4 Minimum Flow Hydraulic Model and Channel Velocities

The hydraulic model was evaluated to verify the impact of low flow conditions on channel and pipe velocities. The hydraulic profile, shown on Figure 2.2, includes water surface elevations for the processes for a minimum flow of 1.4 mgd. Figure 2.4 shows a schematic of the OVSD facilities with flow velocities in all critical pipes and channels. Summarized below are some findings:

1. In general, pipe velocities are in the range of 0.5 to 0.7 fps. These velocities are acceptable.
2. If only one channel in the Headworks is in operation, the low flow velocity is 0.7 fps, if both channels are in operation, the low flow velocity drops to 0.3 fps. It is recommended that only one channel be placed in operation during low flow conditions.
3. The low flow velocity in the Grit Removal channels is 0.4 fps. This velocity seems low, however, the channels are downstream of the grit basins.
4. The low flow velocity in the filter influent channel is 0.1 fps. It is recommended that the channel be modified by installing an insert to increase the velocity and reduce the detention time.

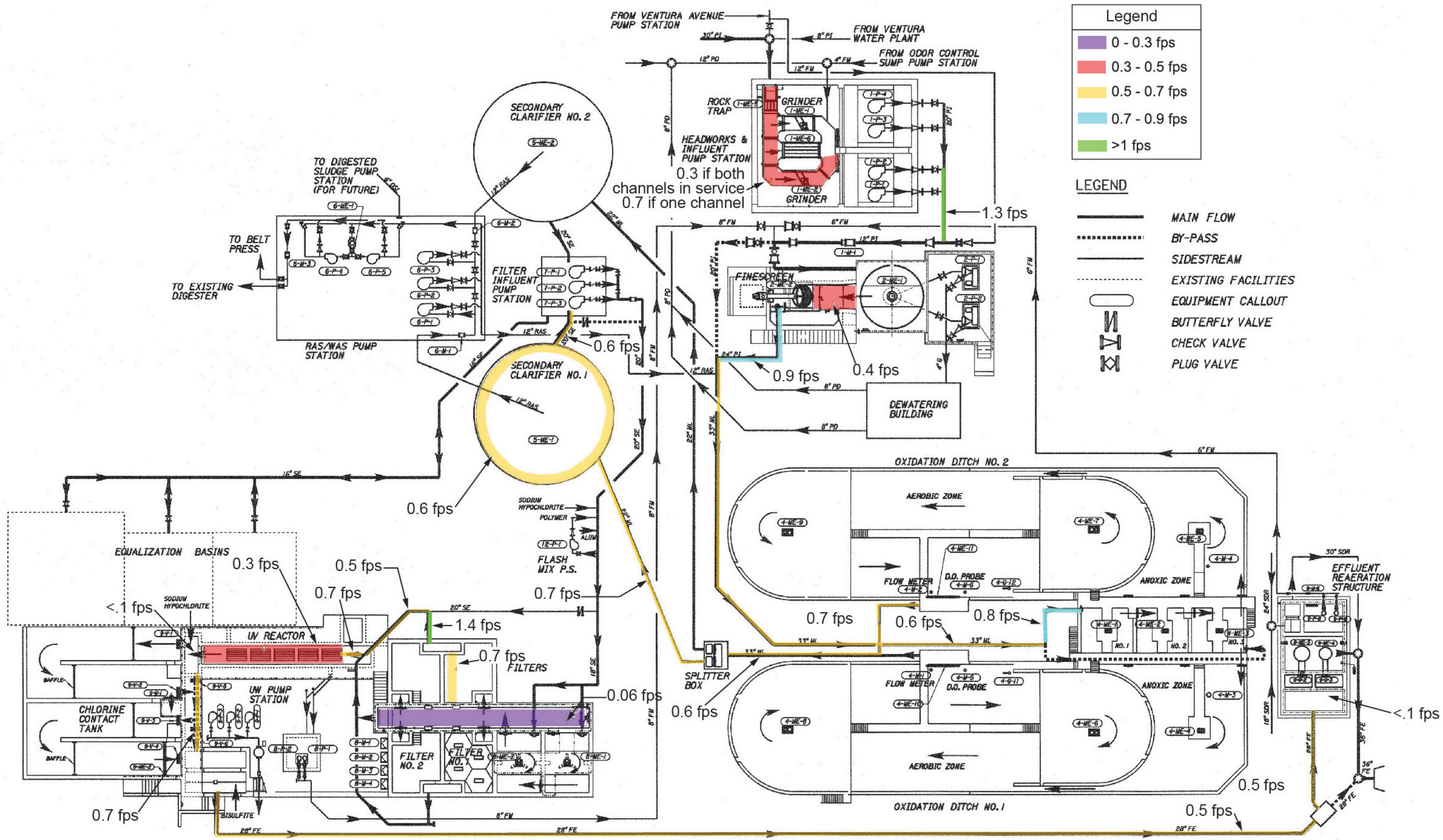

Figure 2.4 Low Flow Channel and Pipe Velocities for Various Process Facilities

2.6 Conclusion and Recommendations

From the analysis presented in this TM, the following conclusions and recommendations can be made:

1. To improve the accuracy of the hydraulic model developed in this study, it is recommended that equipment specifications for channel grinders, the rotary drum screen, and tertiary filters be provided. The information will reduce assumptions made on head loss calculations across the unit process and improve the accuracy of the hydraulic model.
2. OVSD's WWTP preliminary and secondary treatment facilities can handle a peak hour flow (PHF) of 9 mgd if the head loss across the rotary drum screen is as reported in the 1996 design data. The tertiary treatment plant can handle an equalized peak flow of 4.3 mgd.
3. Replace the 24-inch pipe, which conveys plant influent from the grit chamber and screen and recycles, with a 33-inch pipe to mitigate any hydraulic limitation at the headworks. This could provide up to 2 feet of additional freeboard at the grit chambers.
4. It is recommended that a hydraulic calibration be implemented at OVSD's WWTP as part of detailed design. A hydraulic calibration can be used to confirm calculated head losses to what is measured in the field. A hydraulic calibration can identify weir elevations, flow split issues and clogging of particular equipment or pathways, which can optimize maintenance and maximize the hydraulic capacity of the WWTP.
5. If only one channel in the Headworks is in operation, the low flow velocity is 0.7 fps, if both channels are in operation, the low flow velocity drops to 0.3 fps. It is recommended that only one channel be placed in operation during low flow conditions.
6. The low flow velocity in the filter influent channel is 0.1 fps. It is recommended that the channel be modified by installing an insert to increase the velocity and reduce the detention time.

Appendix 2A

HYDRAULIC MODEL CALCULATIONS

Appendix 2A-1

HYDRAULIC MODEL CALCULATIONS
3 MGD

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

Hydraulix® by carollo

JOB # : 101321A00	**REVISION:**	**CHECKED :** TL	**DATE :** 2/6/2019	**BY :** WME	**DATE :** 2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Aeration Basins	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

			HGL	EGL
EGL = 184.00		Assume Free Discharge to Static Aerator	184.00	184.00
Flow = 3.00 mgd = 4.64 cfs				

AREA 9 — BEGIN REARATION STRUCTURE

STATIC AERATOR
[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 3.0 mgd = 4.6 cfs

Downstream HGL < Invert + Crit. Depth: CRITICAL DEPTH USED

Downstream WSE	184.00	ft	Assume Free Discharge
Downstream EGL	184.00	ft	
Channel Width, W	2.0	ft	
Critical. Depth, y_c	0.55	ft	
Channel Invert @ Exit	192.75	ft	Reference 9M-5

Flooded Condition

Depth Upstream of Drop	N/A	ft
Velocity Upstream of Drop	N/A	fps
Channel Exp./Bend "K"	2.00	
Energy Loss	N/A	ft
EGL Upstream of Drop	N/A	ft
HGL Upstream of Drop	N/A	ft

Freefall Condition

Depth Upstream of Drop	0.55
Velocity Upstream of Drop	4.21
EGL Upstream of Drop	193.58
HGL Upstream of Drop	193.30094

	HGL	EGL
Condition Upstream of Drop	193.30	193.58

AERATOR INLET GATE

Downstream Flow	3.0		
Gates Open	2		
Flow per Gate	1.50	mgd =	2.3 cfs
Gate Width	1.5	ft	Reference M-14
Height of Gate	2.0	ft	Reference M-14
Invert Elevation of Gate	192.75		

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C	N/A	
Velocity through gate, v	N/A	fps

Not Submerged Condition

K	1.7		Modeled as gate frame (0.2) and entrance and exit (1.5)
Water Depth thru Gate	0.55	ft	
Velocity through Outlet, v	2.81	fps	

Energy Loss thru Gate, h_L 0.2081 ft

	HGL	EGL
Condition Upstream of Gate	193.66	193.78

EFFLUENT REARATION CHAMBER PART 2

Friction Loss

Flow	1.50	mgd =	2.3 cfs
Channel Width	8.75	ft	
Total Channel Length	4.00	ft	
Downstream Invert El	191.75		Reference 9M-5
Upstream Invert El	191.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.91	ft	
Velocity (Average)	0.14	fps	
Hydraulic Radius (Average)	1.33	ft	

Friction Loss 0.0000 ft

	HGL	EGL
Upstream Condition	193.78	193.78

Hydraulix BY carollo

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

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Equation Ref.	HGL	EGL

EFFLUENT REARATION CHAMBER PART 1

Friction Loss

Downstream Flow	1.5		
Gates Open	2		
Upstream Flow	3.00	mgd =	4.6 cfs

Friction Loss

Flow	1.50	mgd =	2.3 cfs
Channel Width	18.50	ft	
Total Channel Length	6.00	ft	
Downstream Invert El	191.75		Reference 9M-5
Upstream Invert El	191.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	2.03	ft	
Velocity (Average)	0.06	fps	
Hydraulic Radius (Average)	1.67	ft	

Friction Loss 0.0000 ft

	HGL	EGL
Upstream Condition	193.78	193.78

END REARATION STRUCTURE
BEGIN YARD

28-FE FROM EFFLUENT METERING STRUCTURE TO REAERATION STRUCTURE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 3.0 mgd = 4.6 cfs

Pipe Diameter, D	28	inch
Pipe Length, L	60	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.09	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	253247	
Friction factor, f	0.0164	Equivalent Hazen-Williams "C" = 146.8925

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.0 mgd = 4.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	3.00	4.64	0.50	----	28	----	1.09	0.02	0.01
1	Mitre Bend - 22.5 ° Deflection	3.00	4.64	0.15	28	----	1.09	----	0.02	0.00
1	Mitre Bend - 22.5 ° Deflection	3.00	4.64	0.15	28	----	1.09	----	0.02	0.00
1	Outlet Loss - Still Water	3.00	4.64	1.00	28	----	1.09	----	0.02	0.02
									Sum =	0.03

Total Energy Loss = 0.04 ft

	HGL	EGL
Upstream Condition	193.83	193.83

END YARD
AREA 9 BEGIN EFFLUENT METERING STRUCTURE

EFFLUENT METERING STRUCTURE

Friction Loss

Flow	3.00	mgd =	4.6 cfs
Channel Width	5.00	ft	Reference 9S-12
Total Channel Length	4.00	ft	Reference 9S-12
Downstream Invert El	188.75		Reference 9S-12
Upstream Invert El	188.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	5.08	ft	
Velocity (Average)	0.18	fps	
Hydraulic Radius (Average)	1.67	ft	

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)

												Equation Ref.	HGL	EGL
1	90 Degree Bend	3.00	4.64	1.30	6.00	5.00	5.08	0.15	0.18	0.00	0.00			
									Sum Minor Loss=		0.0002 ft			

Total Energy Loss = 0.0002 ft

Upstream Condition — HGL 193.83 — EGL 193.83

EFFLUENT METERING STRUCTURE CONTROL POINT
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 3.0 mgd = 4.6 cfs

WSE Downstream of Weir 193.83 ft
Weir Crest Elevation 194.50 ft Reference 9S-12
Downstream head, Hd -0.67 ft
Length of Weir, L 4.00 ft Reference 9S-12

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }
Head on Weir, H 0.50 ft
Upstream WSE 195.00 ft

Submerged Weir Computation { 7 }
K NA
M NA
Increment NA ft
Upstream Head, Hu1 NA ft
F(H1) NA
F'(H1) NA
Upstream Head, Hu2 NA ft
Upstream WSE NA ft

Head over Weir *0.50 ft*

Condition Upstream of Weir — HGL 195.00 — EGL 195.00

EFFLUENT METERING STRUCTURE

Friction Loss
Flow 3.00 mgd = 4.6 cfs
Channel Width 5.00 ft Reference 9S-12
Total Channel Length 4.00 ft Reference 9S-12
Downstream Invert El 188.75 Reference 9S-12
Upstream Invert El 188.75
Slope 0.00%
Manning Coeff, n 0.015
Depth (Average) 6.25 ft
Velocity (Average) 0.15 fps
Hydraulic Radius (Average) 1.79 ft

Friction Loss 0.0000 ft

Upstream Condition — HGL 194.99 — EGL 195.00

END EFFLUENT METERING STRUCTURE
BEGIN YARD

28-FE PIPE FROM UTILITY PUMP STATION TO EFFLUENT METERING STRUCTURE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 3.0 mgd = 4.6 cfs

Pipe Diameter, D 28 inch
Pipe Length, L 400 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.09 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 253247
Friction factor, f 0.0164 Equivalent Hazen-Williams "C" = 146.8925

Friction Energy Loss, h_L 0.05 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.0 mgd = 4.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)

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												Equation Ref.	HGL	EGL
1	Entrance Loss - Flush	3.00	4.64	0.50	----	28	----	1.09	0.02	0.01				
1	Mitre Bend - 90 º Deflection	3.00	4.64	1.27	28	----	1.09	----	0.02	0.02				
1	Mitre Bend - 45 º Deflection	3.00	4.64	0.32	28	----	1.09	----	0.02	0.01				
1	Mitre Bend - 15 º Deflection	3.00	4.64	0.06	28	----	1.09	----	0.02	0.00				
1	Outlet Loss - Still Water	3.00	4.64	1.00	28	----	1.09	----	0.02	0.02				
									Sum =	0.06				

Total Energy Loss = 0.11 ft

Upstream Condition | 195.10 | 195.10

END YARD

AREA 9 **BEGIN UTILITY PUMP STATION**

UTILITY PUMP STATION

Friction Loss

Flow	3.00 mgd =	4.6 cfs	
Channel Width	18.00 ft	Reference 9S-5	
Total Channel Length	6.00 ft	Reference 9S-5	
Downstream Invert El	186.25	Reference 9S-6	
Upstream Invert El	186.25		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	8.85 ft		
Velocity (Average)	0.03 fps		
Hydraulic Radius (Average)	4.46 ft		

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Turn Around Baffle	3.00	4.64	3.20	4	----	2	0.58	----	0.01	0.02
									Sum Minor Loss=		0.0167 ft

Total Energy Loss = 0.0167 ft

Upstream Condition | 195.12 | 195.12

UTILITY PUMP STATION CONTROL POINT 1
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	3.0 mgd =	4.6 cfs
WSE Downstream of Weir	195.12 ft	
Weir Crest Elevation	195.85 ft	Reference 9S-6
Downstream head, Hd	-0.73 ft	
Length of Weir, L	18.00 ft	Reference 9S-5

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }
Head on Weir, H 0.18 ft
Upstream WSE 196.03 ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.18 ft*

Condition Upstream of Weir | 196.03 | 196.03

UTILITY PUMP STATION

Friction Loss

Flow	3.00 mgd =	4.6 cfs
Channel Width	18.00 ft	Reference 9S-12
Total Channel Length	6.00 ft	Reference 9S-12
Downstream Invert El	186.25	Reference 9S-12
Upstream Invert El	186.25	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	9.78 ft	
Velocity (Average)	0.03 fps	
Hydraulic Radius (Average)	4.69 ft	

	Equation Ref.	HGL	EGL
Friction Loss 0.0000 ft			
Upstream Condition		196.03	196.03

END UTILITY PUMP STATION
BEGIN YARD

BYPASS PIPE FROM UV REACTOR TO UTILITY WATER PUMP STATION
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 3.0 mgd = 4.6 cfs

Pipe Diameter, D	24	inch
Pipe Length, L	12	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.48	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	295455	
Friction factor, f	0.0163	Equivalent Hazen-Williams "C" = 145.6997

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.0 mgd = 4.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	3.00	4.64	0.50	----	24	----	1.48	0.03	0.02
1	Butterfly Valve (Open)	3.00	4.64	0.50	24	----	1.48	----	0.03	0.02
1	Butterfly Valve (Open)	3.00	4.64	0.50	24	----	1.48	----	0.03	0.02
2	Tee - Thru Straight Run	3.00	4.64	0.60	24	----	1.48	----	0.03	0.04
1	Outlet Loss - Still Water	3.00	4.64	1.00	24	----	1.48	----	0.03	0.03
									Sum =	0.13

Total Energy Loss = 0.13 ft

	HGL	EGL
Upstream Condition	196.16	196.16

END YARD
START: UV REACTOR

UV EFFLUENT

Friction Loss

Flow	3.00	mgd = 4.6 cfs	
Channel Width	4.00	ft	Reference 15S-1
Total Channel Length	6.00	ft	Reference 15S-1
Downstream Invert El	190.00		Reference 15S-4
Upstream Invert El	190.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	6.16	ft	
Velocity (Average)	0.19	fps	
Hydraulic Radius (Average)	1.51	ft	

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 Degree Bend - 0° Radius	3.00	4.64	1.30	4.00	----	6.16	0.19	----	0.00	0.00
									Sum Minor Loss=		0.0007 ft

Total Energy Loss = 0.0007 ft

	HGL	EGL
Upstream Condition	196.16	196.16

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 3.0 mgd = 4.6 cfs

Downstream HGL < Invert + Crit. Depth: CRITICAL DEPTH USED

Downstream WSE	196.16	ft	
Downstream EGL	196.16	ft	
Channel Width, W	5.5	ft	Reference 15S-1
Critical. Depth, y_c	0.28	ft	

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			Equation Ref.	HGL	EGL
Channel Invert @ Exit	196.25	ft			
Flooded Condition					
Depth Upstream of Drop	N/A	ft			
Velocity Upstream of Drop	N/A	fps			
Channel Exp./Bend "K"	2.00	(Note: Modify K value as appropriate)			
Energy Loss	N/A	ft			
EGL Upstream of Drop	N/A	ft			
HGL Upstream of Drop	N/A	ft			
Freefall Condition					
Depth Upstream of Drop	0.28				
Velocity Upstream of Drop	3.01				
EGL Upstream of Drop	196.67				
HGL Upstream of Drop	196.53068				
Condition Upstream of Drop				196.53	196.67

UV REACTOR

Friction Loss

Flow	3.00	mgd =	4.6 cfs		
Channel Width	5.50	ft	Reference 15S-1		
Total Channel Length	70.00	ft	Reference 15S-1		
Downstream Invert El	195.25		Reference 15S-4		
Upstream Invert El	195.25				
Slope	0.00%				
Manning Coeff, n	0.015				
Depth (Average)	1.28	ft			
Velocity (Average)	0.66	fps			
Hydraulic Radius (Average)	0.87	ft			
Friction Loss	0.0037	ft			
Upstream Condition				196.67	196.67

UV REACTOR CONTROL POINT 1
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	3.0	mgd =	4.6 cfs		
WSE Downstream of Weir	196.16	ft			
Weir Crest Elevation	197.75	ft	Reference 15S-4		
Downstream head, Hd	-1.59	ft			
Length of Weir, L	5.50	ft			
WEIR IS FREE-DISCHARGING					
Free Discharging Weir Computation			{ 6 }		
Head on Weir, H	0.40	ft			
Upstream WSE	198.15	ft			
Submerged Weir Computation			{ 7 }		
K	NA				
M	NA				
Increment	NA	ft			
Upstream Head, Hu1	NA	ft			
F(H1)	NA				
F'(H1)	NA				
Upstream Head, Hu2	NA	ft			
Upstream WSE	NA	ft			
Head over Weir	*0.40*	*ft*			
Condition Upstream of Weir				198.15	198.15

END: **UV REACTOR**
START: **YARD**

24-FE FROM FILTERS EFFLUENT BOX TO UV REACTOR
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	3.0	mgd =	4.6 cfs
Pipe Diameter, D	24	inch	
Pipe Length, L	15	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	1.48	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	295455		
Friction factor, f	0.0163		Equivalent Hazen-Williams "C" = 145.6997
Friction Energy Loss, h_L	0.00	ft	

MINOR PIPE LOSS HEADING

Equation Ref.	HGL	EGL

Flow, Q 3.0 mgd = 4.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee - Thru Side Outlet	3.00	4.64	1.80	24	----	1.48	----	0.03	0.06
1	Outlet Loss - Still Water	3.00	4.64	1.00	24	----	1.48	----	0.03	0.03
									Sum =	0.09

Total Energy Loss = 0.10 ft

		HGL	EGL
Upstream Condition		198.25	198.2495

FILTERS 1&2 AND FILTERS 3&4 FLOWS JOIN TO UV REACTOR
FLOW SPLIT

Downstream Flow 3.0 mgd = 4.6 cfs
Filter Halves in Service 2
New Flow 1.5 mgd = 2.3 cfs Assume even flow split

20- FROM FILTERS 3&4 EFFLUENT BOX TO UV REACTOR
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 1.5 mgd = 2.3 cfs

Pipe Diameter, D 20 inch
Pipe Length, L 20 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.06 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 177273
Friction factor, f 0.0176 Equivalent Hazen-Williams "C" = 145.6845

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 1.5 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Entrance Loss - Flush	1.50	2.32	0.50	----	20	----	1.06	0.02	0.00
2	90 ° Elbow - Regular Fl.	1.50	2.32	0.30	20	----	1.06	----	0.02	0.01
0	Increaser	1.50	2.32	0.25	20	24	1.06	0.74	0.01	0.00
0	Tee - Thru Side Outlet	1.50	2.32	1.80	20	----	1.06	----	0.02	0.00
1	Mitre Bend - 45 ° Deflection	1.50	2.32	0.32	24	----	0.74	----	0.01	0.00
									Sum =	0.01

Total Energy Loss = 0.02 ft

		HGL	EGL
Upstream Condition		198.27	198.27

PIPE FROM FILTERS 3&4 EFFLUENT BOX TO UV REACTOR
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 1.5 mgd = 2.3 cfs

Pipe Diameter, D 12 inch
Pipe Length, L 6 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 2.95 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 295455
Friction factor, f 0.0176 Equivalent Hazen-Williams "C" = 140.2964

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 1.5 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)	Equation Ref.	HGL	EGL
1	Entrance Loss - Flush	1.50	2.32	0.50	----	12	----	2.95	0.14	0.07			
1	Tee - Thru Side Outlet	1.50	2.32	1.80	20	----	1.06	----	0.02	0.03			
									Sum =	0.10			
	Total Energy Loss =	0.11 ft											
											Upstream Condition	198.38	198.38

END: **YARD**
START: **FLOCCULATION BASIN**

FILTER EFFLUENT CHIMNEY

Friction Loss

Flow	1.50 mgd =	2.3 cfs	
Channel Width	2.50 ft	Reference 8S-1	
Total Channel Length	11.50 ft	Reference 8S-1	
Downstream Invert El	185.00	Reference 8S-3	
Upstream Invert El	185.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	13.38 ft		
Velocity (Average)	0.07 fps		
Hydraulic Radius (Average)	1.14 ft		
Friction Loss	0.0000 ft		

Upstream Condition — HGL 198.38 — EGL 198.38

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Downsteram Flow	1.50	
Filters per Launder	2	
Flow per Filter	0.75	
Flow, Q	0.8 mgd =	1.2 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	198.38 ft	
Downstream EGL	198.38 ft	
Channel Width, W	1.5 ft	Reference 8S-3
Critical. Depth, y_c	0.26 ft	
Channel Invert @ Exit	197.50 ft	Reference 8S-3

Flooded Condition

Depth Upstream of Drop	0.89 ft	
Velocity Upstream of Drop	0.87 fps	
Channel Exp./Bend "K"	2.00	(Note: Modify K value as appropriate)
Energy Loss	0.02 ft	
EGL Upstream of Drop	198.40 ft	
HGL Upstream of Drop	198.39 ft	

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

Condition Upstream of Drop — HGL 198.39 — EGL 198.40

FILTER EFFLUENT

Friction Loss

Flow	0.75 mgd =	1.2 cfs
Channel Width	1.50 ft	Reference 8S-1
Total Channel Length	10.60 ft	Reference 8S-3
Downstream Invert El	197.50	Reference 8S-3
Upstream Invert El	197.50	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	0.89 ft	
Velocity (Average)	0.87 fps	
Hydraulic Radius (Average)	0.41 ft	
Friction Loss	0.0027 ft	

Upstream Condition — HGL 198.39 — EGL 198.41

FILTER EFFLUENT LAUNDER
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 0.8 mgd = 1.2 cfs

				Equation Ref.	HGL	EGL
WSE Downstream of Weir	198.39	ft				
Weir Crest Elevation	200.00	ft	Reference 8S-3			
Downstream head, Hd	-1.61	ft				
Length of Weir, L	10.60	ft	Reference 8S-3			

WEIR IS FREE-DISCHARGING

				Equation Ref.
Free Discharging Weir Computation				{ 6 }
Head on Weir, H	0.10	ft		
Upstream WSE	200.10	ft		
Submerged Weir Computation				{ 7 }
K	NA			
M	NA			
Increment	NA	ft		
Upstream Head, Hu1	NA	ft		
F(H1)	NA			
F'(H1)	NA			
Upstream Head, Hu2	NA	ft		
Upstream WSE	NA	ft		
Head over Weir	*0.10*	*ft*		

Condition Upstream of Weir — HGL 200.10, EGL 200.10

FILTERS

Flow per Filter	0.75	mgd	
Filter Headloss	30.00	in	Assumed

Upstream Condition — HGL 202.60, EGL 202.60

FILTERS

Friction Loss

Flow	0.75	mgd = 1.2 cfs	
Channel Width	14.00	ft	Reference 8S-3
Total Channel Length	14.00	ft	Reference 8S-3
Downstream Invert El	185.00		Reference 8S-3
Upstream Invert El	185.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	17.60	ft	
Velocity (Average)	0.00	fps	
Hydraulic Radius (Average)	5.01	ft	
Friction Loss	0.0000	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Turn Around Baffle	0.75	1.16	3.20	2.50	----	2.5	0.19	----	0.00	0.00
									Sum Minor Loss=		0.0017 ft

Total Energy Loss = 0.0017 ft

Upstream Condition — HGL 202.60, EGL 202.60

FILTER INFLUENT WEIR (downward opening weir gate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.8	mgd = 1.2 cfs	
WSE Downstream of Weir	202.60	ft	
Weir Crest Elevation	203.21	ft	Assumed EL per G-3
Downstream head, Hd	-0.61	ft	Gate is 36x42, low position 201.35, high position 204.85 (not full closed)
Length of Weir, L	3.00	ft	

WEIR IS FREE-DISCHARGING

				Equation Ref.
Free Discharging Weir Computation				{ 6 }
Head on Weir, H	0.24	ft		
Upstream WSE	203.45	ft		
Submerged Weir Computation				{ 7 }
K	NA			
M	NA			
Increment	NA	ft		
Upstream Head, Hu1	NA	ft		
F(H1)	NA			
F'(H1)	NA			
Upstream Head, Hu2	NA	ft		

			Equation Ref.	HGL	EGL
Upstream WSE	NA	ft			
Head over Weir	*0.24*	*ft*			
		Condition Upstream of Weir		203.45	203.45

FLOW SPLIT

Downstream Flow	0.8	mgd =	1.2	cfs
Filters in Service	4			
New Flow	3.0	mgd =	4.6	cfs

Friction Loss

Flow	3.00	mgd =	4.6 cfs
Channel Width	6.00	ft	Reference 8S-3
Total Channel Length	60.00	ft	Reference 8S-2
Downstream Invert El	197.50		Reference 8S-3
Upstream Invert El	197.50		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	5.95	ft	
Velocity (Average)	0.13	fps	
Hydraulic Radius (Average)	1.99	ft	

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 Degree Bend - 0º Radius	3.00	4.64	1.30	6.00	----	6.00	0.13	----	0.00	0.00
									Sum Minor Loss=		0.0003 ft

Total Energy Loss = 0.0004 ft

	HGL	EGL
Upstream Condition	203.45	203.45

FLOCCULATION BASIN 1&2 DISCHARGE TO CHANNEL

FLOW SPLIT

Downstream Flow	3.0	mgd =	4.6	cfs
Flocculation Basins	2			
New Flow	1.5	mgd =	2.3	cfs

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	1.50	mgd =	2.3 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	3.5	ft	Reference M-14
Invert Elevation of Gate	200.50		Reference 8S-5
			Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C	N/A	
Velocity through gate, v	N/A	fps

Not Submerged Condition

K	1.7		Modeled as gate frame (0.2), entrance (0.5), exit(1.0)
Water Depth thru Gate	2.95	ft	
Velocity through Outlet, v	0.26	fps	

Energy Loss thru Gate, h_L 0.0018 ft

	HGL	EGL
Condition Upstream of Gate	203.45	203.45

FLOCCULATION BASIN

Friction Loss

Flow	1.50	mgd =	2.3 cfs
Channel Width	6.00	ft	Reference 8S-3
Total Channel Length	60.00	ft	Reference 8S-2
Downstream Invert El	197.50		Reference 8S-3
Upstream Invert El	197.50		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	5.95	ft	
Velocity (Average)	0.07	fps	
Hydraulic Radius (Average)	1.99	ft	

Friction Loss 0.0000 ft

Minor Loss

Flow	Flow	Width Up	Width Down	Depth	Vel Up	Vel Down	Vel Head	Minor Loss

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00	**REVISION:**	**CHECKED :** TL	**DATE :** 2/6/2019	**BY :** WME	**DATE :** 2/6/2017

No.	Description	(mgd)	(cfs)	K	(ft)	(ft)	(ft)	(fps)	(fps)	(ft)	(ft)	Equation Ref.	HGL	EGL
1	Turn Around Baffle	1.50	2.32	3.20	5	----	5	0.09	----	0.00	0.00			
									Sum Minor Loss=		0.0004 ft			

Minor Loss

Flocculation Basin 2.00 in

Total Energy Loss = 0.1671 ft

Upstream Condition	203.62	203.62

END: FLOCCULATION BASIN

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00	REVISION:	CHECKED : TL	BY : WME
		DATE : 2/6/2019	DATE : 2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Bioreactors	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

EGL = 199.05

Flow = 3.00 mgd = 4.64 cfs

HGL 199.05 | EGL 199.05

AREA 7 — FILTER INFLUENT PUMP STATION

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	4.30	mgd
Flow to EQ Basin	-1.30	mgd
Influent Flow	3.00	mgd
Filter Backwash	0.17	mgd
Upstream Flow	3.17	mgd

HWL	199.05	ASSUMED USE HWL AT ALL FLOWS	REFERENCE 7M-1
LWL	185.00		

PS Wetwell Elevation	*Upstream Condition*	199.05	199.05

FILTER INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	3.17 mgd =	4.9 cfs		
Channel Width	8.25 ft		Reference 7M-1	Assumed, need S dwgs
Total Channel Length	18.00 ft		Reference 7M-2	Assumed, need S dwgs
Downstream Invert El	176.00		Reference 7M-3	Assumed, need S dwgs
Upstream Invert El	176.00			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	23.05 ft			
Velocity (Average)	0.03 fps			
Hydraulic Radius (Average)	3.50 ft			

Friction Loss 0.0000 ft

Other Loss

Turbulence 0.00 in Assumed

Total Energy Loss = 0.0000 ft

Upstream Condition	199.05	199.05

FILTER INFLUENT PUMP STATION
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 3.2 mgd = 4.9 cfs

WSE Downstream of Weir	199.05 ft	
Weir Crest Elevation	191.25 ft	Assumed per Reference 7M-1
Downstream head, Hd	7.80 ft	
Length of Weir, L	18.00 ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation { 7 }

K	0.00	
M	21.78	
Increment	0.10	ft
Upstream Head, Hu1	7.80	ft
F(H1)	0.00	
F'(H1)	-0.19	
Upstream Head, Hu2	7.80	ft

	Equation Ref.	HGL	EGL
Upstream WSE 199.05 ft			
Head over Weir 7.80 ft			
Condition Upstream of Weir		199.05	199.05

CLARIFIER 1&2 EFFLUENT JUNCTION
FLOW SPLIT

Downstream Flow	3.2	mgd =	4.9	cfs
No. of clarifiers	2.0			
New Flow	1.6	mgd =	2.5	cfs

20-SE SECONDARY CLARIFIER DISCHARGE PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	1.6	mgd = 2.5 cfs
Pipe Diameter, D	20	inch
Pipe Length, L	15	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.12	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	187437	
Friction factor, f	0.0175	Equivalent Hazen-Williams "C" = 145.607
Friction Energy Loss, h_L	0.00	ft

MINOR PIPE LOSS HEADING

Flow, Q 1.6 mgd = 2.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	1.59	2.45	1.00	----	20	----	1.12	0.02	0.02
1	Mitre Bend - 45 º Deflection	1.59	2.45	0.32	20	----	1.12	----	0.02	0.01
1	Tee - Thru Straight Run	1.59	2.45	0.60	20	----	1.12	----	0.02	0.01
1	Outlet Loss - Still Water	1.59	2.45	1.00	20	----	1.12	----	0.02	0.02
									Sum =	0.06

Total Energy Loss = 0.06 ft

	HGL	EGL
Upstream Condition	199.11	199.11

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 1.6 mgd = 2.5 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	199.11	ft	
Downstream EGL	199.11	ft	
Channel Width, W	2.0	ft	Reference 5S-2
Critical. Depth, y_c	0.36	ft	
Channel Invert @ Exit	197.55	ft	Reference 5S-2

Flooded Condition

Depth Upstream of Drop	1.57	ft
Velocity Upstream of Drop	0.78	fps
Channel Exp./Bend "K"	2.00	
Energy Loss	0.02	ft
EGL Upstream of Drop	199.13	ft
HGL Upstream of Drop	199.12	ft

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

	HGL	EGL
Condition Upstream of Drop	199.12	199.13

SECONDARY CLARIFIER EFFLUENT LAUNDER

Clarifier Diameter	85	feet	Reference 5S-2
Weir Diameter	80	feet	

					Equation Ref.	HGL	EGL
Flow							
Downstream Flow	1.6	mgd =	2.5				
Split launder	2.0						
New Flow	0.8	mgd =	1.2				
Friction Loss							
Flow	1.59	mgd =	2.5 cfs				
Channel Width	2.00	ft		Reference 5S-2			
Total Channel Length	130.38	ft		Reference 5S-2			
Downstream Invert El	197.55			Reference 5S-2			
Upstream Invert El	198.37						
Slope	0.63%						
Manning Coeff, n	0.015						
Depth (Average)	1.16	ft					
Velocity (Average)	1.06	fps					
Hydraulic Radius (Average)	0.54	ft					
Friction Loss	0.0341	ft					
Upstream Condition						199.15	199.16

SECONDARY CLARIFIER EFFLUENT WEIR
[V-NOTCH WEIR]

					Equation Ref.	HGL	EGL
Flow	1.59	mgd =	2.5 cfs				
WSE Downstream of Weir	199.15	ft					
Weir Crest Elevation	200.43	ft		Reference 5S-4			
Downstream head, Hd	-1.28	ft					
Weir Length	251.33	ft					
Distance Between Notches	6.00	in		Reference 5S-4			
Number of Notches	502						
WEIR IS FREE-DISCHARGING							
Free Discharging Weir Computation					{ 8 }		
Head on Weir, H	0.08	ft					
Upstream WSE	200.51	ft					
Submerged Weir Computation					{ 9 }		
K	NA						
M	NA						
Increment	NA	ft					
Upstream Head, Hu1	NA	ft					
F(H1)	NA						
F'(H1)	NA						
Upstream Head, Hu2	NA	ft					
Upstream WSE	NA	ft					
Head over Weir	*0.08*	*ft*					
Condition Upstream of Weir						200.51	200.51

SECONDARY CLARIFIER INFLUENT BAFFLE
[V-NOTCH WEIR]

					Equation Ref.	HGL	EGL
Downstream Flow	1.59						
RAS	0.26	mgd =					
Dewatering Recycles	0.14						
Upstream Flow	2.0	mgd =	3.1 cfs				
WSE Downstream of Weir	200.51	ft					
Weir Crest Elevation	200.43	ft					
Downstream head, Hd	0.08	ft					
Weir Length	78.54	ft		Assumed			
Distance Between Notches	6.00	in		Assumed			
Number of Notches	157						
WEIR IS SUBMERGED							
Free Discharging Weir Computation					{ 8 }		
Head on Weir, H	NA	ft					
Upstream WSE	NA	ft					
Submerged Weir Computation					{ 9 }		
K	0.00						
M	0.00						
Increment	0.10	ft					
Upstream Head, Hu1	0.15	ft					
F(H1)	0.00						
F'(H1)	-37.41						
Upstream Head, Hu2	0.15	ft					
Upstream WSE	200.58	ft					

	Equation Ref.	HGL	EGL
Head over Weir *0.15 ft*			
Condition Upstream of Weir		200.58	200.58

SECONDARY CLARIFIER FLOCCULATING WELL

- Treat as a submerged orifice.

Flow, Q	1.99	mgd =	3.1 cfs
Downstream WSE	200.51	ft	
Flocculation Diameter	25.00	ft	
EDI Diameter	8.50	ft	
Opening Area	434	sf	
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.01	fps	
Energy Loss , h_L	0.00	ft	

Condition in Flocculating Well — HGL 200.58 — EGL 200.58

SECONDARY CLARIFIER ENERGY DISSIPATION INLET

- Treat as a submerged orifice
- Assume Upstream EGL = HGL

Flow, Q	1.99	mgd =	3.1 cfs
Downstream WSE	200.58	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.00	sf	
Total Port Area	16	sf	
Submerged Port Area	189423%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.19	fps	
Energy Loss , h_L	0.00	ft	

Condition in Influent Well — HGL 200.58 — EGL 200.58

SECONDARY CLARIFIER CENTER COLUMN OUTLETS

- Treat as a submerged orifice

Orifice Loss

Flow, Q	1.99	mgd =	3.1 cfs
Downstream WSE	200.58	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.0	sf	
Total Port Area	16	sf	
Submerged Port Area	189426%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.19	fps	
Energy Loss , h_L	0.00	ft	

Minor Losses

Flow, Q 1.99 mgd = 3.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mounding Loss	1.99	3.08	0.25	22	----	1.17	----	0.02	0.01
									Sum =	0.01

Total Energy Loss = 0.01 ft

HGL 200.59 — EGL 200.59

22"ML SECONDARY CLARIFIER CENTER COLUMN
PIPE FRICTION LOSSES (MANNING) - Full Pipe Flow Only

Flow	1.99	mgd =	3.1 cfs **Flow + Total Recycle + RAS**
Pipe Diameter, D	22	inch	
Pipe Length, L	22	ft	
Manning Coef., n	0.015	ft	

		Equation Ref.	HGL	EGL

Velocity 1.17 fps
Hydraulic Radius 0.46 ft

Friction Energy Loss 0.01 ft

MINOR PIPE LOSS HEADING

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Bend	1.99	3.08	0.60	22	----	1.17	----	0.02	0.01
						Total Minor Losses =				0.01 ft

Total Energy Loss = 0.02 ft

		HGL	EGL
Clarifier center column	*Upstream Condition*	200.59	200.61

22"ML JUNCTION BOX TO CLARIFIER 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 2.0 mgd = 3.1 cfs

Pipe Diameter, D 22 inch
Pipe Length, L 220 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.17 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 213910
Friction factor, f 0.0171 Equivalent Hazen-Williams "C" = 145.9125

Friction Energy Loss, h_L 0.04 ft

MINOR PIPE LOSS HEADING

Flow, Q 2.0 mgd = 3.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mitre Bend - 90 º Deflection	1.99	3.08	1.27	22	----	1.17	----	0.02	0.03
1	Mitre Bend - 45 º Deflection	1.99	3.08	0.32	22	----	1.17	----	0.02	0.01
1	Entrance Loss - Flush	1.99	3.08	0.50	----	22	----	1.17	0.02	0.01
									Sum =	0.044

Total Energy Loss = 0.09 ft

	HGL	EGL
Upstream Condition	200.70	200.70

AREA 4 **MIXED LIQUOR SPLITTER BOX**

ML JUNCTION FLOW SPLIT
FLOW SPLIT

Downstream Flow 2.0 mgd = 3.1 cfs
No. of SCs Oline 2.0
New Flow 4.0 mgd = 6.2 cfs

MIXED LIQUOR SPLITTER EFFLUENT

Friction Loss

Flow	3.98	mgd =	6.2 cfs
Channel Width	6.00	ft	Reference 4S-1
Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	9.70	ft	
Velocity (Average)	0.11	fps	
Hydraulic Radius (Average)	2.29	ft	

Friction Loss 0.0000 ft

Other Loss

				Equation Ref.	HGL	EGL
Turbulence	0.00	in	Assumed			
Total Energy Loss =	0.0000 ft					
			Upstream Condition		200.70	200.70

ML SPLITTER BOX WEIR (downward opening weir gates)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	4.0	mgd =	6.2 cfs			
WSE Downstream of Weir	200.70	ft				
Weir Crest Elevation	201.97	ft	Assume EL per Reference G-3 60"x24" weir gate.			
Downstream head, Hd	-1.27	ft	Low position 201.75, top of STR opening is 203.25, so can full close			
Length of Weir, L	5.00	ft				

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation				{ 6 }		
Head on Weir, H	0.52	ft				
Upstream WSE	202.49	ft				
Submerged Weir Computation				{ 7 }		
K	NA					
M	NA					
Increment	NA	ft				
Upstream Head, Hu1	NA	ft				
F(H1)	NA					
F'(H1)	NA					
Upstream Head, Hu2	NA	ft				
Upstream WSE	NA	ft				
Head over Weir	*0.52*	*ft*				
			Condition Upstream of Weir		202.49	202.49

MIXED LIQUOR SPLITTER INFLUENT

Friction Loss

Flow	3.98	mgd =	6.2 cfs			
Channel Width	15.00	ft	Reference 4S-1			
Total Channel Length	4.00	ft	Reference 4S-1			
Downstream Invert El	191.00		Reference 4S-4			
Upstream Invert El	191.00					
Slope	0.00%					
Manning Coeff, n	0.015					
Depth (Average)	11.49	ft				
Velocity (Average)	0.04	fps				
Hydraulic Radius (Average)	4.54	ft				
Friction Loss	0.0000	ft				

Other Loss

Turbulence	0.00	in	Assumed			
Total Energy Loss =	0.0000	ft				
			Upstream Condition		202.49	202.49

33"ML FROM OX DITCH TEE TO ML SPLITTER BOX
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	4.0	mgd =	6.2 cfs			
Pipe Diameter, D	33	inch				
Pipe Length, L	95	ft				
Absolute Roughness, ε	0.00040	ft				
Pipe velocity, v	1.04	fps				
Kinematic Viscosity	1.000E-05	ft^2/sec				
Reynold's Number, R	285213					
Friction factor, f	0.0159		Equivalent Hazen-Williams "C" = 147.5033			
Friction Energy Loss, h_L	0.01	ft				

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 4.0 mgd = 6.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	3.98	6.16	0.50	----	33	----	1.04	0.02	0.01
1	Tee - Thru Side	3.98	6.16	1.80	33	----	1.04	----	0.02	0.03
									Sum =	0.04

Total Energy Loss = 0.05 ft

Upstream Condition HGL 202.53 EGL 202.53

OX DITCH TEE
FLOW SPLIT

Downstream Flow	4.0 mgd =	6.2	cfs
Ox Ditch online	2		
New Flow	2.0 mgd =	3.1	cfs

22"ML FROM OX DITCH 1 TEE TO OX DITCH TEE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 2.0 mgd = 3.1 cfs

Pipe Diameter, D	22	inch
Pipe Length, L	50	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.17	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	213910	
Friction factor, f	0.0171	

Equivalent Hazen-Williams "C" = 145.9125

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 2.0 mgd = 3.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Reducer	1.99	3.08	0.25	33	22	0.52	1.17	0.02	0.01
3	Mitre Bend - 90 º Deflection	1.99	3.08	1.27	22	----	1.17	----	0.02	0.08
									Sum =	0.09

Total Energy Loss = 0.10 ft

Upstream Condition HGL 202.63 EGL 202.63

AREA 4 **BIOLOGICAL REACTORS**

OXIDATION DITCH EFFLUENT WEIR (motorized weir)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 1.99 mgd = 3.1 cfs

WSE Downstream of Weir	202.63	ft	
Weir Crest Elevation	206.03	ft	low position: 204.96; high position: 206.03 — Reference 4M-5
Downstream head, Hd	-3.40	ft	
Length of Weir, L	15.00	ft	Reference M-13

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }
Head on Weir, H 0.16 ft
Upstream WSE 206.19 ft

Submerged Weir Computation { 7 }
K NA

				Equation Ref.	HGL	EGL
M	NA					
Increment	NA	ft				
Upstream Head, Hu1	NA	ft				
F(H1)	NA					
F'(H1)	NA					
Upstream Head, Hu2	NA	ft				
Upstream WSE	NA	ft				
Head over Weir	*0.16*	*ft*				
				Condition Upstream of Weir	206.19	206.19

AEROBIC ZONE

Friction Loss

Flow	1.99	mgd =	3.1 cfs	
Channel Width	30.25	ft	Reference 4S-1	
Total Channel Length	256.00	ft	Reference 4S-1	
Downstream Invert El	192.29		Reference 4S-4	
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	13.90	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.24	ft		
Friction Loss	0.0000	ft		

Other Loss

Baffles	1.00	in	Assumed

Total Energy Loss = 0.0833 ft

	HGL	EGL
Upstream Condition	206.27	206.27

TRANSITION FROM ANOXIC TO AEROBIC

Friction Loss

Flow	1.99	mgd =	3.1 cfs
Channel Width	2.50	ft	Reference 4s-4
Total Channel Length	30.00	ft	
Downstream Invert El	192.29		
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	13.98	ft	
Velocity (Average)	0.09	fps	
Hydraulic Radius (Average)	1.15	ft	
Friction Loss	0.0000	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	1.99	3.08	1.00	5.00	8.00	13.98	0.04	0.03	0.00	0.00
1	Sudden Contraction	1.99	3.08	0.50	8.00	5.00	13.98	0.03	0.04	0.00	0.00
										Sum Minor Loss=	0.00 ft

Total Energy Loss = 0.0000 ft

	HGL	EGL
Upstream Condition	206.27	206.27

ANOXIC ZONE

Friction Loss

Flow	1.99	mgd =	3.1 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	13.98	ft	
Velocity (Average)	0.01	fps	
Hydraulic Radius (Average)	7.18	ft	
Friction Loss	0.0000	ft	

Other Loss

Vertical Mixer	0.50	in	Assumed

	Equation Ref.	HGL	EGL
Total Energy Loss = 0.0417 ft			
Upstream Condition		206.31	206.31

ANAEROBIC REACTOR EFFLUENT WEIR (downward opening weir gate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	2.0 mgd =	3.1 cfs	
WSE Downstream of Weir	206.31 ft		
Weir Crest Elevation	206.71 ft	INV opening: 204.96. Gate height 30-inches.	Reference 4S-7
Downstream head, Hd	-0.40 ft	Therefore: low position 204.96, high position 207.46	Reference M-14
Length of Weir, L	3.00 ft		

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation		{ 6 }
Head on Weir, H	0.46 ft	
Upstream WSE	207.17 ft	
Submerged Weir Computation		{ 7 }
K	NA	
M	NA	
Increment	NA ft	
Upstream Head, Hu1	NA ft	
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA ft	
Upstream WSE	NA ft	
Head over Weir	*0.46 ft*	

	HGL	EGL
Condition Upstream of Weir	207.17	207.17

ANAEROBIC ZONE 3

Friction Loss

Downstream Flow	2.0
Ox Ditches in Service	2
Upstream Flow	3.9820023

Friction Loss

Flow	3.98 mgd =	6.2 cfs
Channel Width	29.50 ft	Reference 4S-1
Total Channel Length	25.50 ft	Reference 4S-1
Downstream Invert El	192.29	Reference 4S-7
Upstream Invert El	192.29	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	14.88 ft	
Velocity (Average)	0.01 fps	
Hydraulic Radius (Average)	7.41 ft	
Friction Loss	0.0000 ft	

Other Loss

Vertical Mixer	0.50 in	Assumed
Total Energy Loss =	0.0417 ft	

	HGL	EGL
Upstream Condition	207.21	207.21

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 2 and 3)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	4.0 mgd =	6.2 cfs	
		INV opening: 204.96. Gate height 42-inches.	Reference M-14
WSE Downstream of Weir	207.21 ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close	
Weir Crest Elevation	204.96 ft	ASSUMED	
Downstream head, Hd	2.25 ft		
Length of Weir, L	3.00 ft	Reference 4S-7	

WEIR IS SUBMERGED

Free Discharging Weir Computation		{ 6 }
Head on Weir, H	NA ft	
Upstream WSE	NA ft	

				Equation Ref.	HGL	EGL
Submerged Weir Computation				{ 7 }		
K	0.28					
M	3.37					
Increment	0.10	ft				
Upstream Head, Hu1	2.27	ft				
F(H1)	0.00					
F'(H1)	-0.67					
Upstream Head, Hu2	2.27	ft				
Upstream WSE	207.23	ft				
Head over Weir	*2.27*	*ft*				
			Condition Upstream of Weir		207.23	207.23

ANAEROBIC ZONE 2

Friction Loss

Flow	3.98	mgd =	6.2 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	14.94	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.42	ft		
Friction Loss	0.0000	ft		

Other Loss

Vertical Mixer	0.50	in	Assumed
Total Energy Loss =	0.0417	ft	

	HGL	EGL
Upstream Condition	207.27	207.27

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 1 and 2)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	4.0	mgd =	6.2 cfs
			INV opening: 204.96. Gate height 42-inches. Reference M-14
WSE Downstream of Weir	207.27	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close
Weir Crest Elevation	204.96	ft	ASSUMED
Downstream head, Hd	2.31	ft	
Length of Weir, L	3.00	ft	Reference 4S-7

WEIR IS SUBMERGED

				Equation Ref.	HGL	EGL
Free Discharging Weir Computation				{ 6 }		
Head on Weir, H	NA	ft				
Upstream WSE	NA	ft				
Submerged Weir Computation				{ 7 }		
K	0.28					
M	3.51					
Increment	0.10	ft				
Upstream Head, Hu1	2.32	ft				
F(H1)	0.00					
F'(H1)	-0.66					
Upstream Head, Hu2	2.32	ft				
Upstream WSE	207.28	ft				
Head over Weir	*2.32*	*ft*				
			Condition Upstream of Weir		207.28	207.28

ANAEROBIC ZONE 1

Friction Loss

Flow	3.98	mgd =	6.2 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	14.99	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.44	ft		
Friction Loss	0.0000	ft		

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00 REVISION:

CHECKED : TL BY : WME
DATE : 2/6/2019 DATE : 2/6/2017

	Equation Ref.	HGL	EGL

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition 207.33 207.33

ANAEROBIC REACTOR INFLUENT GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 4.0 mgd = 6.2 cfs

Diameter of Opening 2.5 ft
Sluice Gate Percent Open 100%
Discharge Coefficient, C 0.61
Velocity through gate, v 1.25 fps

Energy Loss thru Gate, h_L 0.07 ft

Condition Upstream of Gate 207.39 207.39

FLOW CHANGE/SPLIT

Downstream Flow 3.98 mgd = 6.2 cfs
Anaerobic Bypass 0.00
Upstream Flow 3.98 mgd =

30" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 3.98 mgd = 6.2 cfs

Pipe Diameter, D 30 inch
Pipe Length, L 13 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.25 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 313734
Friction factor, f 0.0159 Equivalent Hazen-Williams "C" = 146.845

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.0 mgd = 6.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Increaser	3.98	6.16	0.25	30	33	1.25	1.04	0.01	0.00
1	Outlet Loss - Still Water	3.98	6.16	1.00	33	----	1.04	----	0.02	0.02
									Sum =	0.02

Total Energy Loss = 0.02 ft

Upstream Condition 207.41 207.41

33" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 3.98 mgd = 6.2 cfs

Pipe Diameter, D 33 inch
Pipe Length, L 280 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.04 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 285213
Friction factor, f 0.0159 Equivalent Hazen-Williams "C" = 147.5033

Friction Energy Loss, h_L 0.03 ft

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 3.98 mgd = 6.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	Mitre Bend - 90 ° Deflection	3.98	6.16	1.27	33	----	1.04	----	0.02	0.04
1	Tee - standard	3.98	6.16	1.50	33	----	1.04	----	0.02	0.03
2	Mitre Bend - 45 ° Deflection	3.98	6.16	0.32	33	----	1.04	----	0.02	0.01
1	Tee - Thru Straight	3.98	6.16	0.60	33	----	1.04	----	0.02	0.01
									Sum =	0.09

Total Energy Loss = 0.12 ft

Upstream Condition HGL 207.53 EGL 207.53

24" PI (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Downstream Flow	3.98	mgd
RAS split	0.52	mgd
Upstream Flow	3.46	mgd = 5.4 cfs
Pipe Diameter, D	24	inch
Pipe Length, L	120	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.70	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	340759	
Friction factor, f	0.0160	Equivalent Hazen-Williams "C" = 145.2685

Friction Energy Loss, h_L 0.04 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.46 mgd = 5.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	3.46	5.35	0.50	----	24	----	1.70	0.05	0.02
1	Mitre Bend - 90 ° Deflection	3.46	5.35	1.27	24	----	1.70	----	0.05	0.06
1	Tee - Standard	3.46	5.35	1.50	24	----	1.70	----	0.05	0.07
1	Increaser	3.46	5.35	0.25	24	33	1.70	0.90	0.03	0.01
									Sum =	0.16

Total Energy Loss = 0.20 ft

Upstream Condition HGL 207.73 EGL 207.73

AREA 2 **GRIT REMOVAL AND FINE SCREENS**

24-PI INLET GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 3.5 mgd = 5.4 cfs

Diameter of Opening	2	ft
Sluice Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	1.70	fps

Energy Loss thru Gate, h_L 0.12 ft

Condition Upstream of Gate HGL 207.85 EGL 207.85

FINE SCREEN EFFLUENT SUMP { 4 }

Friction Loss

				Equation Ref.	HGL	EGL
Flow	3.46 mgd =	5.4 cfs				
	=	2404.7 gpm				
Channel Width	3 ft		Reference	#21A		
Channel Height	5 ft		Reference	#21B		
Equivalent Pipe Diameter, D	52 inch					
Conduit Length, L	13.66 ft					
Roughness Coefficient, C	120					
Pipe velocity, v	0.36 fps					
Total Friction Loss	0.0002 ft					

MINOR PIPE LOSS HEADING

Flow, Q 3.5 mgd = 5.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Outlet Loss - Still Water	3.46	5.35	1.00	52	----	0.36	----	0.00	0.00
1	Reducer	3.46	5.35	0.25	52	52	0.36	0.36	0.00	0.00
									Sum =	0.00247

Total Energy Loss = 0.00 ft

	HGL	EGL
Upstream Condition	207.85	207.85

FLOW CHANGE/SPLIT

Downstream Flow	3.46 mgd =	5.4 cfs
Fine Screen Bypass	3.00	
Upstream Flow	0.46 mgd =	

FINE SCREEN EFFLUENT WEIR (redwood weir plate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.5 mgd =	0.7 cfs
WSE Downstream of Weir	207.85 ft	
Weir Crest Elevation	208.17 ft	
Downstream head, Hd	-0.32 ft	
Length of Weir, L	5.00 ft	

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation			{ 6 }
Head on Weir, H	0.12 ft	11.52	
Upstream WSE	208.29 ft		

Submerged Weir Computation		{ 7 }
K	NA	
M	NA	
Increment	NA ft	
Upstream Head, Hu1	NA ft	
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA ft	
Upstream WSE	NA ft	

Head over Weir *0.12 ft*

	HGL	EGL
Condition Upstream of Weir	208.29	208.29

FINE SCREEN

Flow	0.5 mgd =	0.7 cfs
Blinding	50%	
Maximum Headloss	0.50 ft	

	HGL	EGL
Condition Upstream of Bar Screen	208.79	208.79

FINE SCREEN INFLUENT

Friction Loss

Downstream Flow	0.5
Fine Screen Bypass	3
Upstream Flow	3.5

Friction Loss

Flow	3.46 mgd =	5.4 cfs
Channel Width	5.00 ft	Reference 2S-1
Total Channel Length	25.50 ft	

	Equation Ref.	HGL	EGL

Downstream Invert El	207.67	
Upstream Invert El	207.67	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	1.12	ft
Velocity (Average)	0.95	fps
Hydraulic Radius (Average)	0.77	ft

Friction Loss 0.0033 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee- Thru Straight	3.46	5.35	0.60	5.00	----	1.12	0.95	----	0.01	0.01
										Sum Minor Loss=	0.01 ft

Total Energy Loss = 0.0118 ft

	HGL	EGL
Upstream Condition	208.79	208.80

GRIT CHAMBER EFFLUENT WEIR
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 3.5 mgd = 5.4 cfs

WSE Downstream of Weir	208.79	ft	
Weir Crest Elevation	209.37	ft	Reference 2M-2
Downstream head, Hd	-0.58	ft	
Length of Weir, L	5.50	ft	Reference 2S-1

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation {6}

Head on Weir, H	0.44	ft
Upstream WSE	209.81	ft

Submerged Weir Computation {7}

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.44 ft*

	HGL	EGL
Condition Upstream of Weir	209.81	209.81

GRIT CHAMBER

Flow 3.46 mgd = 5.4 cfs

Maximum Headloss 2.00 in Assumed

	HGL	EGL
Condition Upstream of Bar Screen	209.98	209.98

PIPE FROM HEADWORKS TO GRIT CHAMBER
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] {4}

Flow 3.5 mgd = 5.4 cfs

Pipe Diameter, D	20	inch
Pipe Length, L	80	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.45	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	408910	
Friction factor, f	0.0161	

Equivalent Hazen-Williams "C" = 143.1881

Friction Energy Loss, h_L 0.07 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.5 mgd = 5.4 cfs

													Equation Ref.	HGL	EGL

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	0.87	1.34	1.00	----	12	----	1.70	0.05	0.05
1	Entrance Loss - Pipe Ext.	0.87	1.34	1.00	----	12	----	1.70	0.05	0.05
1	Entrance Loss - Pipe Ext.	0.87	1.34	1.00	----	12	----	1.70	0.05	0.05
1	Entrance Loss - Pipe Ext.	0.87	1.34	1.00	----	12	----	1.70	0.05	0.05
1	Tee - Thru Side Outlet	3.46	5.35	1.80	20	----	2.45	----	0.09	0.17
1	Reducer	3.46	5.35	0.25	20	16	2.45	3.83	0.23	0.06
1	Mag Meter	3.46	5.35	0	16	----	3.83	----	0.23	0.00
1	Increaser	3.46	5.35	0.25	16	16	3.83	3.83	0.00	0.00
1	Plug Valve (Open)	3.46	5.35	0.77	16	----	3.83	----	0.23	0.18
1	Increaser	3.46	5.35	0.25	16	20	3.83	2.45	0.13	0.03
1	Mitre Bend - 90 º Deflection	3.46	5.35	1.27	20	----	2.45	----	0.09	0.12
1	Mitre Bend - 90 º Deflection	3.46	5.35	1.27	20	----	2.45	----	0.09	0.12
1	Mitre Bend - 90 º Deflection	3.46	5.35	1.27	20	----	2.45	----	0.09	0.12
1	Mitre Bend - 90 º Deflection	3.46	5.35	1.27	20	----	2.45	----	0.09	0.12
1	Mitre Bend - 90 º Deflection	3.46	5.35	1.27	20	----	2.45	----	0.09	0.12
1	Outlet Loss - Still Water	3.46	5.35	1.00	20	----	2.45	----	0.09	0.09
									Sum =	1.30

Total Energy Loss = 1.37 ft

		HGL	EGL
Influent Wet Well	*Upstream Condition*	211.18	211.18210

AREA 1 **HEADWORKS AND INFLUENT PUMP STATION**

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	3.46	mgd
Wetwells in service	2	
Flow Upstream	1.73	mgd
HHWL	182.92	
HWL	181.89	
LWL	180.42	
LLWL	178.42	

ASSUMED USE HWL AT ALL FLOWS

REFERENCE 1M-3

		HGL	EGL
PS Wetwell Elevation	*Upstream Condition*	181.89	181.89

INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	1.73	mgd = 2.7 cfs
Channel Width	13.83	ft
Total Channel Length	15.00	ft
Downstream Invert El	174.42	
Upstream Invert El	174.42	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	7.47	ft
Velocity (Average)	0.03	fps
Hydraulic Radius (Average)	3.59	ft

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Reducer	1.73	2.68	0.25	6.00	13.83	7.47	0.06	0.03	0.00	0.00
									Sum Minor Loss=		0.0000 ft

Total Energy Loss = 0.0000 ft

	HGL	EGL
Upstream Condition	181.89	181.89

IPS WETWELL INLET GATE

Flow per Gate	1.73	mgd = 2.7 cfs
Gate Width	3.5	ft
Height of Gate	5.0	ft
Invert Elevation of Gate	180.42	

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C N/A

			Equation Ref.	HGL	EGL

Velocity through gate, v | N/A | fps

Not Submerged Condition

K	1.7		Modeled as gate frame (0.2) and entrance and exit (1.5)
Water Depth thru Gate	1.47	ft	
Velocity through Outlet, v	0.52	fps	

Energy Loss thru Gate, h_L 0.0071 ft

	HGL	EGL
Condition Upstream of Gate	181.89	181.90

HEADWORKS EFFLUENT CHANNEL

Flow Downstream	1.73	mgd
Wetwells in service	2	
Flow Upstream	3.46	mgd

Friction Loss

Flow	1.73	mgd =	2.7 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	4.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.47	ft	
Velocity (Average)	0.61	fps	
Hydraulic Radius (Average)	0.74	ft	

Friction Loss 0.0002 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Side	1.73	2.68	1.80	6.00	3.00	1.47	0.30	0.61	0.00	0.01
									Sum Minor Loss=		0.0077 ft

Total Energy Loss = 0.0079 ft

	HGL	EGL
Upstream Condition	181.90	181.91

CHANNEL GRINDER EFFLUENT GATE

Flow per Gate	3.46	mgd =	5.4 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C	N/A	
Velocity through gate, v	N/A	fps

Not Submerged Condition

K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.48	ft	
Velocity through Outlet, v	1.21	fps	

Energy Loss thru Gate, h_L 0.0045 ft

	HGL	EGL
Condition Upstream of Gate	181.89	181.91

CHANNEL GRINDER

Friction Loss

Flow	3.46	mgd =	5.4 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	16.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.47	ft	
Velocity (Average)	1.22	fps	
Hydraulic Radius (Average)	0.74	ft	

Friction Loss 0.0036 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
3	45 degree bend	3.46	5.35	0.25	6.00	3.46	1.47	0.61	1.05	0.01	0.01
									Sum Minor Loss=		0.0086 ft

	Equation Ref.	HGL	EGL

Other

Grinder 2.00 in Assumed

Total Energy Loss = 0.1789 ft

Upstream Condition		182.07	182.09

CHANNEL GRINDER INFLUENT GATE

Flow per Gate 3.46 mgd = 5.4 cfs

Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C	N/A	
Velocity through gate, v	N/A	fps

Not Submerged Condition

K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.65	ft	
Velocity through Outlet, v	1.08	fps	

Energy Loss thru Gate, h_L 0.0037 ft

Condition Upstream of Gate		182.07	182.09

HEADWORKS INFLUENT CHANNEL

Friction Loss

Flow	3.46	mgd =	5.4 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	6.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.65	ft	
Velocity (Average)	1.08	fps	
Hydraulic Radius (Average)	0.79	ft	

Friction Loss 0.0010 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	3.46	5.35	0.60	6.00	3.00	1.65	0.54	1.08	0.01	0.01
									Sum Minor Loss=		0.0081 ft

Total Energy Loss = 0.0091 ft

Upstream Condition		182.08	182.10

CHANNEL GRINDER INFLUENT GATE

Flow per Gate 3.46 mgd = 5.4 cfs

Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	3.46		

Gate is Submerged

Submerged Condition

Discharge Coefficient, C	0.61	
Velocity through gate, v	0.36	fps

Not Submerged Condition

K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	178.62	ft	
Velocity through Outlet, v	N/A	fps	

Energy Loss thru Gate, h_L 0.0053 ft

Condition Upstream of Gate		182.10	182.11

HEADWORKS INFLUENT CHANNEL

Flow Downstream	3.46	mgd
Hdwrks Channels in service	1	
Flow Upstream	3.46	mgd

Friction Loss

				Equation Ref.	HGL	EGL
Flow	3.46	mgd =	5.4 cfs			
Channel Width	3.00	ft	Reference 1S-1			
Total Channel Length	6.00	ft	Reference 1S-1			
Downstream Invert El	180.42		Reference 1S-2			
Upstream Invert El	180.42					
Slope	0.00%					
Manning Coeff, n	0.015					
Depth (Average)	1.68	ft				
Velocity (Average)	1.06	fps				
Hydraulic Radius (Average)	0.79	ft				
Friction Loss	0.0009	ft				

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	3.46	5.35	0.60	6.00	3.00	1.68	0.53	1.06	0.01	0.01
										Sum Minor Loss=	0.0078 ft

Total Energy Loss = 0.0088 ft

	HGL	EGL
Upstream Condition	182.10	182.12

ROCK TRAP { 1 }

Total Flow	3.46	mgd		
Number of online screens	1			
Flow per Screen	3.46	mgd =	5.4	cfs
Channel Flow	3.46	mgd	5.4	cfs
Channel Width	3.00	ft		
Channel & bar rack clearance	0.25	ft		Assumed
Bar Rack Width	2.5	ft		
DS Water Surface Elev	182.10	ft		
Bar Screen Invert Elevation	180.42	ft		
Downstream Water Depth	1.68	ft		
Installation Angle	60	deg		Assumed
Sine Angle	0.8660			
Bar Spacing	1.000	in		Assumed
Bar Thickness	0.313	in		Assumed
Bar Rack Efficiency	0.76			
Bar Rack Open Area	3.6904	sf		
V, velocity Clean Bar Rack	1.45	fps		
v, approach velocity	1.27	fps		
Headloss, clean	0.01	ft		
Upstream Water Depth	1.69			
Blockage	40%			
V, velocity Blocked Bar Rack	2.42	fps		
v, approach velocity	1.21	fps		
Headloss, blocked	0.10	ft	1.17	inches
Upstream Water Depth	1.78			

	HGL	EGL
W - Condition just Upstream of Bar Screen No 1	182.19	182.21

Appendix 2A-2

HYDRAULIC MODEL CALCULATIONS 4.3 MGD

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00   REVISION:

CHECKED : TL   DATE : 2/6/2019   BY : WME   DATE : 2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Aeration Basins	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

EGL = 184.00

Flow = 4.30 mgd = 6.65 cfs

Assume Free Discharge to Static Aerator — HGL 184.00, EGL 184.00

AREA 9   BEGIN REARATION STRUCTURE

STATIC AERATOR
[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 4.3 mgd = 6.7 cfs

Downstream HGL < Invert + Crit. Depth: CRITICAL DEPTH USED

Downstream WSE	184.00	ft	Assume Free Discharge
Downstream EGL	184.00	ft	
Channel Width, W	2.0	ft	
Critical. Depth, y_c	0.70	ft	
Channel Invert @ Exit	192.75	ft	Reference 9M-5

Flooded Condition

Depth Upstream of Drop	N/A	ft
Velocity Upstream of Drop	N/A	fps
Channel Exp./Bend "K"	2.00	
Energy Loss	N/A	ft
EGL Upstream of Drop	N/A	ft
HGL Upstream of Drop	N/A	ft

Freefall Condition

Depth Upstream of Drop	0.70
Velocity Upstream of Drop	4.75
EGL Upstream of Drop	193.80
HGL Upstream of Drop	193.45038

Condition Upstream of Drop — HGL 193.45, EGL 193.80

AERATOR INLET GATE

Downstream Flow	4.3		
Gates Open	2		
Flow per Gate	2.15	mgd =	3.3 cfs
Gate Width	1.5	ft	Reference M-14
Height of Gate	2.0	ft	Reference M-14
Invert Elevation of Gate	192.75		

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C	N/A	
Velocity through gate, v	N/A	fps

Not Submerged Condition

K	1.7		Modeled as gate frame (0.2) and entrance and exit (1.5)
Water Depth thru Gate	0.70	ft	
Velocity through Outlet, v	3.17	fps	

Energy Loss thru Gate, h_L 0.2646 ft

Condition Upstream of Gate — HGL 193.91, EGL 194.07

EFFLUENT REARATION CHAMBER PART 2

Friction Loss

Flow	2.15	mgd =	3.3 cfs
Channel Width	8.75	ft	
Total Channel Length	4.00	ft	
Downstream Invert El	191.75		Reference 9M-5
Upstream Invert El	191.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	2.16	ft	
Velocity (Average)	0.18	fps	
Hydraulic Radius (Average)	1.45	ft	

Friction Loss 0.0000 ft

Upstream Condition — HGL 194.06, EGL 194.07

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00 **REVISION:**

CHECKED : TL **DATE :** 2/6/2019 **BY :** WME **DATE :** 2/6/2017

Equation Ref.	HGL	EGL

EFFLUENT REARATION CHAMBER PART 1

Friction Loss

Downstream Flow	2.2		
Gates Open	2		
Upstream Flow	4.30	mgd =	6.7 cfs

Friction Loss

Flow	2.15	mgd =	3.3 cfs
Channel Width	18.50	ft	
Total Channel Length	6.00	ft	
Downstream Invert El	191.75		Reference 9M-5
Upstream Invert El	191.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	2.31	ft	
Velocity (Average)	0.08	fps	
Hydraulic Radius (Average)	1.85	ft	
Friction Loss	0.0000	ft	

Upstream Condition — HGL 194.07 — EGL 194.07

END REARATION STRUCTURE
BEGIN YARD

28-FE FROM EFFLUENT METERING STRUCTURE TO REAERATION STRUCTURE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	4.3	mgd =	6.7 cfs
Pipe Diameter, D	28	inch	
Pipe Length, L	60	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	1.56	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	362988		
Friction factor, f	0.0157		Equivalent Hazen-Williams "C" = 146.0627
Friction Energy Loss, h_L	0.02	ft	

MINOR PIPE LOSS HEADING

Flow, Q 4.3 mgd = 6.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	4.30	6.65	0.50	----	28	----	1.56	0.04	0.02
1	Mitre Bend - 22.5 ° Deflection	4.30	6.65	0.15	28	----	1.56	----	0.04	0.01
1	Mitre Bend - 22.5 ° Deflection	4.30	6.65	0.15	28	----	1.56	----	0.04	0.01
1	Outlet Loss - Still Water	4.30	6.65	1.00	28	----	1.56	----	0.04	0.04
									Sum =	0.07

Total Energy Loss = 0.08 ft

Upstream Condition — HGL 194.15 — EGL 194.15

END YARD
AREA 9 **BEGIN EFFLUENT METERING STRUCTURE**

EFFLUENT METERING STRUCTURE

Friction Loss

Flow	4.30	mgd =	6.7 cfs
Channel Width	5.00	ft	Reference 9S-12
Total Channel Length	4.00	ft	Reference 9S-12
Downstream Invert El	188.75		Reference 9S-12
Upstream Invert El	188.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	5.40	ft	
Velocity (Average)	0.25	fps	
Hydraulic Radius (Average)	1.71	ft	
Friction Loss	0.0000	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)

													Equation Ref.	HGL	EGL
1	90 Degree Bend	4.30	6.65	1.30	6.00	5.00	5.40	0.21	0.25	0.00	0.00				
									Sum Minor Loss=		0.0004	ft			

Total Energy Loss = 0.0004 ft

Upstream Condition — HGL 194.15 — EGL 194.15

EFFLUENT METERING STRUCTURE CONTROL POINT
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	4.3	mgd = 6.7 cfs	
WSE Downstream of Weir	194.15	ft	
Weir Crest Elevation	194.50	ft	Reference 9S-12
Downstream head, Hd	-0.35	ft	
Length of Weir, L	4.00	ft	Reference 9S-12

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.63	ft
Upstream WSE	195.13	ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.63 ft*

Condition Upstream of Weir — HGL 195.13 — EGL 195.13

EFFLUENT METERING STRUCTURE

Friction Loss

Flow	4.30	mgd = 6.7 cfs	
Channel Width	5.00	ft	Reference 9S-12
Total Channel Length	4.00	ft	Reference 9S-12
Downstream Invert El	188.75		Reference 9S-12
Upstream Invert El	188.75		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	6.38	ft	
Velocity (Average)	0.21	fps	
Hydraulic Radius (Average)	1.80	ft	

Friction Loss 0.0000 ft

Upstream Condition — HGL 195.13 — EGL 195.13

END EFFLUENT METERING STRUCTURE
BEGIN YARD

28-FE PIPE FROM UTILITY PUMP STATION TO EFFLUENT METERING STRUCTURE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	4.3	mgd = 6.7 cfs
Pipe Diameter, D	28	inch
Pipe Length, L	400	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.56	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	362988	
Friction factor, f	0.0157	Equivalent Hazen-Williams "C" = 146.0627

Friction Energy Loss, h_L 0.10 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.3 mgd = 6.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	4.30	6.65	0.50	----	28	----	1.56	0.04	0.02	
1	Mitre Bend - 90 º Deflection	4.30	6.65	1.27	28	----	1.56	----	0.04	0.05	
1	Mitre Bend - 45 º Deflection	4.30	6.65	0.32	28	----	1.56	----	0.04	0.01	
1	Mitre Bend - 15 º Deflection	4.30	6.65	0.06	28	----	1.56	----	0.04	0.00	
1	Outlet Loss - Still Water	4.30	6.65	1.00	28	----	1.56	----	0.04	0.04	
									Sum =	0.12	

Total Energy Loss = 0.22 ft

Upstream Condition HGL 195.35 EGL 195.35

END YARD

AREA 9 **BEGIN UTILITY PUMP STATION**

UTILITY PUMP STATION

Friction Loss

Flow	4.30 mgd =	6.7 cfs
Channel Width	18.00 ft	Reference 9S-5
Total Channel Length	6.00 ft	Reference 9S-5
Downstream Invert El	186.25	Reference 9S-6
Upstream Invert El	186.25	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	9.10 ft	
Velocity (Average)	0.04 fps	
Hydraulic Radius (Average)	4.52 ft	

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Turn Around Baffle	4.30	6.65	3.20	4	----	2	0.83	----	0.01	0.03
									Sum Minor Loss=		0.0344 ft

Total Energy Loss = 0.0344 ft

Upstream Condition HGL 195.38 EGL 195.38

UTILITY PUMP STATION CONTROL POINT 1
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	4.3 mgd =	6.7 cfs
WSE Downstream of Weir	195.38 ft	
Weir Crest Elevation	195.85 ft	Reference 9S-6
Downstream head, Hd	-0.47 ft	
Length of Weir, L	18.00 ft	Reference 9S-5

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation {6}
Head on Weir, H 0.23 ft
Upstream WSE 196.08 ft

Submerged Weir Computation {7}

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.23 ft*

Condition Upstream of Weir HGL 196.08 EGL 196.08

UTILITY PUMP STATION

Friction Loss

Flow	4.30 mgd =	6.7 cfs
Channel Width	18.00 ft	Reference 9S-12
Total Channel Length	6.00 ft	Reference 9S-12
Downstream Invert El	186.25	Reference 9S-12
Upstream Invert El	186.25	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	9.83 ft	
Velocity (Average)	0.04 fps	
Hydraulic Radius (Average)	4.70 ft	

	Equation Ref.	HGL	EGL
Friction Loss 0.0000 ft			
Upstream Condition		196.08	196.08

END UTILITY PUMP STATION
BEGIN YARD

BYPASS PIPE FROM UV REACTOR TO UTILITY WATER PUMP STATION
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 4.3 mgd = 6.7 cfs

Pipe Diameter, D	24	inch
Pipe Length, L	12	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.12	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	423486	
Friction factor, f	0.0157	

Equivalent Hazen-Williams "C" = 144.4555

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.3 mgd = 6.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	4.30	6.65	0.50	----	24	----	2.12	0.07	0.03
1	Butterfly Valve (Open)	4.30	6.65	0.50	24	----	2.12	----	0.07	0.03
1	Butterfly Valve (Open)	4.30	6.65	0.50	24	----	2.12	----	0.07	0.03
2	Tee - Thru Straight Run	4.30	6.65	0.60	24	----	2.12	----	0.07	0.08
1	Outlet Loss - Still Water	4.30	6.65	1.00	24	----	2.12	----	0.07	0.07
									Sum =	0.26

Total Energy Loss = 0.26 ft

	HGL	EGL
Upstream Condition	196.35	196.35

END YARD
START: UV REACTOR

UV EFFLUENT

Friction Loss

Flow	4.30	mgd = 6.7 cfs	
Channel Width	4.00	ft	Reference 15S-1
Total Channel Length	6.00	ft	Reference 15S-1
Downstream Invert El	190.00		Reference 15S-4
Upstream Invert El	190.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	6.35	ft	
Velocity (Average)	0.26	fps	
Hydraulic Radius (Average)	1.52	ft	

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 Degree Bend - 0° Radius	4.30	6.65	1.30	4.00	----	6.35	0.26	----	0.00	0.00
									Sum Minor Loss=		0.0014 ft

Total Energy Loss = 0.0014 ft

	HGL	EGL
Upstream Condition	196.35	196.35

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 4.3 mgd = 6.7 cfs

Downstream HGL < Invert + Crit. Depth: CRITICAL DEPTH USED

Downstream WSE	196.35	ft	
Downstream EGL	196.35	ft	
Channel Width, W	5.5	ft	Reference 15S-1
Critical. Depth, y_c	0.36	ft	

			Equation Ref.	HGL	EGL
Channel Invert @ Exit	196.25 ft				
Flooded Condition					
Depth Upstream of Drop	N/A ft				
Velocity Upstream of Drop	N/A fps				
Channel Exp./Bend "K"	2.00	(Note: Modify K value as appropriate)			
Energy Loss	N/A ft				
EGL Upstream of Drop	N/A ft				
HGL Upstream of Drop	N/A ft				
Freefall Condition					
Depth Upstream of Drop	0.36				
Velocity Upstream of Drop	3.39				
EGL Upstream of Drop	196.79				
HGL Upstream of Drop	196.60682				
		Condition Upstream of Drop		196.61	196.79

UV REACTOR

Friction Loss

			Equation Ref.	HGL	EGL
Flow	4.30 mgd =	6.7 cfs			
Channel Width	5.50 ft	Reference 15S-1			
Total Channel Length	70.00 ft	Reference 15S-1			
Downstream Invert El	195.25	Reference 15S-4			
Upstream Invert El	195.25				
Slope	0.00%				
Manning Coeff, n	0.015				
Depth (Average)	1.36 ft				
Velocity (Average)	0.89 fps				
Hydraulic Radius (Average)	0.91 ft				
Friction Loss	0.0064 ft				
		Upstream Condition		196.78	196.79

UV REACTOR CONTROL POINT 1
[STRAIGHT EDGED SHARP CRESTED WEIR]

			Equation Ref.	HGL	EGL
Flow	4.3 mgd =	6.7 cfs			
WSE Downstream of Weir	196.35 ft				
Weir Crest Elevation	197.75 ft	Reference 15S-4			
Downstream head, Hd	-1.40 ft				
Length of Weir, L	5.50 ft				
WEIR IS FREE-DISCHARGING					
Free Discharging Weir Computation			{ 6 }		
Head on Weir, H	0.51 ft				
Upstream WSE	198.26 ft				
Submerged Weir Computation			{ 7 }		
K	NA				
M	NA				
Increment	NA ft				
Upstream Head, Hu1	NA ft				
F(H1)	NA				
F'(H1)	NA				
Upstream Head, Hu2	NA ft				
Upstream WSE	NA ft				
Head over Weir	*0.51 ft*				
		Condition Upstream of Weir		198.26	198.26

END: **UV REACTOR**

START: **YARD**

24-FE FROM FILTERS EFFLUENT BOX TO UV REACTOR
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	4.3 mgd =	6.7 cfs			
Pipe Diameter, D	24 inch				
Pipe Length, L	15 ft				
Absolute Roughness, ε	0.00040 ft				
Pipe velocity, v	2.12 fps				
Kinematic Viscosity	1.000E-05 ft^2/sec				
Reynold's Number, R	423486				
Friction factor, f	0.0157	Equivalent Hazen-Williams "C" = 144.4555			
Friction Energy Loss, h_L	0.01 ft				

MINOR PIPE LOSS HEADING

	Equation Ref.	HGL	EGL

Flow, Q 4.3 mgd = 6.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee - Thru Side Outlet	4.30	6.65	1.80	24	----	2.12	----	0.07	0.13
1	Outlet Loss - Still Water	4.30	6.65	1.00	24	----	2.12	----	0.07	0.07
									Sum =	0.19

Total Energy Loss = 0.20 ft

		HGL	EGL
Upstream Condition		198.46	198.4622

FILTERS 1&2 AND FILTERS 3&4 FLOWS JOIN TO UV REACTOR
FLOW SPLIT

Downstream Flow	4.3 mgd =	6.7 cfs	
Filter Halves in Service	2		
New Flow	2.2 mgd =	3.3 cfs	Assume even flow split

20- FROM FILTERS 3&4 EFFLUENT BOX TO UV REACTOR
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	2.2 mgd =	3.3 cfs
Pipe Diameter, D	20 inch	
Pipe Length, L	20 ft	
Absolute Roughness, ε	0.00040 ft	
Pipe velocity, v	1.52 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	254092	
Friction factor, f	0.0169	Equivalent Hazen-Williams "C" = 144.9581
Friction Energy Loss, h_L	0.01 ft	

MINOR PIPE LOSS HEADING

Flow, Q 2.2 mgd = 3.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Entrance Loss - Flush	2.15	3.33	0.50	----	20	----	1.52	0.04	0.00
2	90 ° Elbow - Regular Fl.	2.15	3.33	0.30	20	----	1.52	----	0.04	0.02
0	Increaser	2.15	3.33	0.25	20	24	1.52	1.06	0.02	0.00
0	Tee - Thru Side Outlet	2.15	3.33	1.80	20	----	1.52	----	0.04	0.00
1	Mitre Bend - 45 ° Deflection	2.15	3.33	0.32	24	----	1.06	----	0.02	0.01
									Sum =	0.03

Total Energy Loss = 0.03 ft

		HGL	EGL
Upstream Condition		198.50	198.50

PIPE FROM FILTERS 3&4 EFFLUENT BOX TO UV REACTOR
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	2.2 mgd =	3.3 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	6 ft	
Absolute Roughness, ε	0.00040 ft	
Pipe velocity, v	4.23 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	423486	
Friction factor, f	0.0172	Equivalent Hazen-Williams "C" = 138.2215
Friction Energy Loss, h_L	0.03 ft	

MINOR PIPE LOSS HEADING

Flow, Q 2.2 mgd = 3.3 cfs

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	Equation Ref.	HGL	EGL

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	2.15	3.33	0.50	----	12	----	4.23	0.28	0.14
1	Tee - Thru Side Outlet	2.15	3.33	1.80	20	----	1.52	----	0.04	0.06
									Sum =	0.20

Total Energy Loss = 0.23 ft

Upstream Condition 198.73 198.73

END: **YARD**
START: **FLOCCULATION BASIN**

FILTER EFFLUENT CHIMNEY

Friction Loss

Flow	2.15 mgd =	3.3 cfs	
Channel Width	2.50 ft		Reference 8S-1
Total Channel Length	11.50 ft		Reference 8S-1
Downstream Invert El	185.00		Reference 8S-3
Upstream Invert El	185.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	13.73 ft		
Velocity (Average)	0.10 fps		
Hydraulic Radius (Average)	1.15 ft		

Friction Loss 0.0000 ft

Upstream Condition 198.73 198.73

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Downsteram Flow	2.15
Filters per Launder	2
Flow per Filter	1.075

Flow, Q 1.1 mgd = 1.7 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	198.73 ft	
Downstream EGL	198.73 ft	
Channel Width, W	1.5 ft	Reference 8S-3
Critical. Depth, y_c	0.34 ft	
Channel Invert @ Exit	197.50 ft	Reference 8S-3

Flooded Condition

Depth Upstream of Drop	1.24 ft	
Velocity Upstream of Drop	0.89 fps	
Channel Exp./Bend "K"	2.00	(Note: Modify K value as appropriate)
Energy Loss	0.02 ft	
EGL Upstream of Drop	198.75 ft	
HGL Upstream of Drop	198.74 ft	

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

Condition Upstream of Drop 198.74 198.75

FILTER EFFLUENT

Friction Loss

Flow	1.08 mgd =	1.7 cfs	
Channel Width	1.50 ft		Reference 8S-1
Total Channel Length	10.60 ft		Reference 8S-3
Downstream Invert El	197.50		Reference 8S-3
Upstream Invert El	197.50		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.24 ft		
Velocity (Average)	0.89 fps		
Hydraulic Radius (Average)	0.47 ft		

Friction Loss 0.0024 ft

Upstream Condition 198.74 198.76

FILTER EFFLUENT LAUNDER
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 1.1 mgd = 1.7 cfs

		Equation Ref.	HGL	EGL
WSE Downstream of Weir	198.74 ft			
Weir Crest Elevation	200.00 ft Reference 8S-3			
Downstream head, Hd	-1.26 ft			
Length of Weir, L	10.60 ft Reference 8S-3			

WEIR IS FREE-DISCHARGING

		Equation Ref.	HGL	EGL
Free Discharging Weir Computation		{ 6 }		
Head on Weir, H	0.13 ft			
Upstream WSE	200.13 ft			
Submerged Weir Computation		{ 7 }		
K	NA			
M	NA			
Increment	NA ft			
Upstream Head, Hu1	NA ft			
F(H1)	NA			
F'(H1)	NA			
Upstream Head, Hu2	NA ft			
Upstream WSE	NA ft			
Head over Weir	*0.13 ft*			
Condition Upstream of Weir			200.13	200.13

FILTERS

				HGL	EGL
Flow per Filter	1.08	mgd			
Filter Headloss	30.00	in	Assumed		
Upstream Condition				202.63	202.63

FILTERS

Friction Loss

Flow	1.08 mgd =	1.7 cfs	
Channel Width	14.00 ft		Reference 8S-3
Total Channel Length	14.00 ft		Reference 8S-3
Downstream Invert El	185.00		Reference 8S-3
Upstream Invert El	185.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	17.63 ft		
Velocity (Average)	0.01 fps		
Hydraulic Radius (Average)	5.01 ft		
Friction Loss	0.0000 ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Turn Around Baffle	1.08	1.66	3.20	2.50	----	2.5	0.27	----	0.00	0.00
									Sum Minor Loss=		0.0035 ft

Total Energy Loss = 0.0035 ft

		HGL	EGL
Upstream Condition		202.63	202.63

FILTER INFLUENT WEIR (downward opening weir gate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	1.1 mgd = 1.7 cfs	
WSE Downstream of Weir	202.63 ft	
Weir Crest Elevation	203.21 ft	Assumed EL per G-3
Downstream head, Hd	-0.58 ft	Gate is 36x42, low position 201.35, high position 204.85 (not full closed)
Length of Weir, L	3.00 ft	

WEIR IS FREE-DISCHARGING

		Equation Ref.
Free Discharging Weir Computation		{ 6 }
Head on Weir, H	0.30 ft	
Upstream WSE	203.51 ft	
Submerged Weir Computation		{ 7 }
K	NA	
M	NA	
Increment	NA ft	
Upstream Head, Hu1	NA ft	
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA ft	

Hydraulix BY carollo

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				Equation Ref.	HGL	EGL
Upstream WSE	NA	ft				
Head over Weir	*0.30*	*ft*				
			Condition Upstream of Weir		203.51	203.51

FLOW SPLIT

Downstream Flow	1.1	mgd =	1.7	cfs
Filters in Service	4			
New Flow	4.3	mgd =	6.7	cfs

Friction Loss

Flow	4.30	mgd =	6.7	cfs
Channel Width	6.00	ft	Reference 8S-3	
Total Channel Length	60.00	ft	Reference 8S-2	
Downstream Invert El	197.50		Reference 8S-3	
Upstream Invert El	197.50			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	6.01	ft		
Velocity (Average)	0.18	fps		
Hydraulic Radius (Average)	2.00	ft		
Friction Loss	0.0001	ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 Degree Bend - 0° Radius	4.30	6.65	1.30	6.00	----	6.00	0.18	----	0.00	0.00
										Sum Minor Loss=	0.0007 ft

Total Energy Loss = 0.0008 ft

	HGL	EGL
Upstream Condition	203.51	203.51

FLOCCULATION BASIN 1&2 DISCHARGE TO CHANNEL

FLOW SPLIT

Downstream Flow	4.3	mgd =	6.7	cfs
Flocculation Basins	2			
New Flow	2.2	mgd =	3.3	cfs

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	2.15	mgd =	3.3 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	3.5	ft	Reference M-14
Invert Elevation of Gate	200.50		Reference 8S-5
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	1.7		Modeled as gate frame (0.2), entrance (0.5), exit(1.0)
Water Depth thru Gate	3.01	ft	
Velocity through Outlet, v	0.37	fps	
Energy Loss thru Gate, h_L	0.0036	ft	

	HGL	EGL
Condition Upstream of Gate	203.51	203.52

FLOCCULATION BASIN

Friction Loss

Flow	2.15	mgd =	3.3	cfs
Channel Width	6.00	ft	Reference 8S-3	
Total Channel Length	60.00	ft	Reference 8S-2	
Downstream Invert El	197.50		Reference 8S-3	
Upstream Invert El	197.50			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	6.01	ft		
Velocity (Average)	0.09	fps		
Hydraulic Radius (Average)	2.00	ft		
Friction Loss	0.0000	ft		

Minor Loss

Flow	Flow	Width Up	Width Down	Depth	Vel Up	Vel Down	Vel Head	Minor Loss

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No.	Description	(mgd)	(cfs)	K	(ft)	(ft)	(ft)	(fps)	(fps)	(ft)	(ft)	Equation Ref.	HGL	EGL
1	Turn Around Baffle	2.15	3.33	3.20	5	----	5	0.13	----	0.00	0.00			
									Sum Minor Loss=		0.0009 ft			

Minor Loss

Flocculation Basin 2.00 in

Total Energy Loss = 0.1676 ft

Upstream Condition		203.68	203.68
END:	**FLOCCULATION BASIN**		

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

Hydraulix® by carollo

JOB # : 101321A00 **REVISION:**

CHECKED : TL **DATE :** 2/6/2019 **BY :** WME **DATE :** 2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Bioreactors	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

			HGL	EGL
EGL =	199.05		199.05	199.05
Flow =	4.30 mgd =	6.65 cfs		

AREA 7 **FILTER INFLUENT PUMP STATION**

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	4.30	mgd
Flow to EQ Basin	0.00	mgd
Influent Flow	4.30	mgd
Filter Backwash	0.17	mgd
Upstream Flow	4.47	mgd

HWL	199.05	ASSUMED USE HWL AT ALL FLOWS	REFERENCE 7M-1
LWL	185.00		

PS Wetwell Elevation — *Upstream Condition* — HGL 199.05 — EGL 199.05

FILTER INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	4.47 mgd =	6.9 cfs		
Channel Width	8.25 ft		Reference 7M-1	Assumed, need S dwgs
Total Channel Length	18.00 ft		Reference 7M-2	Assumed, need S dwgs
Downstream Invert El	176.00		Reference 7M-3	Assumed, need S dwgs
Upstream Invert El	176.00			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	23.05 ft			
Velocity (Average)	0.04 fps			
Hydraulic Radius (Average)	3.50 ft			

Friction Loss 0.0000 ft

Other Loss

Turbulence 0.00 in Assumed

Total Energy Loss = 0.0000 ft

Upstream Condition — HGL 199.05 — EGL 199.05

FILTER INFLUENT PUMP STATION
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 4.5 mgd = 6.9 cfs

WSE Downstream of Weir	199.05 ft	
Weir Crest Elevation	191.25 ft	Assumed per Reference 7M-1
Downstream head, Hd	7.80 ft	
Length of Weir, L	18.00 ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation {6}

Head on Weir, H	NA ft
Upstream WSE	NA ft

Submerged Weir Computation {7}

K	0.00
M	21.78
Increment	0.10 ft
Upstream Head, Hu1	7.80 ft
F(H1)	0.00
F'(H1)	-0.19
Upstream Head, Hu2	7.80 ft

Equation Ref.	HGL	EGL

Upstream WSE 199.05 ft

Head over Weir 7.80 ft

Condition Upstream of Weir	199.05	199.05

CLARIFIER 1&2 EFFLUENT JUNCTION
FLOW SPLIT

Downstream Flow	4.5	mgd =	6.9	cfs
No. of clarifiers	2.0			
New Flow	2.2	mgd =	3.5	cfs

20-SE SECONDARY CLARIFIER DISCHARGE PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	2.2	mgd = 3.5 cfs
Pipe Diameter, D	20	inch
Pipe Length, L	15	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.59	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	264255	
Friction factor, f	0.0168	Equivalent Hazen-Williams "C" = 144.8467
Friction Energy Loss, h_L	0.01	ft

MINOR PIPE LOSS HEADING

Flow, Q 2.2 mgd = 3.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	2.24	3.46	1.00	----	20	----	1.59	0.04	0.04
1	Mitre Bend - 45 º Deflection	2.24	3.46	0.32	20	----	1.59	----	0.04	0.01
1	Tee - Thru Straight Run	2.24	3.46	0.60	20	----	1.59	----	0.04	0.02
1	Outlet Loss - Still Water	2.24	3.46	1.00	20	----	1.59	----	0.04	0.04
									Sum =	0.11

Total Energy Loss = 0.12 ft

Upstream Condition	199.17	199.17

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 2.2 mgd = 3.5 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	199.17	ft	
Downstream EGL	199.17	ft	
Channel Width, W	2.0	ft	Reference 5S-2
Critical. Depth, y_c	0.45	ft	
Channel Invert @ Exit	197.55	ft	Reference 5S-2

Flooded Condition

Depth Upstream of Drop	1.64	ft
Velocity Upstream of Drop	1.06	fps
Channel Exp./Bend "K"	2.00	
Energy Loss	0.03	ft
EGL Upstream of Drop	199.20	ft
HGL Upstream of Drop	199.19	ft

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

Condition Upstream of Drop	199.19	199.20

SECONDARY CLARIFIER EFFLUENT LAUNDER

Clarifier Diameter	85	feet	Reference 5S-2
Weir Diameter	80	feet	

				Equation Ref.	HGL	EGL
Flow						
Downstream Flow	2.2	mgd =	3.5			
Split launder	2.0					
New Flow	1.1	mgd =	1.7			
Friction Loss						
Flow	2.24	mgd =	3.5 cfs			
Channel Width	2.00	ft	Reference 5S-2			
Total Channel Length	130.38	ft	Reference 5S-2			
Downstream Invert El	197.55		Reference 5S-2			
Upstream Invert El	198.37					
Slope	0.63%					
Manning Coeff, n	0.015					
Depth (Average)	1.23	ft				
Velocity (Average)	1.41	fps				
Hydraulic Radius (Average)	0.55	ft				
Friction Loss	0.0584	ft				
			Upstream Condition		199.23	199.26
SECONDARY CLARIFIER EFFLUENT WEIR [V-NOTCH WEIR]						
Flow	2.24	mgd =	3.5 cfs			
WSE Downstream of Weir	199.23	ft				
Weir Crest Elevation	200.43	ft	Reference 5S-4			
Downstream head, Hd	-1.20	ft				
Weir Length	251.33	ft				
Distance Between Notches	6.00	in	Reference 5S-4			
Number of Notches	502					
WEIR IS FREE-DISCHARGING						
Free Discharging Weir Computation				{ 8 }		
Head on Weir, H	0.09	ft				
Upstream WSE	200.52	ft				
Submerged Weir Computation				{ 9 }		
K	NA					
M	NA					
Increment	NA	ft				
Upstream Head, Hu1	NA	ft				
F(H1)	NA					
F'(H1)	NA					
Upstream Head, Hu2	NA	ft				
Upstream WSE	NA	ft				
Head over Weir	*0.09*	*ft*				
			Condition Upstream of Weir		200.52	200.52
SECONDARY CLARIFIER INFLUENT BAFFLE [V-NOTCH WEIR]						
Downstream Flow	2.24					
RAS	0.26	mgd =				
Dewatering Recycles	0.14					
Upstream Flow	2.6	mgd =	4.1 cfs			
WSE Downstream of Weir	200.52	ft				
Weir Crest Elevation	200.43	ft				
Downstream head, Hd	0.09	ft				
Weir Length	78.54	ft	Assumed			
Distance Between Notches	6.00	in	Assumed			
Number of Notches	157					
WEIR IS SUBMERGED						
Free Discharging Weir Computation				{ 8 }		
Head on Weir, H	NA	ft				
Upstream WSE	NA	ft				
Submerged Weir Computation				{ 9 }		
K	0.00					
M	0.00					
Increment	0.10	ft				
Upstream Head, Hu1	0.17	ft				
F(H1)	0.00					
F'(H1)	-33.02					
Upstream Head, Hu2	0.17	ft				
Upstream WSE	200.60	ft				

	Equation Ref.	HGL	EGL

Head over Weir *0.17 ft*

Condition Upstream of Weir		200.60	200.60

SECONDARY CLARIFIER FLOCCULATING WELL

- Treat as a submerged orifice.

Flow, Q	2.64	mgd =	4.1 cfs
Downstream WSE	200.52	ft	
Flocculation Diameter	25.00	ft	
EDI Diameter	8.50	ft	
Opening Area	434	sf	
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.01	fps	
Energy Loss , h_L	0.00	ft	

Condition in Flocculating Well		200.60	200.60

SECONDARY CLARIFIER ENERGY DISSIPATION INLET

- Treat as a submerged orifice
- Assume Upstream EGL = HGL

Flow, Q	2.64	mgd =	4.1 cfs
Downstream WSE	200.60	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.00	sf	
Total Port Area	16	sf	
Submerged Port Area	189453%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.26	fps	
Energy Loss , h_L	0.00	ft	

Condition in Influent Well		200.60	200.60

SECONDARY CLARIFIER CENTER COLUMN OUTLETS

- Treat as a submerged orifice

Orifice Loss

Flow, Q	2.64	mgd =	4.1 cfs
Downstream WSE	200.60	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.0	sf	
Total Port Area	16	sf	
Submerged Port Area	189457%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.26	fps	
Energy Loss , h_L	0.00	ft	

Minor Losses

Flow, Q 2.64 mgd = 4.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mounding Loss	2.64	4.09	0.25	22	----	1.55	----	0.04	0.01
									Sum =	0.01

Total Energy Loss = *0.01 ft*

		200.61	200.61

22"ML SECONDARY CLARIFIER CENTER COLUMN
PIPE FRICTION LOSSES (MANNING) - Full Pipe Flow Only

Flow	2.64	mgd =	4.1 cfs **Flow + Total Recycle + RAS**
Pipe Diameter, D	22	inch	
Pipe Length, L	22	ft	
Manning Coef., n	0.015	ft	

	Equation Ref.	HGL	EGL

Velocity 1.55 fps
Hydraulic Radius 0.46 ft

Friction Energy Loss 0.02 ft

MINOR PIPE LOSS HEADING

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Bend	2.64	4.09	0.60	22	----	1.55	----	0.04	0.02

Total Minor Losses = 0.02 ft

Total Energy Loss = 0.04 ft

		HGL	EGL
Clarifier center column	*Upstream Condition*	200.61	200.65

22"ML JUNCTION BOX TO CLARIFIER 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 2.6 mgd = 4.1 cfs

Pipe Diameter, D 22 inch
Pipe Length, L 220 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.55 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 283745
Friction factor, f 0.0165 Equivalent Hazen-Williams "C" = 145.2635

Friction Energy Loss, h_L 0.07 ft

MINOR PIPE LOSS HEADING

Flow, Q 2.6 mgd = 4.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mitre Bend - 90 º Deflection	2.64	4.09	1.27	22	----	1.55	----	0.04	0.05
1	Mitre Bend - 45 º Deflection	2.64	4.09	0.32	22	----	1.55	----	0.04	0.01
1	Entrance Loss - Flush	2.64	4.09	0.50	----	22	----	1.55	0.04	0.02
									Sum =	0.078

Total Energy Loss = 0.15 ft

	HGL	EGL
Upstream Condition	200.80	200.80

AREA 4 **MIXED LIQUOR SPLITTER BOX**

ML JUNCTION FLOW SPLIT
FLOW SPLIT

Downstream Flow 2.6 mgd = 4.1 cfs
No. of SCs Oline 2.0
New Flow 5.3 mgd = 8.2 cfs

MIXED LIQUOR SPLITTER EFFLUENT

Friction Loss

Flow	5.28	mgd =	8.2 cfs
Channel Width	6.00	ft	Reference 4S-1
Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	9.80	ft	
Velocity (Average)	0.14	fps	
Hydraulic Radius (Average)	2.30	ft	

Friction Loss 0.0000 ft

Other Loss

				Equation Ref.	HGL	EGL
Turbulence	0.00	in	Assumed			
Total Energy Loss =	0.0000 ft					
			Upstream Condition		200.80	200.80

ML SPLITTER BOX WEIR (downward opening weir gates)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	5.3	mgd =	8.2 cfs	
WSE Downstream of Weir	200.80	ft		
Weir Crest Elevation	201.97	ft	Assume EL per Reference G-3	60"x24" weir gate.
Downstream head, Hd	-1.17	ft	Low position 201.75, top of STR opening is 203.25, so can full close	
Length of Weir, L	5.00	ft		

WEIR IS FREE-DISCHARGING

			Equation Ref.
Free Discharging Weir Computation			{ 6 }
Head on Weir, H	0.62	ft	
Upstream WSE	202.59	ft	
Submerged Weir Computation			{ 7 }
K	NA		
M	NA		
Increment	NA	ft	
Upstream Head, Hu1	NA	ft	
F(H1)	NA		
F'(H1)	NA		
Upstream Head, Hu2	NA	ft	
Upstream WSE	NA	ft	
Head over Weir	*0.62*	*ft*	

	HGL	EGL
Condition Upstream of Weir	202.59	202.59

MIXED LIQUOR SPLITTER INFLUENT

Friction Loss

Flow	5.28	mgd =	8.2 cfs
Channel Width	15.00	ft	Reference 4S-1
Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	11.59	ft	
Velocity (Average)	0.05	fps	
Hydraulic Radius (Average)	4.55	ft	
Friction Loss	0.0000	ft	

Other Loss

Turbulence	0.00	in	Assumed
Total Energy Loss =	0.0000	ft	

	HGL	EGL
Upstream Condition	202.59	202.59

33"ML FROM OX DITCH TEE TO ML SPLITTER BOX
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	5.3	mgd =	8.2 cfs
Pipe Diameter, D	33	inch	
Pipe Length, L	95	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	1.38	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	378326		
Friction factor, f	0.0154		Equivalent Hazen-Williams "C" = 146.9184
Friction Energy Loss, h_L	0.02	ft	

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 5.3 mgd = 8.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	5.28	8.17	0.50	----	33	----	1.38	0.03	0.01
1	Tee - Thru Side	5.28	8.17	1.80	33	----	1.38	----	0.03	0.05
									Sum =	0.07

Total Energy Loss = 0.08 ft

Upstream Condition 202.68 202.68

OX DITCH TEE
FLOW SPLIT

Downstream Flow	5.3 mgd =	8.2	cfs
Ox Ditch online	2		
New Flow	2.6 mgd =	4.1	cfs

22"ML FROM OX DITCH 1 TEE TO OX DITCH TEE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 2.6 mgd = 4.1 cfs

Pipe Diameter, D	22	inch
Pipe Length, L	50	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	1.55	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	283745	
Friction factor, f	0.0165	

Equivalent Hazen-Williams "C" = 145.2635

Friction Energy Loss, h_L 0.02 ft

MINOR PIPE LOSS HEADING

Flow, Q 2.6 mgd = 4.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Reducer	2.64	4.09	0.25	33	22	0.69	1.55	0.04	0.01
3	Mitre Bend - 90 º Deflection	2.64	4.09	1.27	22	----	1.55	----	0.04	0.14
									Sum =	0.15

Total Energy Loss = 0.17 ft

Upstream Condition 202.84 202.84

AREA 4 **BIOLOGICAL REACTORS**

OXIDATION DITCH EFFLUENT WEIR (motorized weir)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 2.64 mgd = 4.1 cfs

WSE Downstream of Weir	202.84	ft	
Weir Crest Elevation	206.03	ft	low position: 204.96; high position: 206.03 — Reference 4M-5
Downstream head, Hd	-3.19	ft	
Length of Weir, L	15.00	ft	Reference M-13

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.19	ft
Upstream WSE	206.22	ft

Submerged Weir Computation { 7 }

K NA

			Equation Ref.	HGL	EGL
M	NA				
Increment	NA	ft			
Upstream Head, Hu1	NA	ft			
F(H1)	NA				
F'(H1)	NA				
Upstream Head, Hu2	NA	ft			
Upstream WSE	NA	ft			
Head over Weir	*0.19*	*ft*			
			Condition Upstream of Weir	206.22	206.22

AEROBIC ZONE

Friction Loss

Flow	2.64	mgd =	4.1 cfs	
Channel Width	30.25	ft		Reference 4S-1
Total Channel Length	256.00	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-4
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	13.93	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.25	ft		
Friction Loss	0.0000	ft		

Other Loss

Baffles	1.00	in	Assumed
Total Energy Loss =	0.0833	ft	

	HGL	EGL
Upstream Condition	206.30	206.30

TRANSITION FROM ANOXIC TO AEROBIC

Friction Loss

Flow	2.64	mgd =	4.1 cfs	
Channel Width	2.50	ft	Reference 4s-4	
Total Channel Length	30.00	ft		
Downstream Invert El	192.29			
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	14.01	ft		
Velocity (Average)	0.12	fps		
Hydraulic Radius (Average)	1.15	ft		
Friction Loss	0.0000	ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	2.64	4.09	1.00	5.00	8.00	14.01	0.06	0.04	0.00	0.00
1	Sudden Contraction	2.64	4.09	0.50	8.00	5.00	14.01	0.04	0.06	0.00	0.00
									Sum Minor Loss=		0.00 ft

Total Energy Loss = 0.0001 ft

	HGL	EGL
Upstream Condition	206.30	206.30

ANOXIC ZONE

Friction Loss

Flow	2.64	mgd =	4.1 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	14.01	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.19	ft		
Friction Loss	0.0000	ft		

Other Loss

Vertical Mixer	0.50	in	Assumed

	Equation Ref.	HGL	EGL

Total Energy Loss = 0.0417 ft

Upstream Condition		206.34	206.34

ANAEROBIC REACTOR EFFLUENT WEIR (downward opening weir gate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir 2.6 mgd = 4.1 cfs

WSE Downstream of Weir 206.34 ft
Weir Crest Elevation 206.71 ft — INV opening: 204.96. Gate height 30-inches. — Reference 4S-7
Downstream head, Hd -0.37 ft — Therefore: low position 204.96, high position 207.46 — Reference M-14
Length of Weir, L 3.00 ft

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }
Head on Weir, H 0.55 ft
Upstream WSE 207.26 ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir 0.55 ft

Condition Upstream of Weir		207.26	207.26

ANAEROBIC ZONE 3

Friction Loss

Downstream Flow	2.6
Ox Ditches in Service	2
Upstream Flow	5.2820023

Friction Loss

Flow	5.28	mgd = 8.2 cfs	
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	14.97	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	7.43	ft	

Friction Loss 0.0000 ft

Other Loss

Vertical Mixer 0.50 in — Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition		207.30	207.30

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 2 and 3)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir 5.3 mgd = 8.2 cfs
INV opening: 204.96. Gate height 42-inches. — Reference M-14
WSE Downstream of Weir 207.30 ft — Therefore: low assume position 204.96 and assume high position 208.46 at full close
Weir Crest Elevation 204.96 ft — ASSUMED
Downstream head, Hd 2.34 ft
Length of Weir, L 3.00 ft — Reference 4S-7

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }
Head on Weir, H NA ft
Upstream WSE NA ft

	Equation Ref.	HGL	EGL

Submerged Weir Computation {7}

K	0.59	
M	3.59	
Increment	0.10	ft
Upstream Head, Hu1	2.37	ft
F(H1)	0.00	
F'(H1)	-0.65	
Upstream Head, Hu2	2.37	ft
Upstream WSE	207.33	ft

Head over Weir *2.37 ft*

Condition Upstream of Weir — HGL 207.33 — EGL 207.33

ANAEROBIC ZONE 2

Friction Loss

Flow	5.28	mgd =	8.2 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	15.04	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	7.45	ft	

Friction Loss 0.0000 ft

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition — HGL 207.38 — EGL 207.38

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 1 and 2)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	5.3	mgd =	8.2 cfs
			INV opening: 204.96. Gate height 42-inches. Reference M-14
WSE Downstream of Weir	207.38	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close
Weir Crest Elevation	204.96	ft	ASSUMED
Downstream head, Hd	2.42	ft	
Length of Weir, L	3.00	ft	Reference 4S-7

WEIR IS SUBMERGED

Free Discharging Weir Computation {6}

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation {7}

K	0.59	
M	3.76	
Increment	0.10	ft
Upstream Head, Hu1	2.45	ft
F(H1)	0.00	
F'(H1)	-0.63	
Upstream Head, Hu2	2.45	ft
Upstream WSE	207.41	ft

Head over Weir *2.45 ft*

Condition Upstream of Weir — HGL 207.41 — EGL 207.41

ANAEROBIC ZONE 1

Friction Loss

Flow	5.28	mgd =	8.2 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	15.12	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	7.47	ft	

Friction Loss 0.0000 ft

	Equation Ref.	HGL	EGL

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition		207.45	207.45

ANAEROBIC REACTOR INFLUENT GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 5.3 mgd = 8.2 cfs

Diameter of Opening 2.5 ft
Sluice Gate Percent Open 100%
Discharge Coefficient, C 0.61
Velocity through gate, v 1.66 fps

Energy Loss thru Gate, h_L 0.12 ft

Condition Upstream of Gate		207.56	207.56

FLOW CHANGE/SPLIT

Downstream Flow 5.28 mgd = 8.2 cfs
Anaerobic Bypass 0.00
Upstream Flow 5.28 mgd =

30" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 5.28 mgd = 8.2 cfs

Pipe Diameter, D 30 inch
Pipe Length, L 13 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.66 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 416159
Friction factor, f 0.0153 Equivalent Hazen-Williams "C" = 146.0625

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 5.3 mgd = 8.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Increaser	5.28	8.17	0.25	30	33	1.66	1.38	0.01	0.00
1	Outlet Loss - Still Water	5.28	8.17	1.00	33	----	1.38	----	0.03	0.03
									Sum =	0.03

Total Energy Loss = 0.04 ft

Upstream Condition		207.60	207.60

33" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 5.28 mgd = 8.2 cfs

Pipe Diameter, D 33 inch
Pipe Length, L 280 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.38 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 378326
Friction factor, f 0.0154 Equivalent Hazen-Williams "C" = 146.9184

Friction Energy Loss, h_L 0.05 ft

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 5.28 mgd = 8.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	Mitre Bend - 90 ° Deflection	5.28	8.17	1.27	33	----	1.38	----	0.03	0.07
1	Tee - standard	5.28	8.17	1.50	33	----	1.38	----	0.03	0.04
2	Mitre Bend - 45 ° Deflection	5.28	8.17	0.32	33	----	1.38	----	0.03	0.02
1	Tee - Thru Straight	5.28	8.17	0.60	33	----	1.38	----	0.03	0.02
									Sum =	0.15

Total Energy Loss = 0.20 ft

Upstream Condition 207.80 207.80

24" PI (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Downstream Flow	5.28	mgd
RAS split	0.52	mgd
Upstream Flow	4.76	mgd = 7.4 cfs
Pipe Diameter, D	24	inch
Pipe Length, L	120	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.34	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	468789	
Friction factor, f	0.0155	Equivalent Hazen-Williams "C" = 144.0122

Friction Energy Loss, h_L 0.08 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.76 mgd = 7.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	4.76	7.36	0.50	----	24	----	2.34	0.09	0.04
1	Mitre Bend - 90 ° Deflection	4.76	7.36	1.27	24	----	2.34	----	0.09	0.11
1	Tee - Standard	4.76	7.36	1.50	24	----	2.34	----	0.09	0.13
1	Increaser	4.76	7.36	0.25	24	33	2.34	1.24	0.06	0.02
									Sum =	0.29

Total Energy Loss = 0.37 ft

Upstream Condition 208.17 208.17

AREA 2 **GRIT REMOVAL AND FINE SCREENS**

24-PI INLET GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 4.8 mgd = 7.4 cfs

Diameter of Opening	2	ft
Sluice Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	2.34	fps

Energy Loss thru Gate, h_L 0.23 ft

Condition Upstream of Gate 208.40 208.40

FINE SCREEN EFFLUENT SUMP { 4 }

Friction Loss

				Equation Ref.	HGL	EGL
Flow	4.76 mgd =	7.4 cfs				
	=	3308.2 gpm				
Channel Width	3 ft		Reference	#21A		
Channel Height	5 ft		Reference	#21B		
Equivalent Pipe Diameter, D	52 inch					
Conduit Length, L	13.66 ft					
Roughness Coefficient, C	120					
Pipe velocity, v	0.49 fps					
Total Friction Loss	0.0003 ft					

MINOR PIPE LOSS HEADING

Flow, Q 4.8 mgd = 7.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Outlet Loss - Still Water	4.76	7.36	1.00	52	----	0.49	----	0.00	0.00
1	Reducer	4.76	7.36	0.25	52	52	0.49	0.49	0.00	0.00
									Sum =	0.00468

Total Energy Loss = 0.00 ft

	HGL	EGL
Upstream Condition	208.41	208.41

FLOW CHANGE/SPLIT

Downstream Flow	4.76 mgd =	7.4 cfs
Fine Screen Bypass	3.00	
Upstream Flow	1.76 mgd =	

FINE SCREEN EFFLUENT WEIR (redwood weir plate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	1.8 mgd =	2.7 cfs	
WSE Downstream of Weir	208.41 ft		
Weir Crest Elevation	208.17 ft		
Downstream head, Hd	0.24 ft		
Length of Weir, L	5.00 ft		

WEIR IS SUBMERGED

			Equation Ref.
Free Discharging Weir Computation			{ 6 }
Head on Weir, H	NA ft	11.52	
Upstream WSE	NA ft		
Submerged Weir Computation			{ 7 }
K	0.01		
M	0.12		
Increment	0.10 ft		
Upstream Head, Hu1	0.36 ft		
F(H1)	0.00		
F'(H1)	-7.23		
Upstream Head, Hu2	0.36 ft		
Upstream WSE	208.53 ft		
Head over Weir	*0.36 ft*		

	HGL	EGL
Condition Upstream of Weir	208.53	208.53

FINE SCREEN

Flow	1.8 mgd =	2.7 cfs
Blinding	50%	
Maximum Headloss	0.50 ft	

	HGL	EGL
Condition Upstream of Bar Screen	209.03	209.03

FINE SCREEN INFLUENT

Friction Loss

Downstream Flow	1.8
Fine Screen Bypass	3
Upstream Flow	4.8

Friction Loss

Flow	4.76 mgd =	7.4 cfs
Channel Width	5.00 ft	Reference 2S-1
Total Channel Length	25.50 ft	

			Equation Ref.	HGL	EGL
Downstream Invert El	207.67				
Upstream Invert El	207.67				
Slope	0.00%				
Manning Coeff, n	0.015				
Depth (Average)	1.36	ft			
Velocity (Average)	1.08	fps			
Hydraulic Radius (Average)	0.88	ft			
Friction Loss	0.0036	ft			

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee- Thru Straight	4.76	7.36	0.60	5.00	----	1.36	1.08	----	0.02	0.01
									Sum Minor Loss=		0.01 ft

Total Energy Loss = 0.0145 ft

Upstream Condition — HGL 209.03 — EGL 209.05

GRIT CHAMBER EFFLUENT WEIR
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 4.8 mgd = 7.4 cfs

WSE Downstream of Weir	209.03	ft	
Weir Crest Elevation	209.37	ft	Reference 2M-2
Downstream head, Hd	-0.34	ft	
Length of Weir, L	5.50	ft	Reference 2S-1

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.54	ft
Upstream WSE	209.91	ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.54 ft*

Condition Upstream of Weir — HGL 209.91 — EGL 209.91

GRIT CHAMBER

Flow 4.76 mgd = 7.4 cfs

Maximum Headloss 2.00 in Assumed

Condition Upstream of Bar Screen — HGL 210.08 — EGL 210.08

PIPE FROM HEADWORKS TO GRIT CHAMBER
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 4.8 mgd = 7.4 cfs

Pipe Diameter, D	20	inch
Pipe Length, L	80	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	3.38	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	562547	
Friction factor, f	0.0156	Equivalent Hazen-Williams "C" = 141.518

Friction Energy Loss, h_L 0.13 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.8 mgd = 7.4 cfs

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

Hydraulix BY carollo

JOB # : 101321A00 **REVISION:**

CHECKED : TL **DATE :** 2/6/2019 **BY :** WME **DATE :** 2/6/2017

Equation Ref.	HGL	EGL

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	1.19	1.84	1.00	----	12	----	2.34	0.09	0.09
1	Entrance Loss - Pipe Ext.	1.19	1.84	1.00	----	12	----	2.34	0.09	0.09
1	Entrance Loss - Pipe Ext.	1.19	1.84	1.00	----	12	----	2.34	0.09	0.09
1	Entrance Loss - Pipe Ext.	1.19	1.84	1.00	----	12	----	2.34	0.09	0.09
1	Tee - Thru Side Outlet	4.76	7.36	1.80	20	----	3.38	----	0.18	0.32
1	Reducer	4.76	7.36	0.25	20	16	3.38	5.27	0.43	0.11
1	Mag Meter	4.76	7.36	0	16	----	5.27	----	0.43	0.00
1	Increaser	4.76	7.36	0.25	16	16	5.27	5.27	0.00	0.00
1	Plug Valve (Open)	4.76	7.36	0.77	16	----	5.27	----	0.43	0.33
1	Increaser	4.76	7.36	0.25	16	20	5.27	3.38	0.25	0.06
1	Mitre Bend - 90 º Deflection	4.76	7.36	1.27	20	----	3.38	----	0.18	0.22
1	Mitre Bend - 90 º Deflection	4.76	7.36	1.27	20	----	3.38	----	0.18	0.22
1	Mitre Bend - 90 º Deflection	4.76	7.36	1.27	20	----	3.38	----	0.18	0.22
1	Mitre Bend - 90 º Deflection	4.76	7.36	1.27	20	----	3.38	----	0.18	0.22
1	Mitre Bend - 90 º Deflection	4.76	7.36	1.27	20	----	3.38	----	0.18	0.22
1	Outlet Loss - Still Water	4.76	7.36	1.00	20	----	3.38	----	0.18	0.18
									Sum =	2.46

Total Energy Loss = 2.59 ft

Influent Wet Well	*Upstream Condition*	212.51	212.50734

AREA 1 **HEADWORKS AND INFLUENT PUMP STATION**

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	4.76	mgd
Wetwells in service	2	
Flow Upstream	2.38	mgd
HHWL	182.92	
HWL	181.89	
LWL	180.42	
LLWL	178.42	

ASSUMED USE HWL AT ALL FLOWS REFERENCE 1M-3

PS Wetwell Elevation	*Upstream Condition*	181.89	181.89

INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	2.38	mgd = 3.7 cfs
Channel Width	13.83	ft
Total Channel Length	15.00	ft
Downstream Invert El	174.42	
Upstream Invert El	174.42	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	7.47	ft
Velocity (Average)	0.04	fps
Hydraulic Radius (Average)	3.59	ft

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Reducer	2.38	3.68	0.25	6.00	13.83	7.47	0.08	0.04	0.00	0.00
										Sum Minor Loss=	0.0000 ft

Total Energy Loss = 0.0000 ft

	Upstream Condition	181.89	181.89

IPS WETWELL INLET GATE

Flow per Gate	2.38	mgd = 3.7 cfs
Gate Width	3.5	ft
Height of Gate	5.0	ft
Invert Elevation of Gate	180.42	

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C N/A

			Equation Ref.	HGL	EGL
Velocity through gate, v	N/A	fps			
Not Submerged Condition					
K	1.7	Modeled as gate frame (0.2) and entrance and exit (1.5)			
Water Depth thru Gate	1.47	ft			
Velocity through Outlet, v	0.72	fps			
Energy Loss thru Gate, h_L	0.0135	ft			
			Condition Upstream of Gate	181.90	181.90

HEADWORKS EFFLUENT CHANNEL

Flow Downstream	2.38	mgd	
Wetwells in service	2		
Flow Upstream	4.76	mgd	

Friction Loss

Flow	2.38	mgd =	3.7 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	4.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.48	ft	
Velocity (Average)	0.83	fps	
Hydraulic Radius (Average)	0.74	ft	
Friction Loss	0.0004	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Side	2.38	3.68	1.80	6.00	3.00	1.48	0.42	0.83	0.01	0.01
									Sum Minor Loss=		0.0145 ft

Total Energy Loss = 0.0149 ft

	HGL	EGL
Upstream Condition	181.91	181.92

CHANNEL GRINDER EFFLUENT GATE

Flow per Gate	4.76	mgd =	7.4 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.49	ft	
Velocity through Outlet, v	1.65	fps	
Energy Loss thru Gate, h_L	0.0085	ft	

	HGL	EGL
Condition Upstream of Gate	181.88	181.93

CHANNEL GRINDER

Friction Loss

Flow	4.76	mgd =	7.4 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	16.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.46	ft	
Velocity (Average)	1.68	fps	
Hydraulic Radius (Average)	0.74	ft	
Friction Loss	0.0068	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
3	45 degree bend	4.76	7.36	0.25	6.00	4.76	1.46	0.84	1.06	0.01	0.00
									Sum Minor Loss=		0.0048 ft

	Equation Ref.	HGL	EGL

Other

Grinder 2.00 in Assumed

Total Energy Loss = 0.1783 ft

Upstream Condition		182.06	182.11

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	4.76	mgd =	7.4 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.64	ft	
Velocity through Outlet, v	1.50	fps	

Energy Loss thru Gate, h_L 0.0069 ft

Condition Upstream of Gate		182.08	182.11

HEADWORKS INFLUENT CHANNEL

Friction Loss

Flow	4.76	mgd =	7.4 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	6.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.66	ft	
Velocity (Average)	1.48	fps	
Hydraulic Radius (Average)	0.79	ft	

Friction Loss 0.0018 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)	
1	Tee Thru Straight	4.76	7.36	0.60	6.00	3.00	1.66	0.74	1.48	0.03	0.02	
									Sum Minor Loss=		0.0153	ft

Total Energy Loss = 0.0172 ft

Upstream Condition		182.10	182.13

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	4.76	mgd =	7.4 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	4.76		
			Gate is Submerged
Submerged Condition			
Discharge Coefficient, C	0.61		
Velocity through gate, v	0.49	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	177.34	ft	
Velocity through Outlet, v	N/A	fps	

Energy Loss thru Gate, h_L 0.0101 ft

Condition Upstream of Gate		182.14	182.14

HEADWORKS INFLUENT CHANNEL

Flow Downstream	4.76	mgd
Hdwrks Channels in service	1	
Flow Upstream	4.76	mgd

Friction Loss

				Equation Ref.	HGL	EGL
Flow	4.76	mgd =	7.4 cfs			
Channel Width	3.00	ft	Reference 1S-1			
Total Channel Length	6.00	ft	Reference 1S-1			
Downstream Invert El	180.42		Reference 1S-2			
Upstream Invert El	180.42					
Slope	0.00%					
Manning Coeff, n	0.015					
Depth (Average)	1.72	ft				
Velocity (Average)	1.43	fps				
Hydraulic Radius (Average)	0.80	ft				
Friction Loss	0.0017	ft				

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	4.76	7.36	0.60	6.00	3.00	1.72	0.72	1.43	0.02	0.01
										Sum Minor Loss=	0.0143 ft

Total Energy Loss = 0.0160 ft

	HGL	EGL
Upstream Condition	182.12	182.16

ROCK TRAP { 1 }

Total Flow	4.76	mgd		
Number of online screens	1			
Flow per Screen	4.76	mgd =	7.4	cfs
Channel Flow	4.76	mgd	7.4	cfs
Channel Width	3.00	ft		
Channel & bar rack clearance	0.25	ft		Assumed
Bar Rack Width	2.5	ft		
DS Water Surface Elev	182.12	ft		
Bar Screen Invert Elevation	180.42	ft		
Downstream Water Depth	1.70	ft		
Installation Angle	60	deg		Assumed
Sine Angle	0.8660			
Bar Spacing	1.000	in		Assumed
Bar Thickness	0.313	in		Assumed
Bar Rack Efficiency	0.76			
Bar Rack Open Area	3.7469	sf		
V, velocity Clean Bar Rack	1.97	fps		
v, approach velocity	1.71	fps		
Headloss, clean	0.02	ft		
Upstream Water Depth	1.72			
Blockage	40%			
V, velocity Blocked Bar Rack	3.28	fps		
v, approach velocity	1.56	fps		
Headloss, blocked	0.18	ft	2.21	inches
Upstream Water Depth	1.89			

	HGL	EGL
W - Condition just Upstream of Bar Screen No 1	182.30	182.34

Appendix 2A-3

HYDRAULIC MODEL CALCULATIONS 6 MGD

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00   REVISION:

CHECKED : TL   BY : WME
DATE : 2/6/2019   DATE : 2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Bioreactors	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

		HGL	EGL
EGL =	**199.05**	199.05	199.05
Flow =	**6.00 mgd = 9.28 cfs**		

AREA 7   FILTER INFLUENT PUMP STATION

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	4.30	mgd
Flow to EQ Basin	1.70	mgd
Influent Flow	6.00	mgd
Filter Backwash	0.17	mgd
Upstream Flow	6.17	mgd

HWL	199.05	ASSUMED USE HWL AT ALL FLOWS	REFERENCE 7M-1
LWL	185.00		

		HGL	EGL
PS Wetwell Elevation	*Upstream Condition*	199.05	199.05

FILTER INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	6.17	mgd =	9.5 cfs	
Channel Width	8.25	ft	Reference 7M-1	Assumed, need S dwgs
Total Channel Length	18.00	ft	Reference 7M-2	Assumed, need S dwgs
Downstream Invert El	176.00		Reference 7M-3	Assumed, need S dwgs
Upstream Invert El	176.00			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	23.05	ft		
Velocity (Average)	0.05	fps		
Hydraulic Radius (Average)	3.50	ft		

Friction Loss   0.0000 ft

Other Loss

Turbulence   0.00   in   Assumed

Total Energy Loss =   0.0000 ft

	HGL	EGL
Upstream Condition	199.05	199.05

FILTER INFLUENT PUMP STATION
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow   6.2 mgd =   9.5 cfs

WSE Downstream of Weir	199.05	ft	
Weir Crest Elevation	191.25	ft	Assumed per Reference 7M-1
Downstream head, Hd	7.80	ft	
Length of Weir, L	18.00	ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation   { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation   { 7 }

K	0.01	
M	21.78	
Increment	0.10	ft
Upstream Head, Hu1	7.80	ft
F(H1)	0.00	
F'(H1)	-0.19	
Upstream Head, Hu2	7.80	ft

	Equation Ref.	HGL	EGL
Upstream WSE 199.05 ft			
Head over Weir 7.80 ft			
Condition Upstream of Weir		199.05	199.05

CLARIFIER 1&2 EFFLUENT JUNCTION
FLOW SPLIT

Downstream Flow	6.2	mgd =	9.5	cfs
No. of clarifiers	2.0			
New Flow	3.1	mgd =	4.8	cfs

20-SE SECONDARY CLARIFIER DISCHARGE PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	3.1	mgd = 4.8 cfs
Pipe Diameter, D	20	inch
Pipe Length, L	15	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.19	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	364710	
Friction factor, f	0.0162	Equivalent Hazen-Williams "C" = 143.6954
Friction Energy Loss, h_L	0.01	ft

MINOR PIPE LOSS HEADING

Flow, Q 3.1 mgd = 4.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	3.09	4.77	1.00	----	20	----	2.19	0.07	0.07
1	Mitre Bend - 45 º Deflection	3.09	4.77	0.32	20	----	2.19	----	0.07	0.02
1	Tee - Thru Straight Run	3.09	4.77	0.60	20	----	2.19	----	0.07	0.04
1	Outlet Loss - Still Water	3.09	4.77	1.00	20	----	2.19	----	0.07	0.07
									Sum =	0.22

Total Energy Loss = 0.23 ft

	HGL	EGL
Upstream Condition	199.28	199.28

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 3.1 mgd = 4.8 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	199.28	ft	
Downstream EGL	199.28	ft	
Channel Width, W	2.0	ft	Reference 5S-2
Critical. Depth, y_c	0.56	ft	
Channel Invert @ Exit	197.55	ft	Reference 5S-2

Flooded Condition

Depth Upstream of Drop	1.76	ft
Velocity Upstream of Drop	1.36	fps
Channel Exp./Bend "K"	2.00	
Energy Loss	0.06	ft
EGL Upstream of Drop	199.34	ft
HGL Upstream of Drop	199.31	ft

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

	HGL	EGL
Condition Upstream of Drop	199.31	199.34

SECONDARY CLARIFIER EFFLUENT LAUNDER

Clarifier Diameter	85	feet	Reference 5S-2
Weir Diameter	80	feet	

				Equation Ref.	HGL	EGL
Flow						
Downstream Flow	3.1 mgd =	4.8				
Split launder	2.0					
New Flow	1.5 mgd =	2.4				
Friction Loss						
Flow	3.09 mgd =	4.8 cfs				
Channel Width	2.00 ft		Reference 5S-2			
Total Channel Length	130.38 ft		Reference 5S-2			
Downstream Invert El	197.55		Reference 5S-2			
Upstream Invert El	198.37					
Slope	0.63%					
Manning Coeff, n	0.015					
Depth (Average)	1.35 ft					
Velocity (Average)	1.77 fps					
Hydraulic Radius (Average)	0.57 ft					
Friction Loss	0.0876 ft					
			Upstream Condition		199.37	199.42

SECONDARY CLARIFIER EFFLUENT WEIR
[V-NOTCH WEIR]

				Equation Ref.	HGL	EGL
Flow	3.09 mgd =	4.8 cfs				
WSE Downstream of Weir	199.37 ft					
Weir Crest Elevation	200.43 ft		Reference 5S-4			
Downstream head, Hd	-1.06 ft					
Weir Length	251.33 ft					
Distance Between Notches	6.00 in		Reference 5S-4			
Number of Notches	502					
WEIR IS FREE-DISCHARGING						
Free Discharging Weir Computation				{ 8 }		
Head on Weir, H	0.11 ft					
Upstream WSE	200.54 ft					
Submerged Weir Computation				{ 9 }		
K	NA					
M	NA					
Increment	NA ft					
Upstream Head, Hu1	NA ft					
F(H1)	NA					
F'(H1)	NA					
Upstream Head, Hu2	NA ft					
Upstream WSE	NA ft					
Head over Weir	*0.11 ft*					
			Condition Upstream of Weir		200.54	200.54

SECONDARY CLARIFIER INFLUENT BAFFLE
[V-NOTCH WEIR]

				Equation Ref.	HGL	EGL
Downstream Flow	3.09					
RAS	0.26 mgd =					
Dewatering Recycles	0.14					
Upstream Flow	3.5 mgd =	5.4 cfs				
WSE Downstream of Weir	200.54 ft					
Weir Crest Elevation	200.43 ft					
Downstream head, Hd	0.11 ft					
Weir Length	78.54 ft		Assumed			
Distance Between Notches	6.00 in		Assumed			
Number of Notches	157					
WEIR IS SUBMERGED						
Free Discharging Weir Computation				{ 8 }		
Head on Weir, H	NA ft					
Upstream WSE	NA ft					
Submerged Weir Computation				{ 9 }		
K	0.00					
M	0.00					
Increment	0.10 ft					
Upstream Head, Hu1	0.19 ft					
F(H1)	0.00					
F'(H1)	-29.28					
Upstream Head, Hu2	0.19 ft					
Upstream WSE	200.62 ft					

	Equation Ref.	HGL	EGL
Head over Weir *0.19 ft*			
Condition Upstream of Weir		200.62	200.62

SECONDARY CLARIFIER FLOCCULATING WELL

- Treat as a submerged orifice.

Flow, Q	3.49	mgd =	5.4 cfs
Downstream WSE	200.54	ft	
Flocculation Diameter	25.00	ft	
EDI Diameter	8.50	ft	
Opening Area	434	sf	
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.01	fps	
Energy Loss , h_L	0.00	ft	

Condition in Flocculating Well — HGL 200.62 — EGL 200.62

SECONDARY CLARIFIER ENERGY DISSIPATION INLET

- Treat as a submerged orifice
- Assume Upstream EGL = HGL

Flow, Q	3.49	mgd =	5.4 cfs
Downstream WSE	200.62	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.00	sf	
Total Port Area	16	sf	
Submerged Port Area	189485%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.34	fps	
Energy Loss , h_L	0.00	ft	

Condition in Influent Well — HGL 200.62 — EGL 200.62

SECONDARY CLARIFIER CENTER COLUMN OUTLETS

- Treat as a submerged orifice

Orifice Loss

Flow, Q	3.49	mgd =	5.4 cfs
Downstream WSE	200.62	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.0	sf	
Total Port Area	16	sf	
Submerged Port Area	189493%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.34	fps	
Energy Loss , h_L	0.00	ft	

Minor Losses

Flow, Q 3.49 mgd = 5.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mounding Loss	3.49	5.40	0.25	22	----	2.05	----	0.06	0.02
									Sum =	0.02

Total Energy Loss = 0.02 ft

HGL 200.64 — EGL 200.64

22"ML SECONDARY CLARIFIER CENTER COLUMN
PIPE FRICTION LOSSES (MANNING) - Full Pipe Flow Only

Flow	3.49	mgd =	5.4 cfs	**Flow + Total Recycle + RAS**
Pipe Diameter, D	22	inch		
Pipe Length, L	22	ft		
Manning Coef., n	0.015	ft		

	Equation Ref.	HGL	EGL

Velocity 2.05 fps
Hydraulic Radius 0.46 ft

Friction Energy Loss 0.03 ft

MINOR PIPE LOSS HEADING

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Bend	3.49	5.40	0.60	22	----	2.05	----	0.06	0.04

Total Minor Losses = 0.04 ft

Total Energy Loss = 0.07 ft

Clarifier center column	*Upstream Condition*	200.64	200.71

22"ML JUNCTION BOX TO CLARIFIER 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	3.5 mgd =	5.4 cfs
Pipe Diameter, D	22 inch	
Pipe Length, L	220 ft	
Absolute Roughness, ε	0.00040 ft	
Pipe velocity, v	2.05 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	375067	
Friction factor, f	0.0160	Equivalent Hazen-Williams "C" = 144.3042
Friction Energy Loss, h_L	0.12 ft	

MINOR PIPE LOSS HEADING

Flow, Q 3.5 mgd = 5.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mitre Bend - 90 º Deflection	3.49	5.40	1.27	22	----	2.05	----	0.06	0.08
1	Mitre Bend - 45 º Deflection	3.49	5.40	0.32	22	----	2.05	----	0.06	0.02
1	Entrance Loss - Flush	3.49	5.40	0.50	----	22	----	2.05	0.06	0.03
									Sum =	0.136

Total Energy Loss = 0.26 ft

	Upstream Condition	200.97	200.97

AREA 4 **MIXED LIQUOR SPLITTER BOX**

ML JUNCTION FLOW SPLIT
FLOW SPLIT

Downstream Flow	3.5 mgd =	5.4 cfs
No. of SCs Oline	2.0	
New Flow	7.0 mgd =	10.8 cfs

MIXED LIQUOR SPLITTER EFFLUENT

Friction Loss

Flow	6.98 mgd =	10.8 cfs
Channel Width	6.00 ft	Reference 4S-1
Total Channel Length	4.00 ft	Reference 4S-1
Downstream Invert El	191.00	Reference 4S-4
Upstream Invert El	191.00	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	9.97 ft	
Velocity (Average)	0.18 fps	
Hydraulic Radius (Average)	2.31 ft	
Friction Loss	0.0000 ft	

Other Loss

				Equation Ref.	HGL	EGL
Turbulence	0.00	in	Assumed			
Total Energy Loss =	0.0000	ft				
			Upstream Condition		200.97	200.97

ML SPLITTER BOX WEIR (downward opening weir gates)
[STRAIGHT EDGED SHARP CRESTED WEIR]

				Equation Ref.	HGL	EGL
Flow	7.0	mgd =	10.8 cfs			
WSE Downstream of Weir	200.97	ft				
Weir Crest Elevation	201.97	ft	Assume EL per Reference G-3 60"x24" weir gate.			
Downstream head, Hd	-1.00	ft	Low position 201.75, top of STR opening is 203.25, so can full close			
Length of Weir, L	5.00	ft				
WEIR IS FREE-DISCHARGING						
Free Discharging Weir Computation				{ 6 }		
Head on Weir, H	0.75	ft				
Upstream WSE	202.72	ft				
Submerged Weir Computation				{ 7 }		
K	NA					
M	NA					
Increment	NA	ft				
Upstream Head, Hu1	NA	ft				
F(H1)	NA					
F'(H1)	NA					
Upstream Head, Hu2	NA	ft				
Upstream WSE	NA	ft				
Head over Weir	*0.75*	*ft*				
			Condition Upstream of Weir		202.72	202.72

MIXED LIQUOR SPLITTER INFLUENT

				Equation Ref.	HGL	EGL
Friction Loss						
Flow	6.98	mgd =	10.8 cfs			
Channel Width	15.00	ft	Reference 4S-1			
Total Channel Length	4.00	ft	Reference 4S-1			
Downstream Invert El	191.00		Reference 4S-4			
Upstream Invert El	191.00					
Slope	0.00%					
Manning Coeff, n	0.015					
Depth (Average)	11.72	ft				
Velocity (Average)	0.06	fps				
Hydraulic Radius (Average)	4.57	ft				
Friction Loss	0.0000	ft				
Other Loss						
Turbulence	0.00	in	Assumed			
Total Energy Loss =	0.0000	ft				
			Upstream Condition		202.72	202.72

33"ML FROM OX DITCH TEE TO ML SPLITTER BOX
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)]

				Equation Ref.	HGL	EGL
				{ 4 }		
Flow	7.0	mgd =	10.8 cfs			
Pipe Diameter, D	33	inch				
Pipe Length, L	95	ft				
Absolute Roughness, ε	0.00040	ft				
Pipe velocity, v	1.82	fps				
Kinematic Viscosity	1.000E-05	ft^2/sec				
Reynold's Number, R	500089					
Friction factor, f	0.0149		Equivalent Hazen-Williams "C" = 146.0321			
Friction Energy Loss, h_L	0.03	ft				

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 7.0 mgd = 10.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	6.98	10.80	0.50	----	33	----	1.82	0.05	0.03
1	Tee - Thru Side	6.98	10.80	1.80	33	----	1.82	----	0.05	0.09
									Sum =	0.12

Total Energy Loss = 0.14 ft

Upstream Condition HGL 202.86 EGL 202.86

OX DITCH TEE
FLOW SPLIT

Downstream Flow 7.0 mgd = 10.8 cfs
Ox Ditch online 2
New Flow 3.5 mgd = 5.4 cfs

22"ML FROM OX DITCH 1 TEE TO OX DITCH TEE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 3.5 mgd = 5.4 cfs

Pipe Diameter, D 22 inch
Pipe Length, L 50 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 2.05 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 375067
Friction factor, f 0.0160 Equivalent Hazen-Williams "C" = 144.3042

Friction Energy Loss, h_L 0.03 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.5 mgd = 5.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Reducer	3.49	5.40	0.25	33	22	0.91	2.05	0.06	0.02
3	Mitre Bend - 90 º Deflection	3.49	5.40	1.27	22	----	2.05	----	0.06	0.25
									Sum =	0.26

Total Energy Loss = 0.29 ft

Upstream Condition HGL 203.16 EGL 203.16

AREA 4 **BIOLOGICAL REACTORS**

OXIDATION DITCH EFFLUENT WEIR (motorized weir)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 3.49 mgd = 5.4 cfs

WSE Downstream of Weir 203.16 ft
Weir Crest Elevation 206.03 ft low position: 204.96; high position: 206.03 Reference 4M-5
Downstream head, Hd -2.87 ft
Length of Weir, L 15.00 ft Reference M-13

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }
Head on Weir, H 0.23 ft
Upstream WSE 206.26 ft

Submerged Weir Computation { 7 }
K NA

			Equation Ref.	HGL	EGL
M	NA				
Increment	NA	ft			
Upstream Head, Hu1	NA	ft			
F(H1)	NA				
F'(H1)	NA				
Upstream Head, Hu2	NA	ft			
Upstream WSE	NA	ft			
Head over Weir	*0.23*	*ft*			
			Condition Upstream of Weir	206.26	206.26

AEROBIC ZONE

Friction Loss

Flow	3.49	mgd =	5.4 cfs	
Channel Width	30.25	ft		Reference 4S-1
Total Channel Length	256.00	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-4
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	13.97	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.26	ft		

Friction Loss 0.0000 ft

Other Loss

Baffles 1.00 in Assumed

Total Energy Loss = 0.0833 ft

	HGL	EGL
Upstream Condition	206.34	206.34

TRANSITION FROM ANOXIC TO AEROBIC

Friction Loss

Flow	3.49	mgd =	5.4 cfs	
Channel Width	2.50	ft	Reference 4s-4	
Total Channel Length	30.00	ft		
Downstream Invert El	192.29			
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	14.05	ft		
Velocity (Average)	0.15	fps		
Hydraulic Radius (Average)	1.15	ft		

Friction Loss 0.0001 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	3.49	5.40	1.00	5.00	8.00	14.05	0.08	0.05	0.00	0.00
1	Sudden Contraction	3.49	5.40	0.50	8.00	5.00	14.05	0.05	0.08	0.00	0.00
										Sum Minor Loss=	0.00 ft

Total Energy Loss = 0.0001 ft

	HGL	EGL
Upstream Condition	206.34	206.34

ANOXIC ZONE

Friction Loss

Flow	3.49	mgd =	5.4 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	14.05	ft		
Velocity (Average)	0.01	fps		
Hydraulic Radius (Average)	7.20	ft		

Friction Loss 0.0000 ft

Other Loss

Vertical Mixer 0.50 in Assumed

	Equation Ref.	HGL	EGL
Total Energy Loss = 0.0417 ft			
Upstream Condition		206.38	206.38

ANAEROBIC REACTOR EFFLUENT WEIR (downward opening weir gate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	3.5	mgd =	5.4 cfs	
WSE Downstream of Weir	206.38	ft		
Weir Crest Elevation	206.71	ft	INV opening: 204.96. Gate height 30-inches.	Reference 4S-7
Downstream head, Hd	-0.33	ft	Therefore: low position 204.96, high position 207.46	Reference M-14
Length of Weir, L	3.00	ft		

WEIR IS FREE-DISCHARGING

			Equation Ref.
Free Discharging Weir Computation			{ 6 }
Head on Weir, H	0.66	ft	
Upstream WSE	207.37	ft	
Submerged Weir Computation			{ 7 }
K	NA		
M	NA		
Increment	NA	ft	
Upstream Head, Hu1	NA	ft	
F(H1)	NA		
F'(H1)	NA		
Upstream Head, Hu2	NA	ft	
Upstream WSE	NA	ft	
Head over Weir	*0.66*	*ft*	

	HGL	EGL
Condition Upstream of Weir	207.37	207.37

ANAEROBIC ZONE 3

Friction Loss

Downstream Flow	3.5
Ox Ditches in Service	2
Upstream Flow	6.9820023

Friction Loss

Flow	6.98	mgd =	10.8 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	15.08	ft		
Velocity (Average)	0.02	fps		
Hydraulic Radius (Average)	7.46	ft		
Friction Loss	0.0000	ft		

Other Loss

Vertical Mixer	0.50	in	Assumed

Total Energy Loss = 0.0417 ft

	HGL	EGL
Upstream Condition	207.42	207.42

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 2 and 3)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	7.0	mgd =	10.8 cfs	
			INV opening: 204.96. Gate height 42-inches.	Reference M-14
WSE Downstream of Weir	207.42	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close	
Weir Crest Elevation	204.96	ft	ASSUMED	
Downstream head, Hd	2.46	ft		
Length of Weir, L	3.00	ft	Reference 4S-7	

WEIR IS SUBMERGED

			Equation Ref.
Free Discharging Weir Computation			{ 6 }
Head on Weir, H	NA	ft	
Upstream WSE	NA	ft	

				Equation Ref.	HGL	EGL
Submerged Weir Computation				{ 7 }		
K	1.23					
M	3.85					
Increment	0.10	ft				
Upstream Head, Hu1	2.51	ft				
F(H1)	0.00					
F'(H1)	-0.63					
Upstream Head, Hu2	2.51	ft				
Upstream WSE	207.47	ft				
Head over Weir	*2.51*	*ft*				
			Condition Upstream of Weir		207.47	207.47

ANAEROBIC ZONE 2

Friction Loss

Flow	6.98	mgd =	10.8	cfs
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	15.18	ft		
Velocity (Average)	0.02	fps		
Hydraulic Radius (Average)	7.48	ft		
Friction Loss	0.0000	ft		

Other Loss

Vertical Mixer	0.50	in	Assumed
Total Energy Loss =	0.0417	ft	

		HGL	EGL
	Upstream Condition	207.51	207.51

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 1 and 2)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	7.0	mgd =	10.8 cfs
			INV opening: 204.96. Gate height 42-inches. Reference M-14
WSE Downstream of Weir	207.51	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close
Weir Crest Elevation	204.96	ft	ASSUMED
Downstream head, Hd	2.55	ft	
Length of Weir, L	3.00	ft	Reference 4S-7

WEIR IS SUBMERGED

				Equation Ref.	HGL	EGL
Free Discharging Weir Computation				{ 6 }		
Head on Weir, H	NA	ft				
Upstream WSE	NA	ft				
Submerged Weir Computation				{ 7 }		
K	1.23					
M	4.08					
Increment	0.10	ft				
Upstream Head, Hu1	2.60	ft				
F(H1)	0.00					
F'(H1)	-0.60					
Upstream Head, Hu2	2.60	ft				
Upstream WSE	207.56	ft				
Head over Weir	*2.60*	*ft*				
			Condition Upstream of Weir		207.56	207.56

ANAEROBIC ZONE 1

Friction Loss

Flow	6.98	mgd =	10.8	cfs
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	15.27	ft		
Velocity (Average)	0.02	fps		
Hydraulic Radius (Average)	7.50	ft		
Friction Loss	0.0000	ft		

	Equation Ref.	HGL	EGL

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition HGL 207.61 EGL 207.61

ANAEROBIC REACTOR INFLUENT GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 7.0 mgd = 10.8 cfs

Diameter of Opening 2.5 ft
Sluice Gate Percent Open 100%
Discharge Coefficient, C 0.61
Velocity through gate, v 2.20 fps

Energy Loss thru Gate, h_L 0.20 ft

Condition Upstream of Gate HGL 207.81 EGL 207.81

FLOW CHANGE/SPLIT

Downstream Flow 6.98 mgd = 10.8 cfs
Anaerobic Bypass 0.00
Upstream Flow 6.98 mgd =

30" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 6.98 mgd = 10.8 cfs

Pipe Diameter, D 30 inch
Pipe Length, L 13 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 2.20 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 550098
Friction factor, f 0.0149 Equivalent Hazen-Williams "C" = 144.9817

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 7.0 mgd = 10.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Increaser	6.98	10.80	0.25	30	33	2.20	1.82	0.02	0.01
1	Outlet Loss - Still Water	6.98	10.80	1.00	33	----	1.82	----	0.05	0.05
									Sum =	0.06

Total Energy Loss = 0.06 ft

Upstream Condition HGL 207.87 EGL 207.87

33" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 6.98 mgd = 10.8 cfs

Pipe Diameter, D 33 inch
Pipe Length, L 280 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 1.82 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 500089
Friction factor, f 0.0149 Equivalent Hazen-Williams "C" = 146.0321

Friction Energy Loss, h_L 0.08 ft

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 6.98 mgd = 10.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	Mitre Bend - 90 ° Deflection	6.98	10.80	1.27	33	----	1.82	----	0.05	0.13
1	Tee - standard	6.98	10.80	1.50	33	----	1.82	----	0.05	0.08
2	Mitre Bend - 45 ° Deflection	6.98	10.80	0.32	33	----	1.82	----	0.05	0.03
1	Tee - Thru Straight	6.98	10.80	0.60	33	----	1.82	----	0.05	0.03
									Sum =	0.27

Total Energy Loss = 0.35 ft

Upstream Condition 208.22 208.22

24" PI (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Downstream Flow	6.98	mgd
RAS split	0.52	mgd
Upstream Flow	6.46	mgd = 10.0 cfs
Pipe Diameter, D	24	inch
Pipe Length, L	120	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	3.18	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	636214	
Friction factor, f	0.0151	Equivalent Hazen-Williams "C" = 142.4499

Friction Energy Loss, h_L 0.14 ft

MINOR PIPE LOSS HEADING

Flow, Q 6.46 mgd = 10.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	6.46	9.99	0.50	----	24	----	3.18	0.16	0.08
1	Mitre Bend - 90 ° Deflection	6.46	9.99	1.27	24	----	3.18	----	0.16	0.20
1	Tee - Standard	6.46	9.99	1.50	24	----	3.18	----	0.16	0.24
1	Increaser	6.46	9.99	0.25	24	33	3.18	1.68	0.11	0.03
									Sum =	0.54

Total Energy Loss = 0.68 ft

Upstream Condition 208.90 208.90

AREA 2 GRIT REMOVAL AND FINE SCREENS

24-PI INLET GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q	6.5	mgd = 10.0 cfs
Diameter of Opening	2	ft
Sluice Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	3.18	fps

Energy Loss thru Gate, h_L 0.42 ft

Condition Upstream of Gate 209.33 209.33

FINE SCREEN EFFLUENT SUMP { 4 }

Friction Loss

					Equation Ref.	HGL	EGL
Flow	6.46 mgd =	10.0 cfs					
	=	4489.7 gpm					
Channel Width	3 ft		Reference	#21A			
Channel Height	5 ft		Reference	#21B			
Equivalent Pipe Diameter, D	52 inch						
Conduit Length, L	13.66 ft						
Roughness Coefficient, C	120						
Pipe velocity, v	0.67 fps						
Total Friction Loss	0.0005 ft						

MINOR PIPE LOSS HEADING

Flow, Q 6.5 mgd = 10.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Outlet Loss - Still Water	6.46	9.99	1.00	52	----	0.67	----	0.01	0.01
1	Reducer	6.46	9.99	0.25	52	52	0.67	0.67	0.01	0.00
									Sum =	0.00862

Total Energy Loss = 0.01 ft

	HGL	EGL
Upstream Condition	209.34	209.34

FLOW CHANGE/SPLIT

Downstream Flow	6.46 mgd =	10.0 cfs
Fine Screen Bypass	3.00	
Upstream Flow	3.46 mgd =	

FINE SCREEN EFFLUENT WEIR (redwood weir plate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	3.5 mgd =	5.4 cfs
WSE Downstream of Weir	209.34 ft	
Weir Crest Elevation	208.17 ft	
Downstream head, Hd	1.17 ft	
Length of Weir, L	5.00 ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation			{ 6 }
Head on Weir, H	NA ft	11.52	
Upstream WSE	NA ft		

Submerged Weir Computation			{ 7 }
K	0.05		
M	1.26		
Increment	0.10	ft	
Upstream Head, Hu1	1.19	ft	
F(H1)	0.00		
F'(H1)	-1.32		
Upstream Head, Hu2	1.19	ft	
Upstream WSE	209.36	ft	

Head over Weir *1.19 ft*

	HGL	EGL
Condition Upstream of Weir	209.36	209.36

FINE SCREEN

Flow	3.5 mgd =	5.4 cfs
Blinding	50%	
Maximum Headloss	0.50 ft	

	HGL	EGL
Condition Upstream of Bar Screen	209.86	209.86

FINE SCREEN INFLUENT

Friction Loss

Downstream Flow	3.5
Fine Screen Bypass	3
Upstream Flow	6.5

Friction Loss

Flow	6.46 mgd =	10.0 cfs
Channel Width	5.00 ft	Reference 2S-1
Total Channel Length	25.50 ft	

			Equation Ref.	HGL	EGL
Downstream Invert El	207.67				
Upstream Invert El	207.67				
Slope	0.00%				
Manning Coeff, n	0.015				
Depth (Average)	2.19	ft			
Velocity (Average)	0.91	fps			
Hydraulic Radius (Average)	1.17	ft			

Friction Loss 0.0018 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee- Thru Straight	6.46	9.99	0.60	5.00	----	2.19	0.91	----	0.01	0.01
									Sum Minor Loss=		0.01 ft

Total Energy Loss = 0.0095 ft

	HGL	EGL
Upstream Condition	209.85	209.87

GRIT CHAMBER EFFLUENT WEIR
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 6.5 mgd = 10.0 cfs

WSE Downstream of Weir	209.85	ft	
Weir Crest Elevation	209.37	ft	Reference 2M-2
Downstream head, Hd	0.48	ft	
Length of Weir, L	5.50	ft	Reference 2S-1

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }
Head on Weir, H NA ft
Upstream WSE NA ft

Submerged Weir Computation { 7 }

K	0.21	
M	0.34	
Increment	0.10	ft
Upstream Head, Hu1	0.79	ft
F(H1)	0.00	
F'(H1)	-3.48	
Upstream Head, Hu2	0.79	ft
Upstream WSE	210.16	ft

Head over Weir 0.79 ft

	HGL	EGL
Condition Upstream of Weir	210.16	210.16

GRIT CHAMBER

Flow 6.46 mgd = 10.0 cfs

Maximum Headloss 2.00 in Assumed

	HGL	EGL
Condition Upstream of Bar Screen	210.33	210.33

PIPE FROM HEADWORKS TO GRIT CHAMBER
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 6.5 mgd = 10.0 cfs

Pipe Diameter, D	20	inch
Pipe Length, L	80	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	4.58	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	763457	
Friction factor, f	0.0153	Equivalent Hazen-Williams "C" = 139.5988

Friction Energy Loss, h_L 0.24 ft

MINOR PIPE LOSS HEADING

Flow, Q 6.5 mgd = 10.0 cfs

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00 **REVISION:**

CHECKED : TL **DATE :** 2/6/2019 **BY :** WME **DATE :** 2/6/2017

Equation Ref.	HGL	EGL

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	1.62	2.50	1.00	----	12	----	3.18	0.16	0.16
1	Entrance Loss - Pipe Ext.	1.62	2.50	1.00	----	12	----	3.18	0.16	0.16
1	Entrance Loss - Pipe Ext.	1.62	2.50	1.00	----	12	----	3.18	0.16	0.16
1	Entrance Loss - Pipe Ext.	1.62	2.50	1.00	----	12	----	3.18	0.16	0.16
1	Tee - Thru Side Outlet	6.46	9.99	1.80	20	----	4.58	----	0.33	0.59
1	Reducer	6.46	9.99	0.25	20	16	4.58	7.16	0.80	0.20
1	Mag Meter	6.46	9.99	0	16	----	7.16	----	0.80	0.00
1	Increaser	6.46	9.99	0.25	16	16	7.16	7.16	0.00	0.00
1	Plug Valve (Open)	6.46	9.99	0.77	16	----	7.16	----	0.80	0.61
1	Increaser	6.46	9.99	0.25	16	20	7.16	4.58	0.47	0.12
1	Mitre Bend - 90 º Deflection	6.46	9.99	1.27	20	----	4.58	----	0.33	0.41
1	Mitre Bend - 90 º Deflection	6.46	9.99	1.27	20	----	4.58	----	0.33	0.41
1	Mitre Bend - 90 º Deflection	6.46	9.99	1.27	20	----	4.58	----	0.33	0.41
1	Mitre Bend - 90 º Deflection	6.46	9.99	1.27	20	----	4.58	----	0.33	0.41
1	Mitre Bend - 90 º Deflection	6.46	9.99	1.27	20	----	4.58	----	0.33	0.41
1	Outlet Loss - Still Water	6.46	9.99	1.00	20	----	4.58	----	0.33	0.33
									Sum =	4.53

Total Energy Loss = 4.77 ft

Influent Wet Well	*Upstream Condition*	214.93	214.92955

AREA 1 **HEADWORKS AND INFLUENT PUMP STATION**

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	6.46	mgd
Wetwells in service	2	
Flow Upstream	3.23	mgd
HHWL	182.92	
HWL	181.89	
LWL	180.42	
LLWL	178.42	

ASSUMED USE HWL AT ALL FLOWS REFERENCE 1M-3

PS Wetwell Elevation	*Upstream Condition*	181.89	181.89

INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	3.23	mgd = 5.0 cfs
Channel Width	13.83	ft
Total Channel Length	15.00	ft
Downstream Invert El	174.42	
Upstream Invert El	174.42	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	7.47	ft
Velocity (Average)	0.05	fps
Hydraulic Radius (Average)	3.59	ft

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Reducer	3.23	5.00	0.25	6.00	13.83	7.47	0.11	0.05	0.00	0.00
									Sum Minor Loss=		0.0000 ft

Total Energy Loss = 0.0000 ft

	Upstream Condition	181.89	181.89

IPS WETWELL INLET GATE

Flow per Gate	3.23	mgd = 5.0 cfs
Gate Width	3.5	ft
Height of Gate	5.0	ft
Invert Elevation of Gate	180.42	

Gate is Not Submerged

Submerged Condition

Discharge Coefficient, C N/A

		Equation Ref.	HGL	EGL

Velocity through gate, v | N/A fps

Not Submerged Condition
K | 1.7 — Modeled as gate frame (0.2) and entrance and exit (1.5)
Water Depth thru Gate | 1.47 ft
Velocity through Outlet, v | 0.97 fps

Energy Loss thru Gate, h_L 0.0249 ft

Condition Upstream of Gate — HGL 181.90 — EGL 181.91

HEADWORKS EFFLUENT CHANNEL

Flow Downstream	3.23	mgd
Wetwells in service	2	
Flow Upstream	6.46	mgd

Friction Loss

Flow	3.23	mgd =	5.0 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	4.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.48	ft	
Velocity (Average)	1.13	fps	
Hydraulic Radius (Average)	0.75	ft	

Friction Loss 0.0008 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Side	3.23	5.00	1.80	6.00	3.00	1.48	0.56	1.13	0.01	0.03
										Sum Minor Loss=	0.0265 ft

Total Energy Loss = 0.0273 ft

Upstream Condition — HGL 181.92 — EGL 181.94

CHANNEL GRINDER EFFLUENT GATE

Flow per Gate | 6.46 mgd = 10.0 cfs

Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		

Gate is Not Submerged

Submerged Condition
Discharge Coefficient, C | N/A
Velocity through gate, v | N/A fps

Not Submerged Condition
K | 0.2 — Modeled as gate frame (0.2)
Water Depth thru Gate | 1.50 ft
Velocity through Outlet, v | 2.22 fps

Energy Loss thru Gate, h_L 0.0153 ft

Condition Upstream of Gate — HGL 181.88 — EGL 181.96

CHANNEL GRINDER

Friction Loss

Flow	6.46	mgd =	10.0 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	16.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.46	ft	
Velocity (Average)	2.28	fps	
Hydraulic Radius (Average)	0.74	ft	

Friction Loss 0.0126 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
3	45 degree bend	6.46	9.99	0.25	6.00	6.46	1.46	1.14	1.06	0.00	0.00
										Sum Minor Loss=	-0.0021 ft

	Equation Ref.	HGL	EGL

Other

Grinder 2.00 in Assumed

Total Energy Loss = 0.1772 ft

Upstream Condition		182.05	182.13

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	6.46	mgd =	10.0 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.63	ft	
Velocity through Outlet, v	2.04	fps	
Energy Loss thru Gate, h_L	0.0129	ft	

Condition Upstream of Gate		182.08	182.15

HEADWORKS INFLUENT CHANNEL

Friction Loss

Flow	6.46	mgd =	10.0 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	6.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.66	ft	
Velocity (Average)	2.00	fps	
Hydraulic Radius (Average)	0.79	ft	
Friction Loss	0.0034	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	6.46	9.99	0.60	6.00	3.00	1.66	1.00	2.00	0.05	0.03
									Sum Minor Loss=		0.0280 ft

Total Energy Loss = 0.0314 ft

Upstream Condition		182.12	182.18

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	6.46	mgd =	10.0 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	6.46		
			Gate is Submerged
Submerged Condition			
Discharge Coefficient, C	0.61		
Velocity through gate, v	0.67	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	175.66	ft	
Velocity through Outlet, v	N/A	fps	
Energy Loss thru Gate, h_L	0.0185	ft	

Condition Upstream of Gate		182.19	182.20

HEADWORKS INFLUENT CHANNEL

Flow Downstream	6.46	mgd
Hdwrks Channels in service	1	
Flow Upstream	6.46	mgd

Friction Loss

					Equation Ref.	HGL	EGL
Flow	6.46	mgd =	10.0	cfs			
Channel Width	3.00	ft		Reference 1S-1			
Total Channel Length	6.00	ft		Reference 1S-1			
Downstream Invert El	180.42			Reference 1S-2			
Upstream Invert El	180.42						
Slope	0.00%						
Manning Coeff, n	0.015						
Depth (Average)	1.77	ft					
Velocity (Average)	1.88	fps					
Hydraulic Radius (Average)	0.81	ft					
Friction Loss	0.0029	ft					

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	6.46	9.99	0.60	6.00	3.00	1.77	0.94	1.88	0.04	0.02
									Sum Minor Loss=		0.0247 ft

Total Energy Loss = 0.0276 ft

	HGL	EGL
Upstream Condition	182.17	182.23

ROCK TRAP { 1 }

Total Flow	6.46	mgd		
Number of online screens	1			
Flow per Screen	6.46	mgd =	10.0	cfs
Channel Flow	6.46	mgd	10.0	cfs
Channel Width	3.00	ft		
Channel & bar rack clearance	0.25	ft		Assumed
Bar Rack Width	2.5	ft		
DS Water Surface Elev	182.17	ft		
Bar Screen Invert Elevation	180.42	ft		
Downstream Water Depth	1.75	ft		
Installation Angle	60	deg		Assumed
Sine Angle	0.8660			
Bar Spacing	1.000	in		Assumed
Bar Thickness	0.313	in		Assumed
Bar Rack Efficiency	0.76			
Bar Rack Open Area	3.8494	sf		
V, velocity Clean Bar Rack	2.60	fps		
v, approach velocity	2.23	fps		
Headloss, clean	0.04	ft		
Upstream Water Depth	1.79			
Blockage	40%			
V, velocity Blocked Bar Rack	4.33	fps		
v, approach velocity	1.92	fps		
Headloss, blocked	0.33	ft	4.00	inches
Upstream Water Depth	2.08			

	HGL	EGL
W - Condition just Upstream of Bar Screen No 1	182.50	182.56

Appendix 2A-4

HYDRAULIC MODEL CALCULATIONS 9 MGD

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

Hydraulix® by carollo

JOB # : 101321A00 **REVISION:**

CHECKED : TL **DATE :** 2/6/2019 **BY :** WME **DATE :** 2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Bioreactors	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

EGL = 199.05

Flow = 9.00 mgd = 13.92 cfs

HGL 199.05 EGL 199.05

AREA 7 **BEGIN:** **FILTER INFLUENT PUMP STATION**

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	4.30	mgd
Flow to EQ Basin	0.00	mgd
Influent Flow	9.00	mgd
Filter Backwash	0.17	mgd
Upstream Flow	9.17	mgd

HWL 199.05 ASSUMED USE HWL AT ALL FLOWS REFERENCE 7M-1

LWL 185.00

PS Wetwell Elevation *Upstream Condition* HGL 199.05 EGL 199.05

FILTER INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	9.17	mgd = 14.2 cfs		
Channel Width	8.25	ft	Reference 7M-1	Assumed, need S dwgs
Total Channel Length	18.00	ft	Reference 7M-2	Assumed, need S dwgs
Downstream Invert El	176.00		Reference 7M-3	Assumed, need S dwgs
Upstream Invert El	176.00			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	23.05	ft		
Velocity (Average)	0.07	fps		
Hydraulic Radius (Average)	3.50	ft		

Friction Loss 0.0000 ft

Other Loss

Turbulence 0.00 in Assumed

Total Energy Loss = 0.0000 ft

Upstream Condition HGL 199.05 EGL 199.05

FILTER INFLUENT PUMP STATION
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 9.2 mgd = 14.2 cfs

WSE Downstream of Weir	199.05	ft	
Weir Crest Elevation	191.25	ft	Assumed per Reference 7M-1
Downstream head, Hd	7.80	ft	
Length of Weir, L	18.00	ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation { 7 }

K	0.02	
M	21.78	
Increment	0.10	ft
Upstream Head, Hu1	7.80	ft
F(H1)	0.00	
F'(H1)	-0.19	
Upstream Head, Hu2	7.80	ft
Upstream WSE	199.05	ft

	Equation Ref.	HGL	EGL
Head over Weir 7.80 ft			
Condition Upstream of Weir		199.05	199.05

END: FILTER INFLUENT PUMP STATION
BEGIN: YARD

CLARIFIER 1&2 EFFLUENT JUNCTION
FLOW SPLIT

Downstream Flow	9.2	mgd =	14.2	cfs
No. of clarifiers	2.0			
New Flow	4.6	mgd =	7.1	cfs

20-SE SECONDARY CLARIFIER DISCHARGE PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	4.6	mgd = 7.1 cfs	
Pipe Diameter, D	20	inch	
Pipe Length, L	15	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	3.25	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	541983		
Friction factor, f	0.0157		Equivalent Hazen-Williams "C" = 141.7315
Friction Energy Loss, h_L	0.02	ft	

MINOR PIPE LOSS HEADING

Flow, Q 4.6 mgd = 7.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	4.59	7.09	1.00	----	20	----	3.25	0.16	0.16
1	Mitre Bend - 45 º Deflection	4.59	7.09	0.32	20	----	3.25	----	0.16	0.05
1	Tee - Thru Straight Run	4.59	7.09	0.60	20	----	3.25	----	0.16	0.10
1	Outlet Loss - Still Water	4.59	7.09	1.00	20	----	3.25	----	0.16	0.16
									Sum =	0.48

Total Energy Loss = 0.50 ft

	HGL	EGL
Upstream Condition	199.55	199.55

END: YARD
BEGIN: SECONDARY CLARIFIERS

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 4.6 mgd = 7.1 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	199.55	ft	
Downstream EGL	199.55	ft	
Channel Width, W	2.0	ft	Reference 5S-2
Critical. Depth, y_c	0.73	ft	
Channel Invert @ Exit	197.55	ft	Reference 5S-2

Flooded Condition

Depth Upstream of Drop	2.05	ft
Velocity Upstream of Drop	1.73	fps
Channel Exp./Bend "K"	2.00	
Energy Loss	0.09	ft
EGL Upstream of Drop	199.65	ft
HGL Upstream of Drop	199.60	ft

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

	HGL	EGL
Condition Upstream of Drop	199.60	199.65

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

Hydraulix BY carollo

JOB # : 101321A00 REVISION:

CHECKED : TL DATE : 2/6/2019 BY : WME DATE : 2/6/2017

					Equation Ref.	HGL	EGL
SECONDARY CLARIFIER EFFLUENT LAUNDER			Clarifier Diameter 85 feet		Reference 5S-2		
			Weir Diameter 80 feet				
Flow							
Downstream Flow	4.6	mgd =	7.1				
Split launder	2.0						
New Flow	2.3	mgd =	3.5				
Friction Loss							
Flow	4.59	mgd =	7.1 cfs				
Channel Width	2.00	ft	Reference 5S-2				
Total Channel Length	130.38	ft	Reference 5S-2				
Downstream Invert El	197.55		Reference 5S-2				
Upstream Invert El	198.37						
Slope	0.63%						
Manning Coeff, n	0.015						
Depth (Average)	1.64	ft					
Velocity (Average)	2.16	fps					
Hydraulic Radius (Average)	0.62	ft					
Friction Loss	0.1174	ft					
				Upstream Condition		199.69	199.76
SECONDARY CLARIFIER EFFLUENT WEIR							
[V-NOTCH WEIR]							
Flow	4.59	mgd =	7.1 cfs				
WSE Downstream of Weir	199.69	ft					
Weir Crest Elevation	200.43	ft	Reference 5S-4				
Downstream head, Hd	-0.74	ft					
Weir Length	251.33	ft					
Distance Between Notches	6.00	in	Reference 5S-4				
Number of Notches	502						
WEIR IS FREE-DISCHARGING							
Free Discharging Weir Computation					{ 8 }		
Head on Weir, H	0.13	ft					
Upstream WSE	200.56	ft					
Submerged Weir Computation					{ 9 }		
K	NA						
M	NA						
Increment	NA	ft					
Upstream Head, Hu1	NA	ft					
F(H1)	NA						
F'(H1)	NA						
Upstream Head, Hu2	NA	ft					
Upstream WSE	NA	ft					
Head over Weir	*0.13*	*ft*					
				Condition Upstream of Weir		200.56	200.56
SECONDARY CLARIFIER INFLUENT BAFFLE							
[V-NOTCH WEIR]							
Downstream Flow	4.59						
RAS	0.26	mgd =					
Dewatering Recycles	0.14						
Upstream Flow	5.0	mgd =	7.7 cfs				
WSE Downstream of Weir	200.56	ft					
Weir Crest Elevation	200.43	ft					
Downstream head, Hd	0.13	ft					
Weir Length	78.54	ft	Assumed				
Distance Between Notches	6.00	in	Assumed				
Number of Notches	157						
WEIR IS SUBMERGED							
Free Discharging Weir Computation					{ 8 }		
Head on Weir, H	NA	ft					
Upstream WSE	NA	ft					
Submerged Weir Computation					{ 9 }		
K	0.00						
M	0.01						
Increment	0.10	ft					
Upstream Head, Hu1	0.22	ft					
F(H1)	0.00						
F'(H1)	-25.18						

			Equation Ref.	HGL	EGL
Upstream Head, Hu2	0.22	ft			
Upstream WSE	200.65	ft			
Head over Weir	*0.22*	*ft*			
		Condition Upstream of Weir		200.65	200.65

SECONDARY CLARIFIER FLOCCULATING WELL

- Treat as a submerged orifice.

Flow, Q	4.99	mgd =	7.7 cfs
Downstream WSE	200.56	ft	
Flocculation Diameter	25.00	ft	
EDI Diameter	8.50	ft	
Opening Area	434	sf	
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.02	fps	
Energy Loss , h_L	0.00	ft	

	HGL	EGL
Condition in Flocculating Well	200.65	200.65

SECONDARY CLARIFIER ENERGY DISSIPATION INLET

- Treat as a submerged orifice
- Assume Upstream EGL = HGL

Flow, Q	4.99	mgd =	7.7 cfs
Downstream WSE	200.65	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.00	sf	
Total Port Area	16	sf	
Submerged Port Area	189532%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.48	fps	
Energy Loss , h_L	0.01	ft	

	HGL	EGL
Condition in Influent Well	200.66	200.66

SECONDARY CLARIFIER CENTER COLUMN OUTLETS

- Treat as a submerged orifice

Orifice Loss

Flow, Q	4.99	mgd =	7.7 cfs
Downstream WSE	200.66	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.0	sf	
Total Port Area	16	sf	
Submerged Port Area	189547%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.48	fps	
Energy Loss , h_L	0.01	ft	

Minor Losses

Flow, Q 4.99 mgd = 7.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mounding Loss	4.99	7.72	0.25	22	----	2.92	----	0.13	0.03
									Sum =	0.03

Total Energy Loss = 0.04 ft

	HGL	EGL
	200.70	200.70

22"ML SECONDARY CLARIFIER CENTER COLUMN
PIPE FRICTION LOSSES (MANNING) - Full Pipe Flow Only

Flow	4.99	mgd =	7.7 cfs	**Flow + Total Recycle + RAS**
Pipe Diameter, D	22	inch		
Pipe Length, L	22	ft		

	Equation Ref.	HGL	EGL

Manning Coef., n 0.015 ft

Velocity 2.92 fps
Hydraulic Radius 0.46 ft

Friction Energy Loss 0.05 ft

MINOR PIPE LOSS HEADING

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 ° Bend	4.99	7.72	0.60	22	----	2.92	----	0.13	0.08

Total Minor Losses = 0.08 ft

Total Energy Loss = 0.13 ft

Clarifier center column	*Upstream Condition*	HGL 200.70	EGL 200.83

END: SECONDARY CLARIFIERS
BEGIN: YARD

22"ML JUNCTION BOX TO CLARIFIER 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 5.0 mgd = 7.7 cfs

Pipe Diameter, D 22 inch
Pipe Length, L 220 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 2.92 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 536225
Friction factor, f 0.0155 Equivalent Hazen-Williams "C" = 142.633

Friction Energy Loss, h_L 0.25 ft

MINOR PIPE LOSS HEADING

Flow, Q 5.0 mgd = 7.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mitre Bend - 90 ° Deflection	4.99	7.72	1.27	22	----	2.92	----	0.13	0.17
1	Mitre Bend - 45 ° Deflection	4.99	7.72	0.32	22	----	2.92	----	0.13	0.04
1	Entrance Loss - Flush	4.99	7.72	0.50	----	22	----	2.92	0.13	0.07
									Sum =	0.277

Total Energy Loss = 0.52 ft

	Upstream Condition	HGL 201.36	EGL 201.36

END: YARD
AREA 4 BEGIN: MIXED LIQUOR SPLITTER BOX

ML JUNCTION FLOW SPLIT
FLOW SPLIT

Downstream Flow 5.0 mgd = 7.7 cfs
No. of SCs Oline 2.0
New Flow 10.0 mgd = 15.4 cfs

MIXED LIQUOR SPLITTER EFFLUENT

Friction Loss

Flow	9.98	mgd =	15.4 cfs
Channel Width	6.00	ft	Reference 4S-1
Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	10.36	ft	
Velocity (Average)	0.25	fps	
Hydraulic Radius (Average)	2.33	ft	

Friction Loss 0.0000 ft

	Equation Ref.	HGL	EGL

Other Loss

Turbulence	0.00	in	Assumed
Total Energy Loss =	0.0000 ft		

		HGL	EGL
Upstream Condition		201.36	201.36

ML SPLITTER BOX WEIR (downward opening weir gates)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	10.0	mgd =	15.4 cfs
WSE Downstream of Weir	201.36	ft	
Weir Crest Elevation	201.97	ft	Assume EL per Reference G-3 60"x24" weir gate.
Downstream head, Hd	-0.61	ft	Low position 201.75, top of STR opening is 203.25, so can full close
Length of Weir, L	5.00	ft	

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.95	ft
Upstream WSE	202.92	ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir 0.95 ft

		HGL	EGL
Condition Upstream of Weir		202.92	202.92

MIXED LIQUOR SPLITTER INFLUENT

Friction Loss

Flow	9.98	mgd =	15.4 cfs
Channel Width	15.00	ft	Reference 4S-1
Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	11.92	ft	
Velocity (Average)	0.09	fps	
Hydraulic Radius (Average)	4.60	ft	
Friction Loss	0.0000	ft	

Other Loss

Turbulence	0.00	in	Assumed
Total Energy Loss =	0.0000 ft		

		HGL	EGL
Upstream Condition		202.92	202.92

END: MIXED LIQUOR SPLITTER BOX
BEGIN: YARD

33"ML FROM OX DITCH TEE TO ML SPLITTER BOX
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	10.0	mgd =	15.4 cfs
Pipe Diameter, D	33	inch	
Pipe Length, L	95	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	2.60	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	714966		
Friction factor, f	0.0144		Equivalent Hazen-Williams "C" = 144.4593
Friction Energy Loss, h_L	0.05	ft	

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 10.0 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	9.98	15.44	0.50	----	33	----	2.60	0.10	0.05
1	Tee - Thru Side	9.98	15.44	1.80	33	----	2.60	----	0.10	0.19
									Sum =	0.24

Total Energy Loss = 0.29 ft

	Equation Ref.	HGL	EGL
Upstream Condition		203.21	203.21

OX DITCH TEE
FLOW SPLIT

Downstream Flow	10.0 mgd =	15.4 cfs
Ox Ditch online	2	
New Flow	5.0 mgd =	7.7 cfs

22"ML FROM OX DITCH 1 TEE TO OX DITCH TEE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 5.0 mgd = 7.7 cfs

Pipe Diameter, D	22	inch
Pipe Length, L	50	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.92	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	536225	
Friction factor, f	0.0155	Equivalent Hazen-Williams "C" = 142.633

Friction Energy Loss, h_L 0.06 ft

MINOR PIPE LOSS HEADING

Flow, Q 5.0 mgd = 7.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Reducer	4.99	7.72	0.25	33	22	1.30	2.92	0.13	0.03
3	Mitre Bend - 90 ° Deflection	4.99	7.72	1.27	22	----	2.92	----	0.13	0.50
									Sum =	0.54

Total Energy Loss = 0.59 ft

	Equation Ref.	HGL	EGL
Upstream Condition		203.81	203.81

END: YARD

AREA 4 BEGIN: BIOLOGICAL REACTORS

OXIDATION DITCH EFFLUENT WEIR (motorized weir)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 4.99 mgd = 7.7 cfs

WSE Downstream of Weir	203.81	ft	
Weir Crest Elevation	206.03	ft	low position: 204.96; high position: 206.03 — Reference 4M-5
Downstream head, Hd	-2.22	ft	
Length of Weir, L	15.00	ft	Reference M-13

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.29	ft
Upstream WSE	206.32	ft

		Equation Ref.	HGL	EGL

Submerged Weir Computation {7}

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.29 ft*

Condition Upstream of Weir | HGL 206.32 | EGL 206.32

AEROBIC ZONE

Friction Loss

Flow	4.99	mgd =	7.7 cfs
Channel Width	30.25	ft	Reference 4S-1
Total Channel Length	256.00	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-4
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	14.03	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	7.28	ft	

Friction Loss 0.0000 ft

Other Loss

Baffles 1.00 in Assumed

Total Energy Loss = 0.0833 ft

Upstream Condition | HGL 206.40 | EGL 206.40

TRANSITION FROM ANOXIC TO AEROBIC

Friction Loss

Flow	4.99	mgd =	7.7 cfs
Channel Width	2.50	ft	Reference 4s-4
Total Channel Length	30.00	ft	
Downstream Invert El	192.29		
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	14.11	ft	
Velocity (Average)	0.22	fps	
Hydraulic Radius (Average)	1.15	ft	

Friction Loss 0.0001 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	4.99	7.72	1.00	5.00	8.00	14.11	0.11	0.07	0.00	0.00
1	Sudden Contraction	4.99	7.72	0.50	8.00	5.00	14.11	0.07	0.11	0.00	0.00
									Sum Minor Loss=		0.00 ft

Total Energy Loss = 0.0002 ft

Upstream Condition | HGL 206.40 | EGL 206.40

ANOXIC ZONE

Friction Loss

Flow	4.99	mgd =	7.7 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	14.11	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	7.21	ft	

Friction Loss 0.0000 ft

Other Loss

				Equation Ref.	HGL	EGL
Vertical Mixer	0.50	in	Assumed			
Total Energy Loss =	0.0417 ft					
			Upstream Condition		206.44	206.44

ANAERObIC REACTOR EFFLUENT WEIR (downward opening weir gate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	5.0	mgd =	7.7 cfs		
WSE Downstream of Weir	206.44	ft			
Weir Crest Elevation	206.71	ft	INV opening: 204.96. Gate height 30-inches.	Reference 4S-7	
Downstream head, Hd	-0.27	ft	Therefore: low position 204.96, high position 207.46	Reference M-14	
Length of Weir, L	3.00	ft			

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation			{ 6 }
Head on Weir, H	0.84	ft	
Upstream WSE	207.55	ft	
Submerged Weir Computation			{ 7 }
K	NA		
M	NA		
Increment	NA	ft	
Upstream Head, Hu1	NA	ft	
F(H1)	NA		
F'(H1)	NA		
Upstream Head, Hu2	NA	ft	
Upstream WSE	NA	ft	
Head over Weir	*0.84*	*ft*	

Condition Upstream of Weir	207.55	207.55

ANAEROBIC ZONE 3

Friction Loss

Downstream Flow	5.0
Ox Ditches in Service	2
Upstream Flow	9.9820023

Friction Loss

Flow	9.98	mgd =	15.4 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	15.26	ft		
Velocity (Average)	0.03	fps		
Hydraulic Radius (Average)	7.50	ft		
Friction Loss	0.0000 ft			

Other Loss

Vertical Mixer	0.50	in	Assumed		
Total Energy Loss =	0.0417 ft				
			Upstream Condition	207.59	207.59

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 2 and 3)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	10.0	mgd =	15.4 cfs	
			INV opening: 204.96. Gate height 42-inches.	Reference M-14
WSE Downstream of Weir	207.59	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close	
Weir Crest Elevation	204.96	ft	ASSUMED	
Downstream head, Hd	2.63	ft		
Length of Weir, L	3.00	ft	Reference 4S-7	

WEIR IS SUBMERGED

Free Discharging Weir Computation			{ 6 }
Head on Weir, H	NA	ft	

				Equation Ref.	HGL	EGL
Upstream WSE	NA ft					
Submerged Weir Computation				{ 7 }		
K	3.10					
M	4.27					
Increment	0.10	ft				
Upstream Head, Hu1	2.75	ft				
F(H1)	0.00					
F'(H1)	-0.60					
Upstream Head, Hu2	2.75	ft				
Upstream WSE	207.71	ft				
Head over Weir	*2.75*	*ft*				
			Condition Upstream of Weir		207.71	207.71

ANAEROBIC ZONE 2

Friction Loss

Flow	9.98	mgd =	15.4 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	15.42	ft		
Velocity (Average)	0.03	fps		
Hydraulic Radius (Average)	7.54	ft		
Friction Loss	0.0000	ft		

Other Loss

Vertical Mixer	0.50	in	Assumed
Total Energy Loss =	0.0417	ft	

	HGL	EGL
Upstream Condition	207.75	207.75

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 1 and 2)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	10.0	mgd =	15.4 cfs
			INV opening: 204.96. Gate height 42-inches. Reference M-14
WSE Downstream of Weir	207.75	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close
Weir Crest Elevation	204.96	ft	ASSUMED
Downstream head, Hd	2.79	ft	
Length of Weir, L	3.00	ft	Reference 4S-7

WEIR IS SUBMERGED

			Equation Ref.	HGL	EGL
Free Discharging Weir Computation			{ 6 }		
Head on Weir, H	NA ft				
Upstream WSE	NA ft				
Submerged Weir Computation			{ 7 }		
K	3.10				
M	4.65				
Increment	0.10	ft			
Upstream Head, Hu1	2.88	ft			
F(H1)	0.00				
F'(H1)	-0.56				
Upstream Head, Hu2	2.88	ft			
Upstream WSE	207.84	ft			
Head over Weir	*2.88*	*ft*			
Condition Upstream of Weir				207.84	207.84

ANAEROBIC ZONE 1

Friction Loss

Flow	9.98	mgd =	15.4 cfs	
Channel Width	29.50	ft		Reference 4S-1
Total Channel Length	25.50	ft		Reference 4S-1
Downstream Invert El	192.29			Reference 4S-7
Upstream Invert El	192.29			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	15.55	ft		
Velocity (Average)	0.03	fps		
Hydraulic Radius (Average)	7.57	ft		

	Equation Ref.	HGL	EGL

Friction Loss 0.0000 ft

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition		207.89	207.89

ANAEROBIC REACTOR INFLUENT GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 10.0 mgd = 15.4 cfs

Diameter of Opening 2.5 ft
Sluice Gate Percent Open 100%
Discharge Coefficient, C 0.61
Velocity through gate, v 3.15 fps

Energy Loss thru Gate, h_L 0.41 ft

Condition Upstream of Gate	208.30	208.30

END: BIOLOGICAL REACTORS
END: YARD

FLOW CHANGE/SPLIT

Downstream Flow 9.98 mgd = 15.4 cfs
Anaerobic Bypass 0.00
Upstream Flow 9.98 mgd =

30" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 9.98 mgd = 15.4 cfs

Pipe Diameter, D 30 inch
Pipe Length, L 13 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 3.15 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 786463
Friction factor, f 0.0145 Equivalent Hazen-Williams "C" = 143.1717

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 10.0 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Increaser	9.98	15.44	0.25	30	33	3.15	2.60	0.05	0.01
1	Outlet Loss - Still Water	9.98	15.44	1.00	33	----	2.60	----	0.10	0.10
									Sum =	0.12

Total Energy Loss = 0.13 ft

Upstream Condition		208.43	208.43

33" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 9.98 mgd = 15.4 cfs

Pipe Diameter, D 33 inch
Pipe Length, L 280 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 2.60 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 714966
Friction factor, f 0.0144 Equivalent Hazen-Williams "C" = 144.4593

	Equation Ref.	HGL	EGL

Friction Energy Loss, h_L 0.15 ft

MINOR PIPE LOSS HEADING

Flow, Q 9.98 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	Mitre Bend - 90 º Deflection	9.98	15.44	1.27	33	----	2.60	----	0.10	0.27
1	Tee - standard	9.98	15.44	1.50	33	----	2.60	----	0.10	0.16
2	Mitre Bend - 45 º Deflection	9.98	15.44	0.32	33	----	2.60	----	0.10	0.07
1	Tee - Thru Straight	9.98	15.44	0.60	33	----	2.60	----	0.10	0.06
									Sum =	0.55

Total Energy Loss = 0.71 ft

Upstream Condition HGL 209.13 EGL 209.13

GRIT CHAMBER AND FINE SCREEN BYPASS

Downstream Flow 9.98 mgd
Bypass 4.00 mgd
Upstream Flow 5.98 mgd =

24" PI (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Downstream Flow 5.98 mgd
RAS split 0.52 mgd
Upstream Flow 9.46 mgd = 14.6 cfs

Pipe Diameter, D 24 inch
Pipe Length, L 120 ft
Absolute Roughness, ε 0.00040 ft
Pipe velocity, v 4.66 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 931669
Friction factor, f 0.0147 Equivalent Hazen-Williams "C" = 140.0577

Friction Energy Loss, h_L 0.30 ft

MINOR PIPE LOSS HEADING

Flow, Q 5.98 mgd = 9.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	5.98	9.25	0.50	----	24	----	2.95	0.13	0.07
1	Mitre Bend - 90 º Deflection	5.98	9.25	1.27	24	----	2.95	----	0.13	0.17
1	Tee - Standard	5.98	9.25	1.50	24	----	2.95	----	0.13	0.20
1	Increaser	5.98	9.25	0.25	24	33	2.95	1.56	0.10	0.02
									Sum =	0.46

Total Energy Loss = 0.76 ft

Upstream Condition HGL 209.90 EGL 209.90

END: YARD
AREA 2 BEGIN: GRIT REMOVAL AND FINE SCREENS

24-PI INLET GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 6.0 mgd = 9.3 cfs

Diameter of Opening 2 ft
Sluice Gate Percent Open 100%
Discharge Coefficient, C 0.61

	Equation Ref.	HGL	EGL
Velocity through gate, v 2.95 fps			
Energy Loss thru Gate, h_L 0.36 ft			
Condition Upstream of Gate		210.26	210.26

FINE SCREEN EFFLUENT SUMP { 4 }

Friction Loss

Flow	5.98	mgd =	9.3 cfs			
		=	4157.5 gpm			
Channel Width	3	ft		Reference	#21A	
Channel Height	5	ft		Reference	#21B	
Equivalent Pipe Diameter, D	52	inch				
Conduit Length, L	13.66	ft				
Roughness Coefficient, C	120					
Pipe velocity, v	0.62	fps				
Total Friction Loss	0.0004	ft				

MINOR PIPE LOSS HEADING

Flow, Q 6.0 mgd = 9.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Outlet Loss - Still Water	5.98	9.25	1.00	52	----	0.62	----	0.01	0.01
1	Reducer	5.98	9.25	0.25	52	52	0.62	0.62	0.01	0.00
									Sum =	0.00739

Total Energy Loss = 0.01 ft

	HGL	EGL
Upstream Condition	210.27	210.27

FLOW CHANGE/SPLIT

Downstream Flow	5.98	mgd = 9.3 cfs
Fine Screen Bypass	3.00	
Upstream Flow	2.98	mgd =

FINE SCREEN EFFLUENT WEIR (redwood weir plate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 3.0 mgd = 4.6 cfs

WSE Downstream of Weir	210.27	ft
Weir Crest Elevation	208.17	ft
Downstream head, Hd	2.10	ft
Length of Weir, L	5.00	ft

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft	11.52
Upstream WSE	NA	ft	

Submerged Weir Computation { 7 }

K	0.04	
M	3.04	
Increment	0.10	ft
Upstream Head, Hu1	2.10	ft
F(H1)	0.00	
F'(H1)	-0.72	
Upstream Head, Hu2	2.10	ft
Upstream WSE	210.27	ft

Head over Weir *2.10 ft*

	HGL	EGL
Condition Upstream of Weir	210.27	210.27

FINE SCREEN

Flow	3.0	mgd = 4.6 cfs
Blinding	50%	
Maximum Headloss	0.50	ft

	HGL	EGL
Condition Upstream of Bar Screen	210.77	210.77

FINE SCREEN INFLUENT

	Equation Ref.	HGL	EGL

Friction Loss

Downstream Flow	3.0	
Fine Screen Bypass	0	
Upstream Flow	3.0	

Friction Loss

Flow	2.98	mgd =	4.6 cfs
Channel Width	5.00	ft	Reference 2S-1
Total Channel Length	25.50	ft	
Downstream Invert El	207.67		
Upstream Invert El	207.67		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	3.10	ft	
Velocity (Average)	0.30	fps	
Hydraulic Radius (Average)	1.38	ft	

Friction Loss 0.0001 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee- Thru Straight	2.98	4.61	0.60	5.00	----	3.10	0.30	----	0.00	0.00
									Sum Minor Loss=		0.00 ft

Total Energy Loss = 0.0010 ft

Upstream Condition — HGL 210.77 — EGL 210.77

GRIT CHAMBER EFFLUENT WEIR
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	3.0	mgd =	4.6 cfs
WSE Downstream of Weir	210.77	ft	
Weir Crest Elevation	209.37	ft	Reference 2M-2
Downstream head, Hd	1.40	ft	
Length of Weir, L	5.50	ft	Reference 2S-1

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation { 7 }

K	0.03	
M	1.65	
Increment	0.10	ft
Upstream Head, Hu1	1.41	ft
F(H1)	0.00	
F'(H1)	-1.08	
Upstream Head, Hu2	1.41	ft
Upstream WSE	210.78	ft

Head over Weir *1.41 ft*

Condition Upstream of Weir — HGL 210.78 — EGL 210.78

GRIT CHAMBER

Flow	2.98	mgd =	4.6 cfs
Maximum Headloss	2.00	in	Assumed

Condition Upstream of Bar Screen — HGL 210.94 — EGL 210.94

END: GRIT REMOVAL AND FINE SCREENS
BEGIN: YARD

PIPE FROM HEADWORKS TO GRIT CHAMBER
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	3.0	mgd =	4.6 cfs
Pipe Diameter, D	20	inch	
Pipe Length, L	80	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	2.11	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	352419		
Friction factor, f	0.0163		Equivalent Hazen-Williams "C" = 143.8375

	Equation Ref.	HGL	EGL

Friction Energy Loss, h_L 0.05 ft

MINOR PIPE LOSS HEADING

Flow, Q 3.0 mgd = 4.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	0.75	1.15	1.00	----	12	----	1.47	0.03	0.03
1	Entrance Loss - Pipe Ext.	0.75	1.15	1.00	----	12	----	1.47	0.03	0.03
1	Entrance Loss - Pipe Ext.	0.75	1.15	1.00	----	12	----	1.47	0.03	0.03
1	Entrance Loss - Pipe Ext.	0.75	1.15	1.00	----	12	----	1.47	0.03	0.03
1	Tee - Thru Side Outlet	2.98	4.61	1.80	20	----	2.11	----	0.07	0.12
1	Reducer	2.98	4.61	0.25	20	16	2.11	3.30	0.17	0.04
1	Mag Meter	2.98	4.61	0	16	----	3.30	----	0.17	0.00
1	Increaser	2.98	4.61	0.25	16	16	3.30	3.30	0.00	0.00
1	Plug Valve (Open)	2.98	4.61	0.77	16	----	3.30	----	0.17	0.13
1	Increaser	2.98	4.61	0.25	16	20	3.30	2.11	0.10	0.03
1	Mitre Bend - 90 º Deflection	2.98	4.61	1.27	20	----	2.11	----	0.07	0.09
1	Mitre Bend - 90 º Deflection	2.98	4.61	1.27	20	----	2.11	----	0.07	0.09
1	Mitre Bend - 90 º Deflection	2.98	4.61	1.27	20	----	2.11	----	0.07	0.09
1	Mitre Bend - 90 º Deflection	2.98	4.61	1.27	20	----	2.11	----	0.07	0.09
1	Mitre Bend - 90 º Deflection	2.98	4.61	1.27	20	----	2.11	----	0.07	0.09
1	Outlet Loss - Still Water	2.98	4.61	1.00	20	----	2.11	----	0.07	0.07
									Sum =	0.97

Total Energy Loss = 1.02 ft

		HGL	EGL
Influent Wet Well	*Upstream Condition*	211.80	211.79553

	END:	YARD
AREA 1	BEGIN:	HEADWORKS AND INFLUENT PUMP STATION

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	2.98	mgd
Wetwells in service	2	
Flow Upstream	1.49	mgd

HHWL	182.92	ASSUMED USE HWL AT ALL FLOWS	REFERENCE 1M-3
HWL	181.89		
LWL	180.42		
LLWL	178.42		

		HGL	EGL
PS Wetwell Elevation	*Upstream Condition*	181.89	181.89

INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	1.49	mgd =	2.3 cfs
Channel Width	13.83	ft	
Total Channel Length	15.00	ft	
Downstream Invert El	174.42		
Upstream Invert El	174.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	7.47	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	3.59	ft	

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Reducer	1.49	2.31	0.25	6.00	13.83	7.47	0.05	0.02	0.00	0.00
									Sum Minor Loss=		0.0000 ft

Total Energy Loss = 0.0000 ft

	HGL	EGL
Upstream Condition	181.89	181.89

IPS WETWELL INLET GATE

Hydraulix® BY carollo

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

CHECKED : TL **BY :** WME

JOB # : 101321A00 **REVISION:** **DATE :** 2/6/2019 **DATE :** 2/6/2017

				Equation Ref.	HGL	EGL
Flow per Gate	1.49	mgd =	2.3 cfs			
Gate Width	3.5	ft				
Height of Gate	5.0	ft				
Invert Elevation of Gate	180.42					
			Gate is Not Submerged			
Submerged Condition						
Discharge Coefficient, C	N/A					
Velocity through gate, v	N/A	fps				
Not Submerged Condition						
K	1.7		Modeled as gate frame (0.2) and entrance and exit (1.5)			
Water Depth thru Gate	1.47	ft				
Velocity through Outlet, v	0.45	fps				
Energy Loss thru Gate, h_L	0.0053	ft				
			Condition Upstream of Gate		181.89	181.90

HEADWORKS EFFLUENT CHANNEL

Flow Downstream	1.49	mgd	
Wetwells in service	2		
Flow Upstream	2.98	mgd	
Friction Loss			
Flow	1.49	mgd =	2.3 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	4.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.47	ft	
Velocity (Average)	0.52	fps	
Hydraulic Radius (Average)	0.74	ft	
Friction Loss	0.0002	ft	

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Side	1.49	2.31	1.80	6.00	3.00	1.47	0.26	0.52	0.00	0.01
										Sum Minor Loss=	0.0057 ft

Total Energy Loss = 0.0059 ft

	HGL	EGL
Upstream Condition	181.90	181.90

CHANNEL GRINDER EFFLUENT GATE

Flow per Gate	2.98	mgd =	4.6 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.48	ft	
Velocity through Outlet, v	1.04	fps	
Energy Loss thru Gate, h_L	0.0034	ft	

	HGL	EGL
Condition Upstream of Gate	181.89	181.90

CHANNEL GRINDER

Friction Loss			
Flow	2.98	mgd =	4.6 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	16.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.47	ft	
Velocity (Average)	1.05	fps	
Hydraulic Radius (Average)	0.74	ft	

	Equation Ref.	HGL	EGL

Friction Loss 0.0027 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
3	45 degree bend	2.98	4.61	0.25	6.00	2.98	1.47	0.52	1.05	0.01	0.01
									Sum Minor Loss=		0.0097 ft

Other

Grinder 2.00 in Assumed

Total Energy Loss = 0.1791 ft

Upstream Condition	182.07	182.08

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	2.98 mgd =	4.6 cfs
Gate Width	3.0 ft	Reference M-14
Height of Gate	5.0 ft	Reference M-14
Invert Elevation of Gate	180.42	
		Gate is Not Submerged
Submerged Condition		
Discharge Coefficient, C	N/A	
Velocity through gate, v	N/A fps	
Not Submerged Condition		
K	0.2	Modeled as gate frame (0.2)
Water Depth thru Gate	1.65 ft	
Velocity through Outlet, v	0.93 fps	

Energy Loss thru Gate, h_L 0.0027 ft

Condition Upstream of Gate	182.07	182.09

HEADWORKS INFLUENT CHANNEL

Friction Loss

Flow	2.98 mgd =	4.6 cfs
Channel Width	3.00 ft	Reference 1S-1
Total Channel Length	6.00 ft	Reference 1S-1
Downstream Invert El	180.42	Reference 1S-2
Upstream Invert El	180.42	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	1.65 ft	
Velocity (Average)	0.93 fps	
Hydraulic Radius (Average)	0.79 ft	

Friction Loss 0.0007 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	2.98	4.61	0.60	6.00	3.00	1.65	0.47	0.93	0.01	0.01
									Sum Minor Loss=		0.0060 ft

Total Energy Loss = 0.0068 ft

Upstream Condition	182.08	182.09

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	2.98 mgd =	4.6 cfs
Gate Width	3.0 ft	Reference M-14
Height of Gate	5.0 ft	Reference M-14
Invert Elevation of Gate	2.98	
		Gate is Submerged
Submerged Condition		
Discharge Coefficient, C	0.61	
Velocity through gate, v	0.31 fps	
Not Submerged Condition		
K	0.2	Modeled as gate frame (0.2)
Water Depth thru Gate	179.10 ft	
Velocity through Outlet, v	N/A fps	

Energy Loss thru Gate, h_L 0.0039 ft

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00 REVISION:

CHECKED : TL DATE : 2/6/2019 BY : WME DATE : 2/6/2017

	Equation Ref.	HGL	EGL
Condition Upstream of Gate		182.10	182.10

HEADWORKS INFLUENT CHANNEL

Flow Downstream	2.98	mgd
Hdwrks Channels in service	1	
Flow Upstream	2.98	mgd

Friction Loss

Flow	2.98	mgd =	4.6	cfs
Channel Width	3.00	ft		Reference 1S-1
Total Channel Length	6.00	ft		Reference 1S-1
Downstream Invert El	180.42			Reference 1S-2
Upstream Invert El	180.42			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	1.68	ft		
Velocity (Average)	0.92	fps		
Hydraulic Radius (Average)	0.79	ft		

Friction Loss 0.0007 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	2.98	4.61	0.60	6.00	3.00	1.68	0.46	0.92	0.01	0.01
									Sum Minor Loss=		0.0059 ft

Total Energy Loss = 0.0066 ft

	HGL	EGL
Upstream Condition	182.09	182.10

ROCK TRAP { 1 }

Total Flow	2.98	mgd		
Number of online screens	1			
Flow per Screen	2.98	mgd =	4.6	cfs
Channel Flow	2.98	mgd	4.6	cfs
Channel Width	3.00	ft		
Channel & bar rack clearance	0.25	ft		Assumed
Bar Rack Width	2.5	ft		
DS Water Surface Elev	182.09	ft		
Bar Screen Invert Elevation	180.42	ft		
Downstream Water Depth	1.67	ft		
Installation Angle	60	deg		Assumed
Sine Angle	0.8660			
Bar Spacing	1.000	in		Assumed
Bar Thickness	0.313	in		Assumed
Bar Rack Efficiency	0.76			
Bar Rack Open Area	3.6743	sf		
V, velocity Clean Bar Rack	1.26	fps		
v, approach velocity	1.10	fps		
Headloss, clean	0.01	ft		
Upstream Water Depth	1.68			
Blockage	40%			
V, velocity Blocked Bar Rack	2.09	fps		
v, approach velocity	1.06	fps		
Headloss, blocked	0.07	ft	0.87	inches
Upstream Water Depth	1.74			

	HGL	EGL
W - Condition just Upstream of Bar Screen No 1	182.16	182.18

END: HEADWORKS AND INFLUENT PUMP STATION

Appendix 2A-5

HYDRAULIC MODEL CALCULATIONS 9 MGD WITH 33 INCH PI

Hydraulix® BY carollo

PROJECT : OJAI VALLEY SANITATION DISTRICT FACILITIES PLAN

JOB # : 101321A00	REVISION:	CHECKED :	TL	BY :	WME
		DATE :	2/6/2019	DATE :	2/6/2017

	Equation Ref.	HGL	EGL

FACILITIES IN SERVICE

	Total	UIS
UV	1	1
Filters	4	4
Secondary Clarifiers	2	2
Bioreactors	2	2
IPS Screens	1	1

DOWNSTREAM CONTROL

			HGL	EGL
EGL =	**199.05**		184.00	184.00
Flow =	**9.00 mgd =**	**13.92 cfs**		

AREA 7 — BEGIN: FILTER INFLUENT PUMP STATION

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	4.30	mgd	
Flow to EQ Basin	0.00	mgd	
Influent Flow	9.00	mgd	
Filter Backwash	0.17	mgd	
Upstream Flow	9.17	mgd	
HWL	199.05	ASSUMED USE HWL AT ALL FLOWS	REFERENCE 7M-1
LWL	185.00		

		HGL	EGL
PS Wetwell Elevation	*Upstream Condition*	199.05	199.05

FILTER INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	9.17	mgd =	14.2 cfs	
Channel Width	8.25	ft	Reference 7M-1	Assumed, need S dwgs
Total Channel Length	18.00	ft	Reference 7M-2	Assumed, need S dwgs
Downstream Invert El	176.00		Reference 7M-3	Assumed, need S dwgs
Upstream Invert El	176.00			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	23.05	ft		
Velocity (Average)	0.07	fps		
Hydraulic Radius (Average)	3.50	ft		

Friction Loss 0.0000 ft

Other Loss

Turbulence 0.00 in Assumed

Total Energy Loss = 0.0000 ft

	HGL	EGL
Upstream Condition	199.05	199.05

FILTER INFLUENT PUMP STATION
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	9.2	mgd =	14.2 cfs
WSE Downstream of Weir	199.05	ft	
Weir Crest Elevation	191.25	ft	Assumed per Reference 7M-1
Downstream head, Hd	7.80	ft	
Length of Weir, L	18.00	ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation {6}

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation {7}

K	0.02	
M	21.78	
Increment	0.10	ft
Upstream Head, Hu1	7.80	ft
F(H1)	0.00	
F'(H1)	-0.19	
Upstream Head, Hu2	7.80	ft
Upstream WSE	199.05	ft

Head over Weir *7.80 ft*

Condition Upstream of Weir 199.05 199.05

END: FILTER INFLUENT PUMP STATION
BEGIN: YARD

CLARIFIER 1&2 EFFLUENT JUNCTION
FLOW SPLIT

Downstream Flow	9.2	mgd =	14.2	cfs
No. of clarifiers	2.0			
New Flow	4.6	mgd =	7.1	cfs

20-SE SECONDARY CLARIFIER DISCHARGE PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] {4}

Flow 4.6 mgd = 7.1 cfs

Pipe Diameter, D	20	inch
Pipe Length, L	15	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	3.25	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	541983	
Friction factor, f	0.0157	

Equivalent Hazen-Williams "C" = 141.7315

Friction Energy Loss, h_L 0.02 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.6 mgd = 7.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	4.59	7.09	1.00	----	20	----	3.25	0.16	0.16
1	Mitre Bend - 45 º Deflection	4.59	7.09	0.32	20	----	3.25	----	0.16	0.05
1	Tee - Thru Straight Run	4.59	7.09	0.60	20	----	3.25	----	0.16	0.10
1	Outlet Loss - Still Water	4.59	7.09	1.00	20	----	3.25	----	0.16	0.16
									Sum =	0.48

Total Energy Loss = 0.50 ft

Upstream Condition 199.55 199.55

END: YARD
BEGIN: SECONDARY CLARIFIERS

[CRITICAL DEPTH AT CHANNEL DROP (Exit From a Rectangular Channel into a Drop Box)]

Flow, Q 4.6 mgd = 7.1 cfs

Downstream HGL > Invert + Crit. Depth: FLOODED EXIT - CRITICAL DEPTH DOES NOT OCCUR

Downstream WSE	199.55	ft	
Downstream EGL	199.55	ft	
Channel Width, W	2.0	ft	Reference 5S-2
Critical. Depth, y_c	0.73	ft	
Channel Invert @ Exit	197.55	ft	Reference 5S-2

Flooded Condition

Depth Upstream of Drop	2.05	ft
Velocity Upstream of Drop	1.73	fps

Channel Exp./Bend "K"	2.00			
Energy Loss	0.09 ft			
EGL Upstream of Drop	199.65 ft			
HGL Upstream of Drop	199.60 ft			

Freefall Condition

Depth Upstream of Drop	N/A
Velocity Upstream of Drop	N/A
EGL Upstream of Drop	N/A
HGL Upstream of Drop	N/A

Condition Upstream of Drop	199.60	199.65

SECONDARY CLARIFIER EFFLUENT LAUNDER

Clarifier Diameter	85	feet	Reference 5S-2
Weir Diameter	80	feet	

Flow

Downstream Flow	4.6	mgd =	7.1
Split launder	2.0		
New Flow	2.3	mgd =	3.5

Friction Loss

Flow	4.59	mgd =	7.1 cfs	
Channel Width	2.00	ft		Reference 5S-2
Total Channel Length	130.38	ft		Reference 5S-2
Downstream Invert El	197.55			Reference 5S-2
Upstream Invert El	198.37			
Slope	0.63%			
Manning Coeff, n	0.015			
Depth (Average)	1.64	ft		
Velocity (Average)	2.16	fps		
Hydraulic Radius (Average)	0.62	ft		

Friction Loss 0.1174 ft

Upstream Condition	199.69	199.76

SECONDARY CLARIFIER EFFLUENT WEIR
[V-NOTCH WEIR]

Flow	4.59	mgd =	7.1 cfs
WSE Downstream of Weir	199.69	ft	
Weir Crest Elevation	200.43	ft	Reference 5S-4
Downstream head, Hd	-0.74	ft	
Weir Length	251.33	ft	
Distance Between Notches	6.00	in	Reference 5S-4
Number of Notches	502		

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 8 }

Head on Weir, H	0.13 ft
Upstream WSE	200.56 ft

Submerged Weir Computation { 9 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.13 ft*

Condition Upstream of Weir	200.56	200.56

SECONDARY CLARIFIER INFLUENT BAFFLE
[V-NOTCH WEIR]

Downstream Flow	4.59		
RAS	0.26	mgd =	
Dewatering Recycles	0.14		
Upstream Flow	5.0	mgd =	7.7 cfs
WSE Downstream of Weir	200.56	ft	
Weir Crest Elevation	200.43	ft	
Downstream head, Hd	0.13	ft	
Weir Length	78.54	ft	Assumed
Distance Between Notches	6.00	in	Assumed
Number of Notches	157		

WEIR IS SUBMERGED

Free Discharging Weir Computation {8}

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation {9}

K	0.00	
M	0.01	
Increment	0.10	ft
Upstream Head, Hu1	0.22	ft
F(H1)	0.00	
F'(H1)	-25.18	
Upstream Head, Hu2	0.22	ft
Upstream WSE	200.65	ft

Head over Weir *0.22 ft*

Condition Upstream of Weir 200.65 200.65

SECONDARY CLARIFIER FLOCCULATING WELL

- Treat as a submerged orifice.

Flow, Q	4.99	mgd =	7.7 cfs
Downstream WSE	200.56	ft	
Flocculation Diameter	25.00	ft	
EDI Diameter	8.50	ft	
Opening Area	434	sf	
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.02	fps	
Energy Loss , h_L	0.00	ft	

Condition in Flocculating Well 200.65 200.65

SECONDARY CLARIFIER ENERGY DISSIPATION INLET

- Treat as a submerged orifice
- Assume Upstream EGL = HGL

Flow, Q	4.99	mgd =	7.7 cfs
Downstream WSE	200.65	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.00	sf	
Total Port Area	16	sf	
Submerged Port Area	189532%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.48	fps	
Energy Loss , h_L	0.01	ft	

Condition in Influent Well 200.66 200.66

SECONDARY CLARIFIER CENTER COLUMN OUTLETS

- Treat as a submerged orifice

Orifice Loss

Flow, Q	4.99	mgd =	7.7 cfs
Downstream WSE	200.66	ft	
Number of Ports	4		
Port Length	48	inches	
Port Depth	12	inches	
Area per Port	4.0	sf	
Total Port Area	16	sf	
Submerged Port Area	189547%		
Use	100%		
Discharge Coefficient, C	0.61		
Velocity through opening, v	0.48	fps	
Energy Loss , h_L	0.01	ft	

Minor Losses

Flow, Q 4.99 mgd = 7.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mounding Loss	4.99	7.72	0.25	22	----	2.92	----	0.13	0.03
									Sum =	0.03

Total Energy Loss = 0.04 ft

	200.70	200.70

22"ML SECONDARY CLARIFIER CENTER COLUMN
PIPE FRICTION LOSSES (MANNING) - Full Pipe Flow Only

Flow	4.99	mgd =	7.7 cfs	**Flow + Total Recycle + RAS**
Pipe Diameter, D	22	inch		
Pipe Length, L	22	ft		
Manning Coef., n	0.015	ft		
Velocity		2.92 fps		
Hydraulic Radius		0.46 ft		
Friction Energy Loss		0.05 ft		

MINOR PIPE LOSS HEADING

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Bend	4.99	7.72	0.60	22	----	2.92	----	0.13	0.08

Total Minor Losses = 0.08 ft

Total Energy Loss = 0.13 ft

Clarifier center column	*Upstream Condition*	200.70	200.83

END: SECONDARY CLARIFIERS
BEGIN: YARD

22"ML JUNCTION BOX TO CLARIFIER 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow	5.0	mgd =	7.7 cfs
Pipe Diameter, D	22	inch	
Pipe Length, L	220	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	2.92	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	536225		
Friction factor, f	0.0155		Equivalent Hazen-Williams "C" = 142.633
Friction Energy Loss, h_L		0.25 ft	

MINOR PIPE LOSS HEADING

Flow, Q 5.0 mgd = 7.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Mitre Bend - 90 º Deflection	4.99	7.72	1.27	22	----	2.92	----	0.13	0.17
1	Mitre Bend - 45 º Deflection	4.99	7.72	0.32	22	----	2.92	----	0.13	0.04
1	Entrance Loss - Flush	4.99	7.72	0.50	----	22	----	2.92	0.13	0.07
									Sum =	0.277

Total Energy Loss = 0.52 ft

Upstream Condition	201.36	201.36

END: YARD
AREA 4 BEGIN: MIXED LIQUOR SPLITTER BOX

ML JUNCTION FLOW SPLIT
FLOW SPLIT

Downstream Flow	5.0	mgd =	7.7 cfs
No. of SCs Oline	2.0		
New Flow	10.0	mgd =	15.4 cfs

MIXED LIQUOR SPLITTER EFFLUENT

Friction Loss

Flow	9.98	mgd =	15.4 cfs
Channel Width	6.00	ft	Reference 4S-1

Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	10.36	ft	
Velocity (Average)	0.25	fps	
Hydraulic Radius (Average)	2.33	ft	

Friction Loss 0.0000 ft

Other Loss

Turbulence 0.00 in Assumed

Total Energy Loss = 0.0000 ft

Upstream Condition	201.36	201.36

ML SPLITTER BOX WEIR (downward opening weir gates)

[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 10.0 mgd = 15.4 cfs

WSE Downstream of Weir	201.36	ft	
Weir Crest Elevation	201.97	ft	Assume EL per Reference G-3 60"x24" weir gate.
Downstream head, Hd	-0.61	ft	Low position 201.75, top of STR opening is 203.25, so can full close
Length of Weir, L	5.00	ft	

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.95	ft
Upstream WSE	202.92	ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.95 ft*

Condition Upstream of Weir	202.92	202.92

MIXED LIQUOR SPLITTER INFLUENT

Friction Loss

Flow	9.98	mgd = 15.4 cfs	
Channel Width	15.00	ft	Reference 4S-1
Total Channel Length	4.00	ft	Reference 4S-1
Downstream Invert El	191.00		Reference 4S-4
Upstream Invert El	191.00		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	11.92	ft	
Velocity (Average)	0.09	fps	
Hydraulic Radius (Average)	4.60	ft	

Friction Loss 0.0000 ft

Other Loss

Turbulence 0.00 in Assumed

Total Energy Loss = 0.0000 ft

Upstream Condition	202.92	202.92

END: MIXED LIQUOR SPLITTER BOX

BEGIN: YARD

33"ML FROM OX DITCH TEE TO ML SPLITTER BOX

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 10.0 mgd = 15.4 cfs

Pipe Diameter, D 33 inch

Pipe Length, L	95	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.60	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	714966	
Friction factor, f	0.0144	

Equivalent Hazen-Williams "C" = 144.4593

Friction Energy Loss, h_L 0.05 ft

MINOR PIPE LOSS HEADING

Flow, Q 10.0 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	9.98	15.44	0.50	----	33	----	2.60	0.10	0.05
1	Tee - Thru Side	9.98	15.44	1.80	33	----	2.60	----	0.10	0.19
									Sum =	0.24

Total Energy Loss = 0.29 ft

Upstream Condition	203.21	203.21

OX DITCH TEE
FLOW SPLIT

Downstream Flow	10.0	mgd =	15.4	cfs
Ox Ditch online	2			
New Flow	5.0	mgd =	7.7	cfs

22"ML FROM OX DITCH 1 TEE TO OX DITCH TEE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 5.0 mgd = 7.7 cfs

Pipe Diameter, D	22	inch
Pipe Length, L	50	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.92	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	536225	
Friction factor, f	0.0155	

Equivalent Hazen-Williams "C" = 142.633

Friction Energy Loss, h_L 0.06 ft

MINOR PIPE LOSS HEADING

Flow, Q 5.0 mgd = 7.7 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Reducer	4.99	7.72	0.25	33	22	1.30	2.92	0.13	0.03
3	Mitre Bend - 90 º Deflection	4.99	7.72	1.27	22	----	2.92	----	0.13	0.50
									Sum =	0.54

Total Energy Loss = 0.59 ft

Upstream Condition	203.81	203.81

	END:	YARD
AREA 4	BEGIN:	BIOLOGICAL REACTORS

OXIDATION DITCH EFFLUENT WEIR (motorized weir)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 4.99 mgd = 7.7 cfs

WSE Downstream of Weir	203.81	ft	
Weir Crest Elevation	206.03	ft	low position: 204.96; high position: 206.03 — Reference 4M-5
Downstream head, Hd	-2.22	ft	

Length of Weir, L	15.00	ft	Reference M-13

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation {6}

Head on Weir, H	0.29	ft
Upstream WSE	206.32	ft

Submerged Weir Computation {7}

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.29 ft*

Condition Upstream of Weir | 206.32 | 206.32

AEROBIC ZONE

Friction Loss

Flow	4.99	mgd =	7.7 cfs
Channel Width	30.25	ft	Reference 4S-1
Total Channel Length	256.00	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-4
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	14.03	ft	
Velocity (Average)	0.02	fps	
Hydraulic Radius (Average)	7.28	ft	

Friction Loss 0.0000 ft

Other Loss

Baffles 1.00 in Assumed

Total Energy Loss = 0.0833 ft

Upstream Condition | 206.40 | 206.40

TRANSITION FROM ANOXIC TO AEROBIC

Friction Loss

Flow	4.99	mgd =	7.7 cfs
Channel Width	2.50	ft	Reference 4s-4
Total Channel Length	30.00	ft	
Downstream Invert El	192.29		
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	14.11	ft	
Velocity (Average)	0.22	fps	
Hydraulic Radius (Average)	1.15	ft	

Friction Loss 0.0001 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	4.99	7.72	1.00	5.00	8.00	14.11	0.11	0.07	0.00	0.00
1	Sudden Contraction	4.99	7.72	0.50	8.00	5.00	14.11	0.07	0.11	0.00	0.00
										Sum Minor Loss=	0.00 ft

Total Energy Loss = 0.0002 ft

Upstream Condition | 206.40 | 206.40

ANOXIC ZONE

Friction Loss

Flow	4.99	mgd =	7.7 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		

Depth (Average)	14.11	ft			
Velocity (Average)	0.02	fps			
Hydraulic Radius (Average)	7.21	ft			
Friction Loss	0.0000	ft			

Other Loss

Vertical Mixer	0.50	in	Assumed		
Total Energy Loss =	0.0417	ft			
			Upstream Condition	206.44	206.44

ANAEROBIC REACTOR EFFLUENT WEIR (downward opening weir gate
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	5.0	mgd =	7.7 cfs		
WSE Downstream of Weir	206.44	ft			
Weir Crest Elevation	206.71	ft	INV opening: 204.96. Gate height 30-inches.	Reference 4S-7	
Downstream head, Hd	-0.27	ft	Therefore: low position 204.96, high position 207.46	Reference M-14	
Length of Weir, L	3.00	ft			

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation			{ 6 }		
Head on Weir, H	0.84	ft			
Upstream WSE	207.55	ft			
Submerged Weir Computation			{ 7 }		
K	NA				
M	NA				
Increment	NA	ft			
Upstream Head, Hu1	NA	ft			
F(H1)	NA				
F'(H1)	NA				
Upstream Head, Hu2	NA	ft			
Upstream WSE	NA	ft			
Head over Weir	*0.84*	*ft*			
			Condition Upstream of Weir	207.55	207.55

ANAEROBIC ZONE 3

Friction Loss

Downstream Flow	5.0				
Ox Ditches in Service	2				
Upstream Flow	9.9820023				

Friction Loss

Flow	9.98	mgd =	15.4 cfs		
Channel Width	29.50	ft	Reference 4S-1		
Total Channel Length	25.50	ft	Reference 4S-1		
Downstream Invert El	192.29		Reference 4S-7		
Upstream Invert El	192.29				
Slope	0.00%				
Manning Coeff, n	0.015				
Depth (Average)	15.26	ft			
Velocity (Average)	0.03	fps			
Hydraulic Radius (Average)	7.50	ft			
Friction Loss	0.0000	ft			

Other Loss

Vertical Mixer	0.50	in	Assumed		
Total Energy Loss =	0.0417	ft			
			Upstream Condition	207.59	207.59

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 2 and 3
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	10.0	mgd =	15.4 cfs		
			INV opening: 204.96. Gate height 42-inches.	Reference M-14	
WSE Downstream of Weir	207.59	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close		
Weir Crest Elevation	204.96	ft	ASSUMED		
Downstream head, Hd	2.63	ft			

Length of Weir, L	3.00	ft	Reference 4S-7

WEIR IS SUBMERGED

Free Discharging Weir Computation {6}

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation {7}

K	3.10	
M	4.27	
Increment	0.10	ft
Upstream Head, Hu1	2.75	ft
F(H1)	0.00	
F'(H1)	-0.60	
Upstream Head, Hu2	2.75	ft
Upstream WSE	207.71	ft

Head over Weir *2.75 ft*

Condition Upstream of Weir 207.71 207.71

ANAEROBIC ZONE 2

Friction Loss

Flow	9.98	mgd =	15.4 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	15.42	ft	
Velocity (Average)	0.03	fps	
Hydraulic Radius (Average)	7.54	ft	

Friction Loss 0.0000 ft

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition 207.75 207.75

ANAEROBIC ZONE TRANSITION WEIR (downward opening weir gate between zones 1 and 2
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow over weir	10.0	mgd =	15.4 cfs
			INV opening: 204.96. Gate height 42-inches. Reference M-14
WSE Downstream of Weir	207.75	ft	Therefore: low assume position 204.96 and assume high position 208.46 at full close
Weir Crest Elevation	204.96	ft	ASSUMED
Downstream head, Hd	2.79	ft	
Length of Weir, L	3.00	ft	Reference 4S-7

WEIR IS SUBMERGED

Free Discharging Weir Computation {6}

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation {7}

K	3.10	
M	4.65	
Increment	0.10	ft
Upstream Head, Hu1	2.88	ft
F(H1)	0.00	
F'(H1)	-0.56	
Upstream Head, Hu2	2.88	ft
Upstream WSE	207.84	ft

Head over Weir *2.88 ft*

Condition Upstream of Weir 207.84 207.84

ANAEROBIC ZONE 1

Friction Loss

Flow	9.98	mgd =	15.4 cfs
Channel Width	29.50	ft	Reference 4S-1
Total Channel Length	25.50	ft	Reference 4S-1
Downstream Invert El	192.29		Reference 4S-7
Upstream Invert El	192.29		

Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	15.55	ft
Velocity (Average)	0.03	fps
Hydraulic Radius (Average)	7.57	ft

Friction Loss 0.0000 ft

Other Loss

Vertical Mixer 0.50 in Assumed

Total Energy Loss = 0.0417 ft

Upstream Condition	207.89	207.89

ANAEROBIC REACTOR INFLUENT GATE

[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 10.0 mgd = 15.4 cfs

Diameter of Opening	2.5	ft
Sluice Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	3.15	fps

Energy Loss thru Gate, h_L 0.41 ft

Condition Upstream of Gate	208.30	208.30

END: BIOLOGICAL REACTORS

END: YARD

FLOW CHANGE/SPLIT

Downstream Flow	9.98	mgd =	15.4 cfs
Anaerobic Bypass	0.00		
Upstream Flow	9.98	mgd =	

30" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 9.98 mgd = 15.4 cfs

Pipe Diameter, D	30	inch
Pipe Length, L	13	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	3.15	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	786463	
Friction factor, f	0.0145	

Equivalent Hazen-Williams "C" = 143.1717

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 10.0 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Increaser	9.98	15.44	0.25	30	33	3.15	2.60	0.05	0.01
1	Outlet Loss - Still Water	9.98	15.44	1.00	33	----	2.60	----	0.10	0.10
									Sum =	0.12

Total Energy Loss = 0.13 ft

Upstream Condition	208.43	208.43

33" ML (FROM GRIT CHAMBER TO ANAEROBIC REACTOR)

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 9.98 mgd = 15.4 cfs

Pipe Diameter, D	33	inch
Pipe Length, L	280	ft
Absolute Roughness, ε	0.00040	ft

Pipe velocity, v	2.60	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	714966		
Friction factor, f	0.0144		Equivalent Hazen-Williams "C" = 144.4593

Friction Energy Loss, h_L 0.15 ft

MINOR PIPE LOSS HEADING

Flow, Q 9.98 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	Mitre Bend - 90 º Deflection	9.98	15.44	1.27	33	----	2.60	----	0.10	0.27
1	Tee - standard	9.98	15.44	1.50	33	----	2.60	----	0.10	0.16
2	Mitre Bend - 45 º Deflection	9.98	15.44	0.32	33	----	2.60	----	0.10	0.07
1	Tee - Thru Straight	9.98	15.44	0.60	33	----	2.60	----	0.10	0.06
									Sum =	0.55

Total Energy Loss = 0.71 ft

Upstream Condition 209.13 209.13

GRIT CHAMBER AND FINE SCREEN BYPASS

Downstream Flow	9.98	mgd
Bypass	0.00	mgd
Upstream Flow	9.98	mgd =

33" PI (UPGRADE THE 24" FROM GRIT CHAMBER TO ANAEROBIC REACTOR)
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Downstream Flow	9.98	mgd	
RAS split	0.52	mgd	
Upstream Flow	9.46	mgd =	14.6 cfs
Pipe Diameter, D	33	inch	
Pipe Length, L	120	ft	
Absolute Roughness, ε	0.00040	ft	
Pipe velocity, v	2.46	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	677577		
Friction factor, f	0.0145		Equivalent Hazen-Williams "C" = 144.7259

Friction Energy Loss, h_L 0.06 ft

MINOR PIPE LOSS HEADING

Flow, Q 9.98 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	9.98	15.44	0.50	----	33	----	2.60	0.10	0.05
1	Mitre Bend - 90 º Deflection	9.98	15.44	1.27	33	----	2.60	----	0.10	0.13
1	Tee - Standard	9.98	15.44	1.50	33	----	2.60	----	0.10	0.16
									Sum =	0.34

Total Energy Loss = 0.40 ft

Upstream Condition 209.54 209.54

	END:	**YARD**
AREA 2	**BEGIN:**	**GRIT REMOVAL AND FINE SCREENS**

33-PI INLET GATE
[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q 10.0 mgd = 15.4 cfs

Diameter of Opening 2.75 ft

Sluice Gate Percent Open	100%			
Discharge Coefficient, C	0.61			
Velocity through gate, v	2.60 fps			
Energy Loss thru Gate, h_L	0.28 ft			
		Condition Upstream of Gate	209.82	209.82

FINE SCREEN EFFLUENT SUMP { 4 }

Friction Loss

Flow	9.98 mgd =	15.4 cfs		
	=	6937.5 gpm		
Channel Width	3 ft		Reference	#21A
Channel Height	5 ft		Reference	#21B
Equivalent Pipe Diameter, D	52 inch			
Conduit Length, L	13.66 ft			
Roughness Coefficient, C	120			
Pipe velocity, v	1.03 fps			
Total Friction Loss	0.0011 ft			

MINOR PIPE LOSS HEADING

Flow, Q 10.0 mgd = 15.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Outlet Loss - Still Water	9.98	15.44	1.00	52	----	1.03	----	0.02	0.02
1	Reducer	9.98	15.44	0.25	52	52	1.03	1.03	0.02	0.00
									Sum =	0.02057

Total Energy Loss = 0.02 ft

Upstream Condition 209.84 209.84

FLOW CHANGE/SPLIT

Downstream Flow	9.98 mgd =	15.4 cfs
Fine Screen Bypass	3.00	
Upstream Flow	6.98 mgd =	

FINE SCREEN EFFLUENT WEIR (redwood weir plate)
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	7.0 mgd =	10.8 cfs
WSE Downstream of Weir	209.84 ft	
Weir Crest Elevation	208.17 ft	
Downstream head, Hd	1.67 ft	
Length of Weir, L	5.00 ft	

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA ft	11.52
Upstream WSE	NA ft	

Submerged Weir Computation { 7 }

K	0.32	
M	2.16	
Increment	0.10	ft
Upstream Head, Hu1	1.72	ft
F(H1)	0.00	
F'(H1)	-0.93	
Upstream Head, Hu2	1.72	ft
Upstream WSE	209.89	ft

Head over Weir *1.72 ft*

Condition Upstream of Weir 209.89 209.89

FINE SCREEN

Flow	7.0 mgd =	10.8 cfs
Blinding	50%	
Maximum Headloss	0.50 ft	

Condition Upstream of Bar Screen 210.39 210.39

FINE SCREEN INFLUENT

Friction Loss

Downstream Flow	7.0	
Fine Screen Bypass	0	
Upstream Flow	7.0	

Friction Loss

Flow	6.98	mgd =	10.8 cfs
Channel Width	5.00	ft	Reference 2S-1
Total Channel Length	25.50	ft	
Downstream Invert El	207.67		
Upstream Invert El	207.67		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	2.72	ft	
Velocity (Average)	0.80	fps	
Hydraulic Radius (Average)	1.30	ft	

Friction Loss 0.0012 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)	
1	Tee- Thru Straight	6.98	10.80	0.60	5.00	----	2.72	0.80	----	0.01	0.01	
									Sum Minor Loss=		0.01	ft

Total Energy Loss = 0.0070 ft

Upstream Condition 210.38 210.39

GRIT CHAMBER EFFLUENT WEIR
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 7.0 mgd = 10.8 cfs

WSE Downstream of Weir	210.38	ft	
Weir Crest Elevation	209.37	ft	Reference 2M-2
Downstream head, Hd	1.01	ft	
Length of Weir, L	5.50	ft	Reference 2S-1

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation { 7 }

K	0.25	
M	1.02	
Increment	0.10	ft
Upstream Head, Hu1	1.13	ft
F(H1)	0.00	
F'(H1)	-1.65	
Upstream Head, Hu2	1.13	ft
Upstream WSE	210.50	ft

Head over Weir *1.13 ft*

Condition Upstream of Weir 210.50 210.50

GRIT CHAMBER

Flow 6.98 mgd = 10.8 cfs

Maximum Headloss 2.00 in Assumed

Condition Upstream of Bar Screen 210.67 210.67

END: GRIT REMOVAL AND FINE SCREENS
BEGIN: YARD

PIPE FROM HEADWORKS TO GRIT CHAMBER
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 4.0 mgd = 6.2 cfs

Pipe Diameter, D	20	inch
Pipe Length, L	80	ft
Absolute Roughness, ε	0.00040	ft
Pipe velocity, v	2.82	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	470601	
Friction factor, f	0.0159	

Equivalent Hazen-Williams "C" = 142.4975

Friction Energy Loss, h_L 0.09 ft

MINOR PIPE LOSS HEADING

Flow, Q 4.0 mgd = 6.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Pipe Ext.	1.00	1.54	1.00	----	12	----	1.96	0.06	0.06
1	Entrance Loss - Pipe Ext.	1.00	1.54	1.00	----	12	----	1.96	0.06	0.06
1	Entrance Loss - Pipe Ext.	1.00	1.54	1.00	----	12	----	1.96	0.06	0.06
1	Entrance Loss - Pipe Ext.	1.00	1.54	1.00	----	12	----	1.96	0.06	0.06
1	Tee - Thru Side Outlet	3.98	6.16	1.80	20	----	2.82	----	0.12	0.22
1	Reducer	3.98	6.16	0.25	20	16	2.82	4.41	0.30	0.08
1	Mag Meter	3.98	6.16	0	16	----	4.41	----	0.30	0.00
1	Increaser	3.98	6.16	0.25	16	16	4.41	4.41	0.00	0.00
1	Plug Valve (Open)	3.98	6.16	0.77	16	----	4.41	----	0.30	0.23
1	Increaser	3.98	6.16	0.25	16	20	4.41	2.82	0.18	0.04
1	Mitre Bend - 90 º Deflection	3.98	6.16	1.27	20	----	2.82	----	0.12	0.16
1	Mitre Bend - 90 º Deflection	3.98	6.16	1.27	20	----	2.82	----	0.12	0.16
1	Mitre Bend - 90 º Deflection	3.98	6.16	1.27	20	----	2.82	----	0.12	0.16
1	Mitre Bend - 90 º Deflection	3.98	6.16	1.27	20	----	2.82	----	0.12	0.16
1	Mitre Bend - 90 º Deflection	3.98	6.16	1.27	20	----	2.82	----	0.12	0.16
1	Outlet Loss - Still Water	3.98	6.16	1.00	20	----	2.82	----	0.12	0.12
									Sum =	1.72

Total Energy Loss = 1.82 ft

Influent Wet Well *Upstream Condition* 212.32 212.31949

	END:	YARD	
AREA 1	BEGIN:	HEADWORKS AND INFLUENT PUMP STATION	

INFLUENT PUMP STATION WET WELL SETPOINTS

Flow Downstream	3.98	mgd
Wetwells in service	2	
Flow Upstream	1.99	mgd

HHWL	182.92	ASSUMED USE HWL AT ALL FLOWS REFERENCE 1M-3
HWL	181.89	
LWL	180.42	
LLWL	178.42	

PS Wetwell Elevation *Upstream Condition* 181.89 181.89

INFLUENT PUMP STATION WET WELL

Friction Loss

Flow	1.99	mgd = 3.1 cfs
Channel Width	13.83	ft
Total Channel Length	15.00	ft
Downstream Invert El	174.42	
Upstream Invert El	174.42	
Slope	0.00%	
Manning Coeff, n	0.015	
Depth (Average)	7.47	ft
Velocity (Average)	0.03	fps
Hydraulic Radius (Average)	3.59	ft

Friction Loss 0.0000 ft

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
0	Reducer	1.99	3.08	0.25	6.00	13.83	7.47	0.07	0.03	0.00	0.00
										Sum Minor Loss=	0.0000 ft

Total Energy Loss = 0.0000 ft

Upstream Condition 181.89 181.89

IPS WETWELL INLET GATE

Flow per Gate	1.99	mgd =	3.1 cfs
Gate Width	3.5	ft	
Height of Gate	5.0	ft	
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	1.7		Modeled as gate frame (0.2) and entrance and exit (1.5)
Water Depth thru Gate	1.47	ft	
Velocity through Outlet, v	0.60	fps	
Energy Loss thru Gate, h_L	0.0095 ft		

Condition Upstream of Gate 181.89 181.90

HEADWORKS EFFLUENT CHANNEL

Flow Downstream	1.99	mgd	
Wetwells in service	2		
Flow Upstream	3.98	mgd	

Friction Loss

Flow	1.99	mgd =	3.1 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	4.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.47	ft	
Velocity (Average)	0.70	fps	
Hydraulic Radius (Average)	0.74	ft	
Friction Loss	0.0003 ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Side	1.99	3.08	1.80	6.00	3.00	1.47	0.35	0.70	0.01	0.01
										Sum Minor Loss=	0.0102 ft

Total Energy Loss = 0.0105 ft

Upstream Condition 181.90 181.91

CHANNEL GRINDER EFFLUENT GATE

Flow per Gate	3.98	mgd =	6.2 cfs
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.48	ft	
Velocity through Outlet, v	1.39	fps	
Energy Loss thru Gate, h_L	0.0060 ft		

Condition Upstream of Gate 181.89 181.92

CHANNEL GRINDER

Friction Loss

Flow	3.98	mgd =	6.2 cfs
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	16.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.47	ft	
Velocity (Average)	1.40	fps	
Hydraulic Radius (Average)	0.74	ft	
Friction Loss	0.0048 ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
3	45 degree bend	3.98	6.16	0.25	6.00	3.98	1.47	0.70	1.06	0.01	0.01
										Sum Minor Loss=	0.0073 ft

Other

Grinder	2.00	in	Assumed
Total Energy Loss =	0.1787 ft		

Upstream Condition	182.06	182.09

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	3.98	mgd = 6.2 cfs	
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	180.42		
			Gate is Not Submerged
Submerged Condition			
Discharge Coefficient, C	N/A		
Velocity through gate, v	N/A	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	1.64	ft	
Velocity through Outlet, v	1.25	fps	
Energy Loss thru Gate, h_L	0.0048 ft		

Condition Upstream of Gate	182.08	182.10

HEADWORKS INFLUENT CHANNEL

Friction Loss

Flow	3.98	mgd = 6.2 cfs	
Channel Width	3.00	ft	Reference 1S-1
Total Channel Length	6.00	ft	Reference 1S-1
Downstream Invert El	180.42		Reference 1S-2
Upstream Invert El	180.42		
Slope	0.00%		
Manning Coeff, n	0.015		
Depth (Average)	1.66	ft	
Velocity (Average)	1.24	fps	
Hydraulic Radius (Average)	0.79	ft	
Friction Loss	0.0013 ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Tee Thru Straight	3.98	6.16	0.60	6.00	3.00	1.66	0.62	1.24	0.02	0.01
										Sum Minor Loss=	0.0108 ft

Total Energy Loss = 0.0120 ft

Upstream Condition	182.09	182.11

CHANNEL GRINDER INFLUENT GATE

Flow per Gate	3.98	mgd = 6.2 cfs	
Gate Width	3.0	ft	Reference M-14
Height of Gate	5.0	ft	Reference M-14
Invert Elevation of Gate	3.98		
			Gate is Submerged
Submerged Condition			
Discharge Coefficient, C	0.61		
Velocity through gate, v	0.41	fps	
Not Submerged Condition			
K	0.2		Modeled as gate frame (0.2)
Water Depth thru Gate	178.11	ft	
Velocity through Outlet, v	N/A	fps	
Energy Loss thru Gate, h_L	0.0070 ft		

Condition Upstream of Gate	182.12	182.12

HEADWORKS INFLUENT CHANNEL

Flow Downstream	3.98	mgd		
Hdwrks Channels in service	1			
Flow Upstream	3.98	mgd		

Friction Loss

Flow	3.98	mgd =	6.2	cfs
Channel Width	3.00	ft		Reference 1S-1
Total Channel Length	6.00	ft		Reference 1S-1
Downstream Invert El	180.42			Reference 1S-2
Upstream Invert El	180.42			
Slope	0.00%			
Manning Coeff, n	0.015			
Depth (Average)	1.70	ft		
Velocity (Average)	1.21	fps		
Hydraulic Radius (Average)	0.80	ft		
Friction Loss	0.0012	ft		

Minor Loss

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)	
1	Tee Thru Straight	3.98	6.16	0.60	6.00	3.00	1.70	0.61	1.21	0.02	0.01	
									Sum Minor Loss=		0.0102	ft

Total Energy Loss = 0.0115 ft

Upstream Condition 182.11 182.13

ROCK TRAP { 1 }

Total Flow	3.98	mgd		
Number of online screens	1			
Flow per Screen	3.98	mgd =	6.2	cfs
Channel Flow	3.98	mgd	6.2	cfs
Channel Width	3.00	ft		
Channel & bar rack clearance	0.25	ft		Assumed
Bar Rack Width	2.5	ft		
DS Water Surface Elev	182.11	ft		
Bar Screen Invert Elevation	180.42	ft		
Downstream Water Depth	1.69	ft		
Installation Angle	60	deg		Assumed
Sine Angle	0.8660			
Bar Spacing	1.000	in		Assumed
Bar Thickness	0.313	in		Assumed
Bar Rack Efficiency	0.76			
Bar Rack Open Area	3.7109	sf		
V, velocity Clean Bar Rack	1.66	fps		
v, approach velocity	1.45	fps		
Headloss, clean	0.01	ft		
Upstream Water Depth	1.70			
Blockage	40%			
V, velocity Blocked Bar Rack	2.77	fps		
v, approach velocity	1.36	fps		
Headloss, blocked	0.13	ft	1.55	inches
Upstream Water Depth	1.82			

W - Condition just Upstream of Bar Screen No 1 182.23 182.26

END: HEADWORKS AND INFLUENT PUMP STATION

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 3
TOTAL MAXIMUM DAILY LOAD REQUIREMENTS

REVISED FINAL | August 2020

LARRY WALKER ASSOCIATES
Science. Policy. Solutions.

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 3
TOTAL MAXIMUM DAILY LOAD REQUIREMENTS

REVISED FINAL | August 2020

Carollo Project No. 11321A00

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Date: 2020.08.14 13:29:25-07'00'

REGISTERED PROFESSIONAL ENGINEER
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CIVIL
STATE OF CALIFORNIA

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TOTAL MAXIMUM DAILY LOAD REQUIREMENTS

3.1 Introduction and Purpose

This Technical Memorandum (TM) is being developed as part of the 20-year Facilities Plan for Ojai Valley Sanitary District's (OVSD's) wastewater treatment plant (WWTP). Review of regulatory requirements is a critical component of planning for future needs. This TM includes 1) a summary of the requirements and assumptions built into the Ventura River in the 2012 Total Maximum Daily Load (TMDL) for Algae, Eutrophic Conditions, and Nutrients in Ventura River, including the Estuary and its Tributaries (TMDL)[1]; 2) an assessment of the required TMDL sampling and publicly submitted results of the required TMDL monitoring program pertaining to the lower river; 3) an illustration of existing flow conditions in the lower river in both the wet and dry seasons and years; 4) discussion of evidence for non-nutrient related contributions to the impairments addressed by the TMDL, to the extent possible using existing data; and 5) discussion of the potential impacts of different forms of nitrogen on benthic algae in Reaches 1-3 of the river. The regulatory reaches of the Ventura River, and the location of OVSD's outfall, are shown on Figure 3.1.

3.2 Summary of Findings and Conclusions

Key findings are detailed at the end of the memorandum, and can be summarized as follows:

- TMDL Requirements and Assumptions:
  - The selection of a benthic algal biomass target for the Algae TMDL (150 milligram per square meter (mg/m^2) chlorophyll-a (chl.a)) drove the quantification of the required load reductions. The sequence of steps used by Los Angeles Regional Water Quality Control Board (Regional Board) staff to derive load allocations (LAs) resulted in required total nitrogen (TN) and total phosphorus (TP) load reductions of 50 percent for most dischargers.
  - The benthic algal biomass target was not based on evidence linking levels of algal biomass to aquatic life beneficial use impairment (such as low dissolved oxygen (DO) or alteration of benthic invertebrate assemblages). Instead, it was based on subjective interpretations of how much stream algae is likely to impair recreational uses such as wading and trout fishing, and data sets that include few (usually none) southern California streams or streams from other Mediterranean climates.
  - A variety of other key assumptions were described that might or might not hold up if the Algae TMDL was re-opened in the future. Among the vulnerable assumptions are 1) that nutrient loading during wet weather does not contribute to algal-related impairments, 2) that existing loading to the estuary is not high enough to cause impairments of beneficial uses, and 3) that nitrate contributions from daylighting groundwater were correctly characterized. In each case, if the assumption was

[1] Los Angeles Regional Water Quality Control Board Resolution R12-011, adopted December 6, 2012, and becoming effective June 28, 2013.

discarded or revised during development of a future TMDL, estimates of assimilative capacity of TN and/or TP could be lower and more stringent load reductions might be required for dischargers.

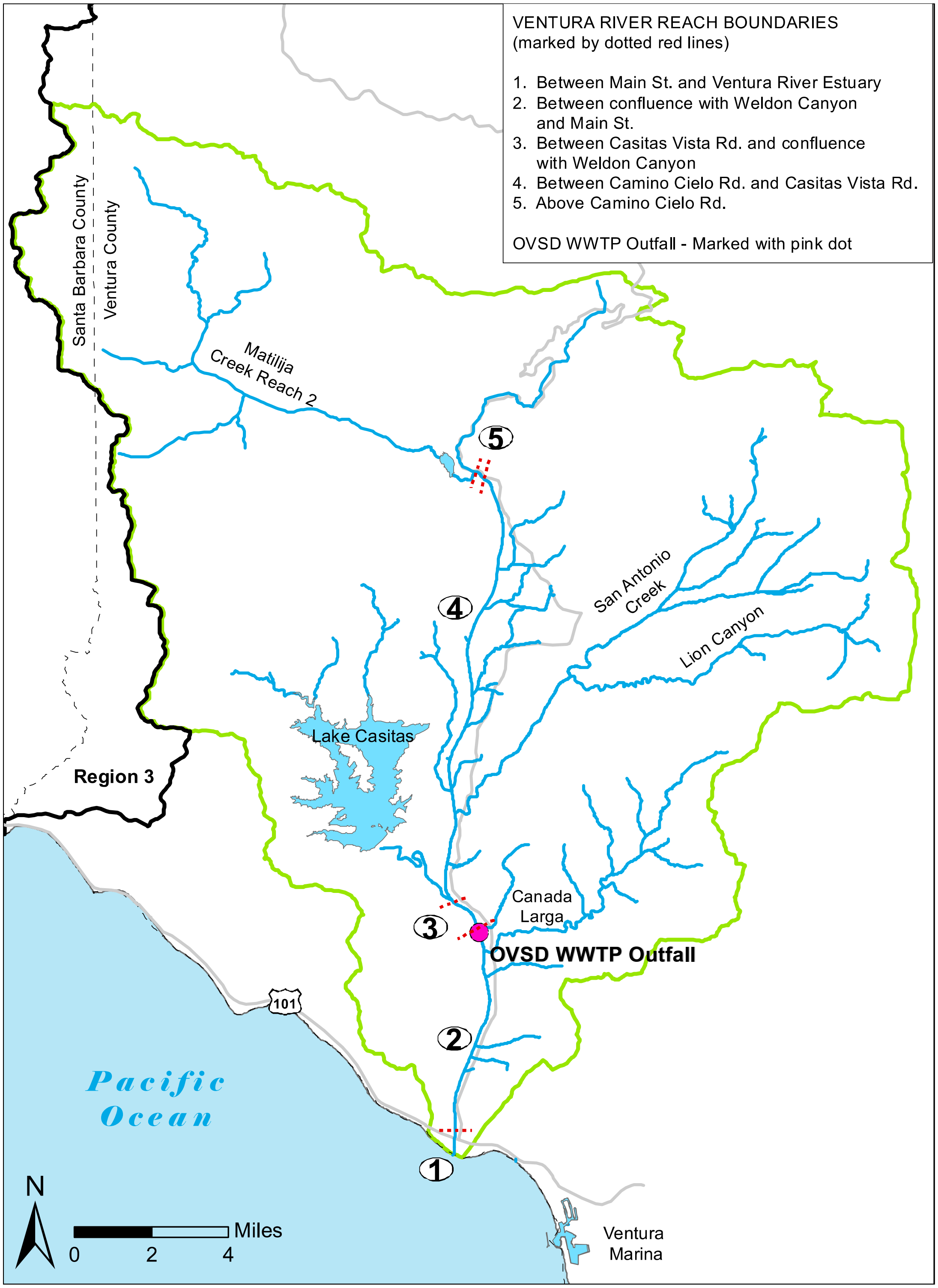

Figure 3.1 Regulatory Reach Designations for the Main Stem of the Ventura River

- TMDL Monitoring Requirements and Results:
  - Exceedances of the algal biomass and DO targets in the TMDL have been frequent in the lower Ventura River and the Estuary since compliance monitoring began in early 2015. Diurnal variations in pH and DO are consistently observed in the river, above and below the discharge, and are strong evidence that submerged plants and algae are exerting an influence on DO. The fact that nutrient loads are lost in a non-conservative fashion in the lower river further supports a strong role of biological uptake in nutrient fate and transport.
  - It is currently unknown whether the river between Foster Park and the OVSD discharge is typically a gaining or losing reach. It will be important to correctly understand the nature of the flow subsidy from the OVSD discharge and the relationship between OVSD nutrient discharges and nutrient loads arriving from upstream.
  - In most months, surface flow decreases between the OVSD outfall and the estuary. The extent to which the loss of surface flow represents direct evaporative losses, uptake and evapotranspiration by aquatic and riparian vegetation, and/or groundwater recharge in the lower river is unknown. Understanding groundwater recharge may become important in the future if groundwater quality in the Lower Ventura River basin becomes an issue with the Regional Board.
  - There is evidence for periodic significant inputs of water and nutrients below the OVSD discharge that are unrelated to OVSD effluent. It will take many years of compliance monitoring to determine whether interannual hydrology (e.g., size and timing of winter storms, juxtaposition of wet and dry years) is responsible for different patterns of fate and transport of nutrients in the lower river. Compliance monitoring is not designed to elucidate which sources of nutrients unrelated to OVSD contribute to those patterns.
- Existing Flow Conditions in the Lower River:
  - An 89-year record of mean daily flows for United States Geological Survey (USGS) Gage 11118500, located at the Foster Park Bridge (top of Reach 3), was used to characterize long-term average patterns of flow for entire Water Year's (WYs) and calendar months.
  - Based on long-term median mean daily flows for calendar months, the OVSD "flow subsidy" ranges from 17 percent (in March) to 92 percent (in September) of total estimated flow at the outfall. However, because days with zero flow are statistically possible during any month at Foster Park, the flow subsidy can intermittently be much higher.
- Evidence for Non-Nutrient Related Contributions to Impairments Addressed by the Algae TMDL:
  - Temperature, conductivity, and flow could all influence DO in the river, but they are not responsible for the strong diurnal variations in DO and pH that are characteristic for the river. Data that would allow evaluation of the effects of canopy cover or other riparian habitat characteristics on algal-related impairments are not being reported by monitoring entities.
  - Empirical relationships between flow, DO, and algal biomass from the Ventura River suggest that mean daily flows $\geq$ 3 cubic feet per second (cfs) would prevent benthic

algae at TMDL target levels (150 mg/m^2 chl.a) from driving pre-dawn DO below 5 milligrams per liter (mg/L).
  - There are several lines of evidence that non-algal factors are influencing daily and monthly patterns of DO in the estuary. The lunar tidal cycle, and particularly the spring/neap tidal cycle, may be driving the timing, frequency, and severity of DO impairments in the estuary. This phenomenon will be important to understand if DO impairments in the estuary are addressed in a new or reopened TMDL.
- Potential Impacts of Different Forms of Nitrogen on Benthic Algae in the Lower River
  - The time frames and extent to which inputs of organic N or particulate N can participate in algal growth has not been studied in the Ventura River. Nutrient spiraling lengths for the Ventura River, are not known. Estimated nutrient uptake lengths cited by the Regional Board in the TMDL Staff Report are approximately half the distance between the OVSD outfall and the head of the estuary, however, the validity of the estimates is not known.
  - Receiving water data from OVSD's required monitoring program revealed high variability in the magnitude and percent organic N in stream water above the OVSD discharge. Frequent high percentages of organic N above the outfall suggest that much of the TN naturally in transport in the lower river would require microbial processing before being eligible to contribute to algal or macrophyte growth.

3.3 Overview of the TMDL Requirements and Assumptions for the Lower Ventura River

Nationwide, the regulatory trend for addressing biostimulatory impairments is to regulate TN and TP. Consistent with this trend, the Regional Board established LAs in the TMDL for a suite of responsible parties for nitrogen and phosphorus based on TN and TP, respectively. The LAs assigned to OVSD in the TMDL were included as effluent limits for the first time in OVSD's National Pollutant Discharge Elimination System (NPDES) permit in 2018. The requirements in the TMDL, and the assumptions that were involved in its development, are summarized in this section.

3.3.1 TMDL Targets

The TMDL established numeric targets for algal-biomass related parameters, DO, and pH for the Ventura River, its tributaries, and the Ventura River Estuary. *The TMDL did not establish targets for nutrients*. Targets are listed in Table 3.1. The DO target is the Basin Plan objective for waters with the COLD and SPWN beneficial uses. The pH target is also from the Basin Plan and applies to all water bodies. The Basin Plan does not contain numeric objectives for biomass or percent cover of phytoplankton or benthic algae. These targets were selected by Regional Board staff based on literature – they were not based on empirical site-specific studies that determined which levels of algae directly or indirectly cause impairments of beneficial uses in the Ventura River.

Table 3.1 Numeric Targets Assigned in the TMDL

Indicator	Numeric Target	Water Body	Basis for Target
Total Algal Biomass(1)	150 mg/m^2 chlorophyll a (May-September average)	Ventura River and Tributaries	Literature
Macroalgal Cover (attached and unattached)	≤ 30 percent (May-September average)	Ventura River and Tributaries	Literature
Phytoplankton Biomass	20 µg/L chlorophyll a (May-September average)	Estuary (shallow subtidal area)	Literature
Macroalgal Cover	≤ 15 percent (May-September average)	Estuary (intertidal and shallow subtidal area)	Literature
Dissolved Oxygen (DO)	≥ 7 mg/L as a daily minimum	Ventura River, Tributaries and Estuary	Basin Plan
pH	6.5 – 8.5 (instantaneous value)	Ventura River, Tributaries and Estuary	Basin Plan

Notes:
Abbreviations: µg/L=micrograms per liter.
(1) Although the label in the TMDL for this target is "Total Algal Biomass", based on the justification for the target in the TMDL Staff Report and based on sampling methods that are being employed in the approved required monitoring program, this target should be interpreted as Benthic Algal Biomass.

3.3.2 TMDL Allocations

The TMDL assigned wasteload allocations (WLAs) for point sources or LAs for non-point sources to the following responsible parties:

- OVSD:
  - Interim dry weather concentration for TN (7.6 mg/L) and TP (2.6 mg/L).
  - Final summer dry weather TN load (8,044 pounds (lb)/season; applies May-September).[2]
  - Final winter dry weather TN load (12, 477 lb/season; applies October-April).[3]
  - Final wet weather TN concentration (7.6 mg/L; applies year round).
  - Final annual dry weather TP load (5,799 lb/season; applies year round during dry weather).[4]
  - Final wet weather TP concentration (2.6 mg/L; applies year round).

For OVSD's permit limits, the TMDL specified that the winter dry-weather seasonal *load* and wet weather *concentration* be combined into a single weighted concentration-based winter season effluent limit assuming fixed constants of 178 dry days and 34 wet days per winter. In the 2018 permit, the outcome of this procedure was a final winter effluent limit of 4.6 mg/L TN.

- Ventura County Municipal Stormwater Dischargers:
  - Final dry weather TN daily load (28 lb/day; applies year round on dry days).[5]

[2] Allocation developed assuming an average of 153 dry weather days between May-September.
[3] Allocation developed assuming an average of 178 dry weather days between October-April.
[4] Allocation developed assuming an average of 331 dry weather days per year.
[5] Ibid.

  - Final dry weather TP daily load (0.5 lb/day; applies year round on dry days).[6]
  - Final wet weather nitrate+nitrite-N concentration (7.4 mg/L in estuary and Reach 1; 10 mg/L in Reach 2; 5 mg/L in other reaches).
- Caltrans:
  - Final dry weather TN daily load (1.1 lb/day; applies year round on dry days).[7]
  - Final dry weather TP daily load (0.11 lb/day; applies year round on dry days).[8]
  - Final wet weather nitrate+nitrite-N concentration (7.4 mg/L in estuary and Reach 1; 10 mg/L in Reach 2; 5 mg/L in other reaches).
- General Industrial and General Construction Stormwater Permittees:
  - Final dry weather TN concentration (1.15 mg/L; as annual dry weather average).
  - Final dry weather TP concentration (0.115 mg/L; as annual dry weather average).
- Agriculture:
  - Final dry weather TN daily load (16 lb/day; applies year round on dry days).[9]
  - Final dry weather TP daily load (0.12 lb/day); applies year round on dry days).[10]
  - Final wet weather nitrate+nitrite-N concentration (10 mg/L in lower reaches, 5 mg/L in upper reaches).
- Horses and Intensive Livestock:
  - Final dry weather TN daily load (0.6 lb/day; applies year round on dry days).[11]
  - Final dry weather TP daily load (0.14 lb/day); applies year round on dry days).[12]
  - Final wet weather nitrate+nitrite-N concentration (10 mg/L in lower reaches, 5 mg/L in upper reaches).
- Grazing Activities:
  - 10 percent reduction of existing TN and TP load, to be determined (TBD) based on required management plan by affected parties.
- Onsite Wastewater Treatment Systems (OWTS):
  - Final annual TN load (7,478 lbs/year; based on 50 percent load reduction).

3.3.3 Relationship Between TMDL Targets and OVSD's Wasteload Allocations

The selection of a target for benthic algal biomass, and the modeling that was conducted by Regional Board staff using that target, were critical elements driving the required allocations assigned in the TMDL. The benthic algal biomass target of 150 mg/m^2 chl.a drove the quantification of the required load reductions in a series of steps that can be summarized as follows:

1. The dry season (May-September) was identified as the critical condition for impairments related to algae, and thus the modeling of required load reductions focused on outcomes during dry weather.
2. A nutrient model for the main stem of the river and its tributaries was developed by Regional Board staff using the QUAL2K framework that translated estimated existing nutrient loads to the river into in-stream nutrient concentrations and predicted growth

[6] Ibid.
[7] Ibid.
[8] Ibid.
[9] Ibid.
[10] Ibid.
[11] Ibid.
[12] Ibid.

of suspended and benthic algae. The model parameterization, calibration and validation are described in Appendix B to the TMDL Staff Report.[13]

3. Regression analysis of model output was used to derive mathematical relationships between benthic algal biomass and TN and TP (shown on Figures 4.5 and 4.6 in Appendix B, respectively).
4. The regression equations were used to identify TN and TP thresholds (1.15 and 0.067 mg/L, respectively) that corresponded to benthic algal biomass of 150 mg/m^2 chl.a. The TP threshold was adjusted by applying a ratio of 10:1 TN/TP to the TN threshold, resulting in an adjusted TP threshold of 0.115 mg/L.
5. Estimated existing annual nutrient loads for eight sources[14] were used in the model to explore a variety of scenarios of nutrient load reductions. The various scenarios are not explained in Appendix B. A single scenario was presented that would achieve the TN and TP thresholds in river water in the various reaches (and thus presumably, the target benthic algal biomass). The identified scenario required that combined TN and TP loads to the river from discharges be reduced by 50 percent.
6. Dry weather WLAs for point sources and LAs for non-point sources were assigned based on the estimated existing loads from the TMDL Source Assessment and the following percent reductions:
   a. Ventura MS4 — 50 percent TN and TP.
   b. Caltrans — 50 percent TN and TP.
   c. OVSD — 49 percent TN, 28 percent TP.
   d. Agriculture — 50 percent TN and TP.
   e. Onsite Wastewater Treatment Systems (OWTS) — 50 percent TN only.
   f. Horses/Intensive Livestock — 99 percent TN and TP.

   The TMDL Staff Report is silent regarding whether other allocation schemes that could also have resulted in the needed reduction in ***total*** watershed TN and TP loading.
7. The TMDL Staff Report identified a phytoplankton biomass target of 20 µg/L for the estuary based on a 1999 NOAA publication.[15] The BATHTUB model was run to estimate predicted phytoplankton chl.a resulting from existing TN and TP loads to the estuary. Because the model-predicted phytoplankton biomass resulting from existing loads was 18 µg/L, Regional Board staff concluded that the load reductions developed from steps 1-7 would be protective of beneficial uses in the estuary as well. Thus no adjustments to the required load reductions were made based on outcomes in the estuary.

[13] Lai, C.P. (2012) Algae and Nutrient Modeling for Ventura River, July 19, 2012. Available at https://www.waterboards.ca.gov/losangeles/board_decisions/basin_plan_amendments/technical_documents/bpa_73_R12-XXX_td.html.

[14] The eight sources were 1) dry weather runoff from undeveloped areas, 2) OVSD WWTP, 3) animal waste from horses and intensive cattle operations, 4) septic tanks, 5) agriculture, 6) dry weather urban runoff, 7) runoff from Caltrans, and 8) atmospheric deposition.

[15] Bricker, S.B., C.G. Clement, D.E. Pirhalla, S.P. Orlandom and D.R.G. Farrow (1999) National Estuarine Eutrophication Assessment: Effects of Nutrient Enrichment in the Nation's Estuaries. NOAA, National Ocean Service, Special Projects Office and the National Centers for Coastal Ocean Science. Silver Springs, MD, 71 pp.

8. Percent cover targets for algae were drawn from literature, and played no direct role in calculation of require load reductions.
9. Because wet weather nutrient loads were not judged to have significant impact on receiving water quality, wet weather allocations for most dischargers were set equal to the Basin Plan objectives for Nitrate-N+Nitrite-N, which are either 10 mg/L (for Reach 2 and Cañada Larga) or 5 mg/L (for Reaches 3-5, and San Antonio Creek). For OVSD, the wet weather WLAs for TN and TP (7.6 and 2.6 mg/L, respectively were set equal to 90th percentile of existing performance between 2000-2012). In other words, the wet weather WLAs and LAs in the TMDL are not based on algae-related impairments.

The Regional Board modeling results were released in July 2012 at the same time as the draft TMDL. Practically speaking, responsible parties did not have time to conduct or commission technical review of the QUAL2K or BATHTUB models (for the river itself and the estuary, respectively) between the release of the draft TMDL and its adoption by the Regional Board in December 2012. However, for the most part, Regional Board staff justified the benthic algal biomass target using information submitted to them in a report by University of California researchers (UCSB Report) following a one-year project (May 2008-April 2009) that was funded by the State Water Resources Control Board (State Board).[16] Among other analyses, Larry Walker Associates (LWA) reviewed the basis for the benthic algal biomass target in a TM prepared for OVSD in 2009.[17] As explained in LWA (2009), the benthic algal biomass target of 150 mg/m^2 chl.a is based primarily on a set of public and scientific perceptions contained in a small number of references from the scientific and gray literature. The literature values are based on data sets that include few (usually none) southern California streams or streams from other Mediterranean climates. None of the literature thresholds were based on data that associate aquatic life beneficial use impairment (such as low DO or alteration of benthic invertebrate assemblages) with the thresholds. Instead, they mostly constitute subjective interpretations of how much stream algae is likely to impair recreational uses, such as wading and trout fishing. Impairment of trout fishing is particularly irrelevant to beneficial uses in the Ventura River, where the only salmonids present are endangered Southern California Steelhead.

3.3.4 Other Key Assumptions in the TMDL

In addition to the key assumption that an 150 mg/m^2 chl.a is an appropriate impairment threshold for the Ventura River (discussed in the preceding paragraphs), Regional Board staff relied on myriad other estimations and assumptions to perform the source assessment, linkage analysis, and model parameterization. A comprehensive evaluation of all of the assumptions used by Regional Board staff would be outside the scope of this memorandum. Selected key assumptions that directly affected the approach used to derive load reductions and allocations are listed as follows:

- Owing to scouring and channel modification during large winter storms, Regional Board staff acknowledged that interannual variation in algal biomass could be closely related to rainfall in the preceding year. Because storm-induced channel modification can

[16] Klose et al. (2009) An Assessment of Numeric Algal and Nutrient Targets for Ventura River Watershed Nutrient TMDLs, prepared for the Los Angeles Regional Water Quality Control Board, May 2009.
[17] LWA (2009) Comments regarding the UCSB (2009) Numeric Target Recommendation Report. Technical Memorandum submitted to Ojai Valley Sanitary District, December 15, 2009. 36 pp.

remove riparian cover and increase habitat suitability for benthic algae, Regional Board staff concluded that watershed-wide projects designed to control stream algae by increasing riparian vegetation and canopy cover would be insufficient to address algae-related impairments (Staff Report, p. 28-30).

- Watershed-wide wet-weather loads were treated as fluxes to the ocean that do not contribute to biostimulatory impairments in the river or estuary (Staff Report, p. 32).
- Annual loading to the estuary was assumed equal to annual dry weather load entering from upstream (Staff Report, p. 66). Based on the BATHTUB modeling exercise, this loading was not viewed high enough to cause exceedances of the phytoplankton or macroalgae targets for the estuary. The model-predicted load reductions necessary to meet the benthic algal biomass target of 150 mg/m^2 in the river were thus viewed sufficiently protective of the beneficial uses in the estuary (Staff Report, p. 71).
- The assumption regarding how many dry-weather days and wet-weather days occur on average each year was based on precipitation data from Ventura County Watershed Protection District (VCWPD) Gage 020 from 1987-2007 (Staff Report, p. 44).
- Values for OVSD effluent used to estimate existing loading were based on averages from 2000-2012, and were as follows (Staff Report, p. 45):
  - Average effluent flow= 2.1 million gallons per day.
  - Average TN = 5.86 mg/L.
  - Average TP = 1.38 mg/L.
- Regional Board staff assumed that no dry weather runoff occurs from orchards in the watershed. Average nitrate and phosphate concentrations in non-orchard dry weather runoff was based on monitoring data from VCAILG's Central Ditch[18] site on the Oxnard Plain, and were 15.4 mg/L and 0.06 mg/L, respectively (Staff Report, p. 48).
- Regional Board staff used the N and P loading for OWTS estimated by LWA (2011)[19] in the Source Assessment and modeling (Staff Report, p. 53).
- Groundwater discharge to surface water overlying the Lower Ventura River sub-basin was estimated as 1.73 cfs, with average nitrate-N of 1.23 mg/L (Staff Report, p. 56).

Based on professional opinion, some of these key assumptions might not hold up if the Algae TMDL was re-opened in the future. Examples are explained in the following.

3.3.4.1 Fate of Wet Weather Nutrient Loads

During modeling of required load reductions, the Regional Board assumed that nutrient loading during wet weather does not contribute to algal-related impairments in the river and that annual loading to the estuary should be based only on dry weather inputs. However, depending on the size and frequency of storm events, particulate matter carried in stormwater can be deposited in stream channels and in estuaries (rather than carried out to the ocean). N and P in these deposits can be liberated and could contribute to algal growth during intervals of dry weather or later in the year when conditions for algal growth are more favorable. Consequently, in a future TMDL, different authors could conclude that some fraction of the particulate nutrient loads entering the river or estuary during wet weather could be considered as contributors to impairments during dry weather.

[18] VCAILG is the abbreviation for the Ventura County Agricultural Irrigated Lands Group.
[19] LWA (2011) Corrected Source Assessment Report: Nitrogen and Phosphorus in the Ventura River Watershed. Prepared for the Ojai Valley Sanitary District, August 9, 2011. 73 pp.

3.3.4.2 Existing Loads as Sufficient to Meet Beneficial Uses in the Estuary

The BATHTUB modeling results for the estuary suggested to the Regional Board that upstream load reductions were not needed to protect beneficial uses in the estuary. However, as explained in the following paragraphs, exceedances of the TMDL targets in the estuary have been common since compliance monitoring began in spring 2015. If a pattern of exceedances continues into the future, the Regional Board would be likely to revisit that assumption. This is important, because changes in assumptions about how the estuary responds to nutrient loads could trigger more stringent load reductions in the river reaches upstream from the estuary, even if TMDL targets were being met in the river reaches.

3.3.4.3 Contributions of Groundwater to Background Loads of N

Better estimates of groundwater discharges to the river will become available through monitoring and modeling being conducted by the Upper Ventura River Groundwater Agency, as part of its development of a Groundwater Sustainability Plan for the Upper Ventura River Basin (see TM 4 for more detail). Consequently, based on new data, the Regional Board might revise their assumptions about the flux of nutrients (principally nitrate) that could be entering surface water from daylighting groundwater. In the TMDL context, background loads of nutrients (such as from open space or groundwater) affect estimates of assimilative capacity for a water body. If groundwater inputs of nitrate increased estimates of background TN loads, larger load reductions might be required for dischargers in a future TMDL.

3.4 TMDL Monitoring Requirements and Results

3.4.1 TMDL Required Monitoring

The TMDL required that OVSD, VCWPD, Ventura County, Cities of Ojai and Ventura, Caltrans, and agricultural dischargers carry out a comprehensive monitoring program (CMP) (TMDL). The monitoring plan for the TMDL CMP[20] was submitted to the Regional Board in June 2014, and compliance monitoring began in January 2015. Four of the monitoring sites from this program are in the lower Ventura River (TMDL-Est, TMDL-R1, TMDL-R2, and TMDL-R3). Flow is measured (at sites in the river) and grab samples for nutrients (TN, total dissolved nitrogen (TDN), NO_3+NO_2-N, total Kjeldahl nitrogen (TKN), dissolved TKN, TP, total dissolved phosphorus (TDP)), DO, pH, temperature, and specific conductivity (SC) are collected on a monthly basis year round. Percent DO saturation is not reported by the implementing agency. Benthic algal biomass (mg/m^2 chl.a) and percent cover of macroalgae are sampled monthly at TMDL-R1, TMDL-R2, and TMDL-R3 during May-September (and at other sites higher in the watershed). In addition, phytoplankton biomass (µg/L chl.a) and percent cover of "land-based" and floating macroalgae are sampled monthly in the estuary (TMDL-Est) during May-September.

Data loggers are deployed for about two weeks four times per year (November, February, May, and September) each year at all of the lower Ventura River sites to collect data for pH, temperature, SC, and DO at 15-minute intervals. Equipment issues (loss of equipment, fouling, calibration issues) have been common in the program.

[20] Ventura River and Tributaries Algae, Eutrophic Conditions, and Nutrients TMDL. Draft Comprehensive Monitoring Plan for Receiving Water. Prepared by the Ventura County Watershed Protection District, June 27, 2014.

3.4.2 Pertinent Monitoring by Other Entities

Other entities that have long-term established monitoring sites in the Ventura River are Santa Barbara Channelkeeper (SBCK), OVSD (through its NPDES permit required receiving water monitoring), and Ventura County Stormwater Quality Monitoring Program (VCSQMP) which operates a mass emission site on the Ventura River (ME-VR2). Pertinent categories of data generated by these entities is described below.

SBCK. Grab samples for grab samples for DO, temperature, SC, pH and nutrients are obtained at sites VR000, VR001, VR3.5, VR6.1, and VR006, on a more-or-less monthly basis. SBCK reports percent DO saturation. Although nutrient samples (just nitrate and phosphate) are collected by SBCK on a monthly basis at several sites in the lower Ventura River, no nutrient samples have been analyzed by their laboratory partner since April 2012, and there is no expectation that results will be available in the near future.[21] In 2013, SBCK began deploying DO data loggers at two sites in the lower Ventura River (DS1 south of Main Street, DS6 between SBCK 6.1 and Foster Park) for durations lasting several weeks. However, results from deployments starting in 2015 have not been publicly available.

OVSD. OVSD monitors DO, temperature, flow, and pH (but not SC) at three receiving water sites (OVSD RSW-003, RSW-004, and RSW-005) on a monthly basis. In addition, several nutrient parameters are measured at the same three receiving water sites quarterly. OVSD does not report percent DO saturation for receiving water samples, and because SC is not measured during their receiving water events, it is not possible to calculate. Biological oxygen demand (BOD) is monitored in effluent and at receiving water sites (the latter only quarterly, coinciding with nutrient samples).

VCSQMP. Ordinarily, the VCSQMP samples one dry-weather and three wet-weather events at ME-VR2 during each Permit year. Pertinent parameters reported are pH, temperature, SC, DO (both concentration and percent saturation), BOD, TN, ammonia-N, TKN, nitrate+nitrite-N, TP, TDP. The spatial juxtaposition of the various programs' monitoring sites in the Lower Ventura River (i.e., Foster Park and below) is depicted on Figure 3.2.

[21] Ben Pitterle, personal communication, April 2016.

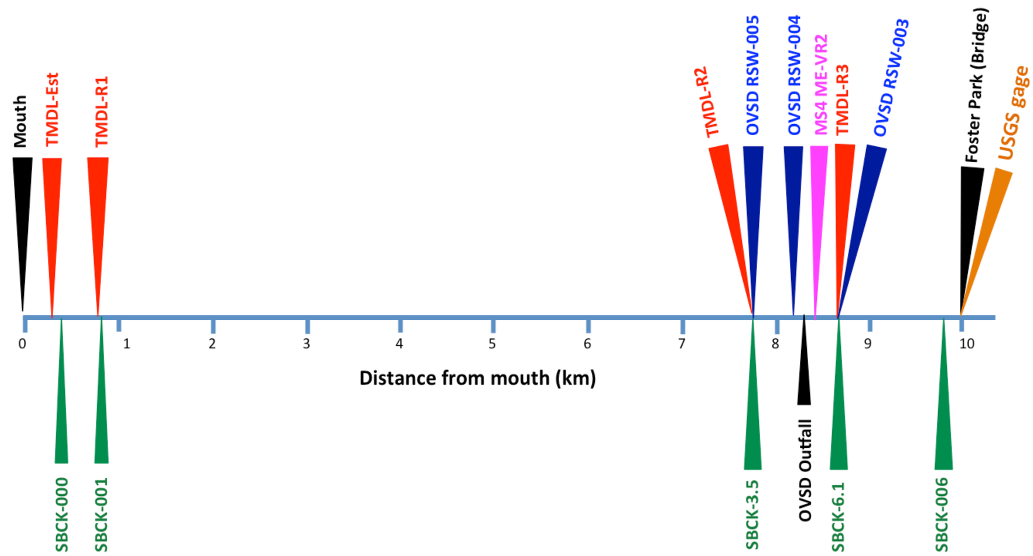

Figure 3.2 Locations of Established Long-term Monitoring Sites in the Lower Ventura River Reaches. Acronyms are explained in the text. Figure is from LWA (2018) *Lower Ventura River Receiving Water Quality Data Review: October 2015-September 2018*. TM prepared for OVSD, June 18, 2018

3.4.3 Assessment of TMDL Compliance Monitoring

3.4.3.1 Summary of TMDL Target Exceedances

Tables 3.2 – 3.4 summarize exceedances of numeric targets in the TMDL that have been reported by the TMDL CMP from the onset of the program in Spring 2015 through the most recent annual report submitted to the Regional Board in June 2018. As stated previously, grab samples for various other parameters that lack TMDL numeric targets, such as nutrient constituents and flow, are monitored in the program, but the results are not placed in any useful context in the TMDL CMP monitoring reports, and are not summarized here. Grab samples for pH and DO obtained during monthly monitoring events are also reported– but are not very informative and were not inspected for exceedances. Instead, continuous monitoring results from the multi-parameter logger deployments were inspected for exceedances of the pH or DO targets. Values that exceed the target are shown in red text.

Table 3.2 Monitoring Results From the TMDL CMP for Algae-Based Targets in the Estuary(1)

May-September Seasonal Average	Phytoplankton Biomass µg/L (Target = 20 µg/L)	Macroalgal Cover (%) (Target ≤ 15%)	
		Land-based %	Floating %
2015	6.4	10.84	0.15
2016	34	3.84	0.10
2017	266	9.01	0.17

Notes:
(1) Exceedances of numeric TMDL targets are in red.

Table 3.3 Summary of Monitoring Results from the TMDL CMP for Algae-Based Targets at Sites in the Lower River(1)

May-September Seasonal Average	Site	Chlorophyll a (Target = 150 mg/m^2)	Macroalgal Cover (Target ≤ 30%)
2015	TMDL-R1	254.5	4.8
	TMDL-R2	89.6	2.7
	TMDL-R3	69.7	19.9
2016	TMDL-R1	173	15.1
	TMDL-R2	180	4.5
	TMDL-R3	80	12.0
2017	TMDL-R1	302	36.0
	TMDL-R2	366	44.7
	TMDL-R3	247	61.3

Notes:
(1) Exceedances of numeric TMDL targets are in red.

Table 3.4 Summary of Continuous Monitoring Results for DO and pH in the Lower River from the TMDL CMP

Deployment	Sites with pH Exceedances	Sites with DO Exceedances
May 2015	none	Estuary, R1, R3
September 2015	none	Estuary, R1, R2, R3
May 2016	none	Estuary, R2
September 2016	none	Estuary, R2, R3
November 2016	none	R3(2)
March 2017	none	Estuary, R2
May 2017	none	Estuary, R1, R2, R3
September 2017	none	R1, R2, R3(1)

Notes:
(1) Sonde deployed in the estuary was lost.
(2) Few brief exceedances.

3.4.3.2 Assessment of Monitoring Results for the Lower River

Starting in 2015, on behalf of OVSD, LWA has conducted periodic reviews of receiving water quality data from the lower Ventura River focusing on parameters that are related to the impairments addressed by the TMDL (benthic algal biomass, pH, DO concentrations and percent saturation, nitrogen and phosphorus parameters, flow, temperature, etc.). The objective of each review has been to 1) compare recent pertinent available monitoring results from SBCK, OVSD's NPDES receiving water and effluent monitoring, the TMDL-CMP, and the VCSQMP mass emission site on the Ventura River (ME-VR2), 2) place the results in context with other pertinent publicly available data as needed (such as precipitation and USGS discharge data), and 3) identify key discrepancies, data gaps, or other features of the data that would be useful for eventual interpretation of long-term data sets by OVSD or other parties. Three technical memoranda have been prepared to date that in combination address available monitoring data for the period

January 2015-September 2017, with an emphasis on monitoring data for the May-September critical algae growth season (as defined by the TMDL).[22,23,24]

Among the data explorations presented in the LWA memoranda, flow data, calculated nutrient loads, and benthic algal biomass data have been evaluated in a number of ways to shed light on fate and transport of nutrients and water in the lower river. Selected observations are provided below.

Longitudinal patterns of surface flows in the lower river

Flow measurements made by different entities consistently provide conflicting evidence regarding whether the river between Foster Park and the OVSD discharge is a gaining or losing reach. Resolution of this discrepancy will be important to correctly understand the nature of the flow subsidy from the OVSD discharge and the relationship between OVSD nutrient discharges and nutrient loads arriving from upstream. However, based only on flow measurements made by the TMDL CMP, surface flow decreases between the OVSD outfall and the estuary in most months. The extent to which the loss of surface flow represents direct evaporative losses, uptake and evapotranspiration by aquatic and riparian vegetation, and/or groundwater recharge in Reaches 1-2 is unknown.

An exception to this pattern was observed in March, April, and May 2017 when surface flow was higher at the Main Street bridge (TMDL-R1) than flow estimated at the outfall (the latter, after accounting for effluent flow). These months coincided with a prolonged descending hydrograph that occurred after the early 2017 storms. Surface or groundwater contributions entering the active channel *below the outfall* apparently created a gaining reach between the outfall and the estuary during Spring of 2017. Although WY2017 had much higher annual discharge than WYs 2015 and 2016, it was still a below-average WY.

Evidence for non-conservative nutrient sinks in the lower river

In most time periods examined, nutrient loads in the water column in the lower river dropped between sampling locations faster than can be accounted for by disappearance of surface flows. Nutrient loads usually decrease by larger percentages in this part of the river during the summer months than winter months. These outcomes support a hypothesis that nutrients are lost in a non-conservative fashion during transport between the OVSD outfall and TMDL-R2 (at Cañada Larga) and TMDL-R1 (Main St. Bridge). Because the loads measured include both dissolved and particulate fractions, values for TN and TP will include N and P incorporated in planktonic algal biomass moving downstream. To the extent that phytoplankton remain in suspension, incorporation of N and P into phytoplankton biomass will not change the load of TN and TP in transport. However, incorporation of N and P into *stationary* biomass (e.g., attached algae and other microbiota, rooted or floating macrophytes, riparian vegetation) can contribute to the retention of nutrients between the outfall and the estuary. The fact that strong diurnal patterns in pH and DO are also observed in the lower river is consistent with a proposal that submerged photosynthetic biomass (either submerged macrophytes or algae) has an important influence on

[22] LWA (2016) Semi-Annual Lower Ventura River Receiving Water Quality Data Review: January-April 2015. Technical Memorandum prepared for OVSD, December 20, 2016.
[23] LWA (2016) Semi-Annual Lower Ventura River Receiving Water Quality Data Review: May-September 2015. TM prepared for OVSD, December 20, 2016.
[24] LWA (2018) Lower Ventura River Receiving Water Quality Data Review: October 2015-September 2018. Technical Memorandum prepared for OVSD, June 18, 2018.

stream chemistry; rooted aquatic vegetation with photosynthetic tissue above water will remove nutrients from water, but will not cause DO maxima during daylight hours.

Interestingly, between May-September 2015, monitoring data showed a potential inverse relationship between nutrient loss and benthic algal biomass between the outfall and TMDL-R2. Larger decreases in nutrient loads in the water column between the OVSD outfall and TMDL-R2 were associated with lower benthic algal biomass at TMDL-R2, and vice versa. This implies a significant non-algal nutrient sink in that reach. Because, as discussed in the preliminary paragraphs, nutrient loads decrease faster than surface flows in the reach, the sink cannot solely represent conservative losses through infiltration. The sink may include uptake by non-algal primary producers, such as aquatic macrophytes or riparian vegetation.

Evidence for influx of nutrients below the OVSD discharge

Data from 2017 provide evidence for a source of nutrients in Reaches 1-2 unrelated to the OVSD discharge. An increase in non-effluent-related TN occurred between the Cañada Larga confluence and the Main St. bridge in February and March 2017. In February, a significant flux of TP below the OVSD discharge (also not from effluent) more than quadrupled the amount of TP in transit between the OVSD outfall and the Main St. bridge. Additions of non-effluent-related TN and TP between the outfall and the confluence with Cañada Larga were observed in March, April, May and July 2017. These months coincide with a prolonged descending hydrograph that occurred after the early 2017 storms; surface or groundwater contributions below the outfall apparently created a gaining reach between the outfall and the estuary during Spring of 2017. Although WY 2017 had higher annual discharge than 2016, it was still a below-average WY. Compliance monitoring is not designed to elucidate the sources of water or nutrients that enter the river below the OVSD discharge.

3.5 Illustration of Existing Flow Conditions in the Lower River

Existing flow conditions in the lower Ventura River were evaluated using all available data for mean daily discharge for the USGS gage located at the top of Reach 3 at the Foster Park Bridge (WYs 1930-2018; USGS 11118500 *VENTURA R NR VENTURA*).[25] A time series of annual discharge is provided on Figure 3.3, and shows the expected highly variable inter-annual pattern wherein most years are either wet years well above the long-term average annual discharge for the period (approximately 45,000 acre-feet) or drier years with annual discharge well below the long-term average.

[25] WYs begin in October and end in September the following year, and are labeled according to the year in which they end (i.e., WY 2018 is for the period October 1, 2017-September 30, 2018).

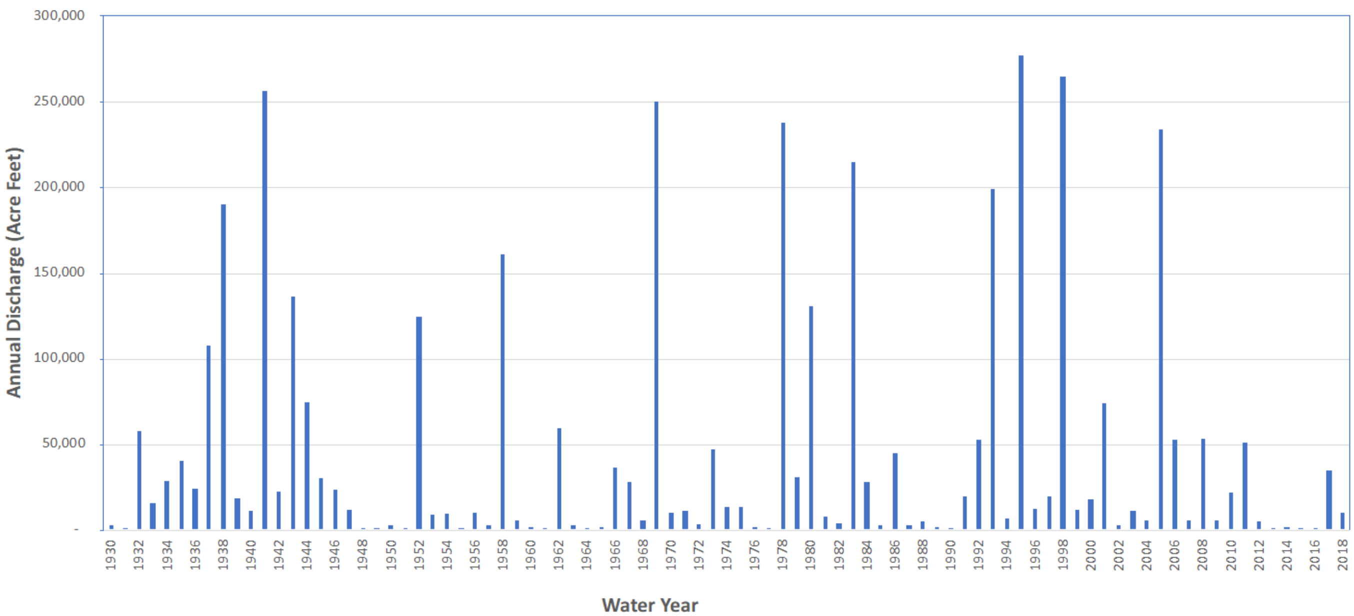

Figure 3.3 Time Series of Annual Discharge at USGS Gage 11118500 for WYs 1930-2018

Surface water from the Ventura River began to be diverted and stored in Lake Casitas in 1958. In order to detect whether the wet-weather diversions that started in 1958 measurably affected the distribution or magnitude of flows reaching Foster Park, plots of the cumulative percentiles for annual discharge for the whole time series (WYs 1930-2018) and the with-diversion time period (WYs 1959-2018) were compared. The results, on Figure 3.4, show very little effect of wet weather diversions to Lake Casitas on the distribution or magnitude of annual discharge. Frequency histograms of annual discharge, using bins of 8000 acre-feet, are compared for the 1930-2018 and 1959-2018 periods on Figure 3.5. They show similar patterns, with a quarter to one-third of WYs having annual discharge of 8,000 acre-feet or less, and a long tail of infrequent large WYs ranging up to 250,000 acre-feet. The cumulative percentile plots for both periods (Figure 3.3) show a common inflection point for annual discharge at about 59,000 acre-feet (representing the 84th percentile for the post 1959 time series). This value was used to distinguish "high flow" years during further analysis.

Although the analysis does not indicate that use of the whole available time record (WYs 1930-2018) would bias further analysis, intra-annual patterns of flow were evaluated using data for the period after diversions to Lake Casitas began (i.e., WY 1959 onward). Before evaluating long-term seasonal patterns in flow, the time series was divided into "high flow" years (i.e., the 10 years with annual discharge above 59,000 acre-feet; see Figure 3.3) and "all other" years. High flow years are identified in Table 3.5. Next, mean daily flows for all days in each month were pooled across all years in both time series, and summary statistics generated for each month (presented in Table 3.6). Box plots of monthly quartiles are provided on Figure 3.6.

In both normal and high flow years, mean daily discharge peaks in March. September and October have the lowest median mean daily flows in normal years, but October and November have the lowest median mean daily flow in high flow years. Mean daily flows of zero cfs have been recorded in every calendar month, except in the high flow years, in which flows of zero have occurred only within the months between October-January.

Table 3.5 Ten Years of Highest Annual Discharge Measured at USGS Gage 11118500 Between WYs 1959-2018

Water Year	Annual Discharge (acre-feet)
1962	59,100
1969	250,090
1978	237,333
1980	131,055
1983	214,770
1993	199,612
1995	277,103
1998	264,269
2001	73,897
2005	233,972

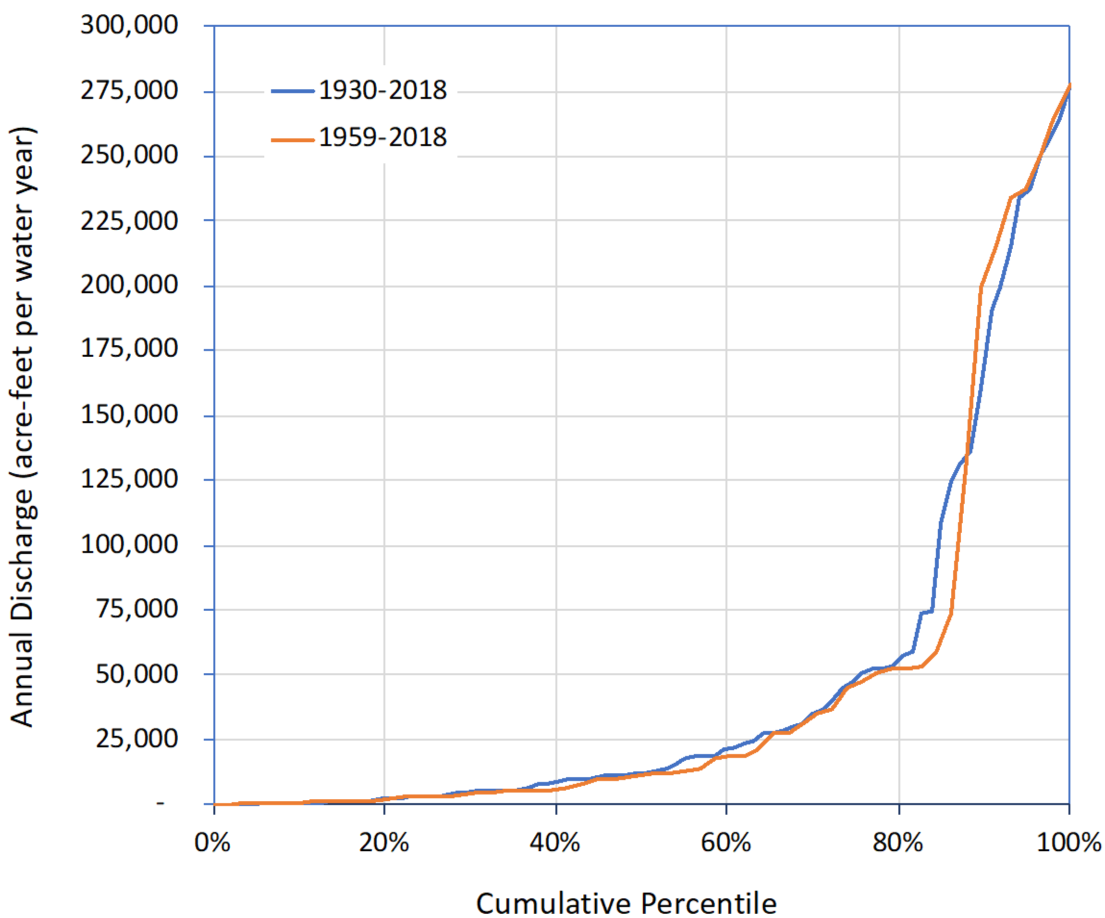

Figure 3.4 Cumulative Percentile Plots for Annual Discharge at USGS Gage 11118500 for WYs 1930-2018 (Blue Line) and WYs 1959-2018 (Orange Line)

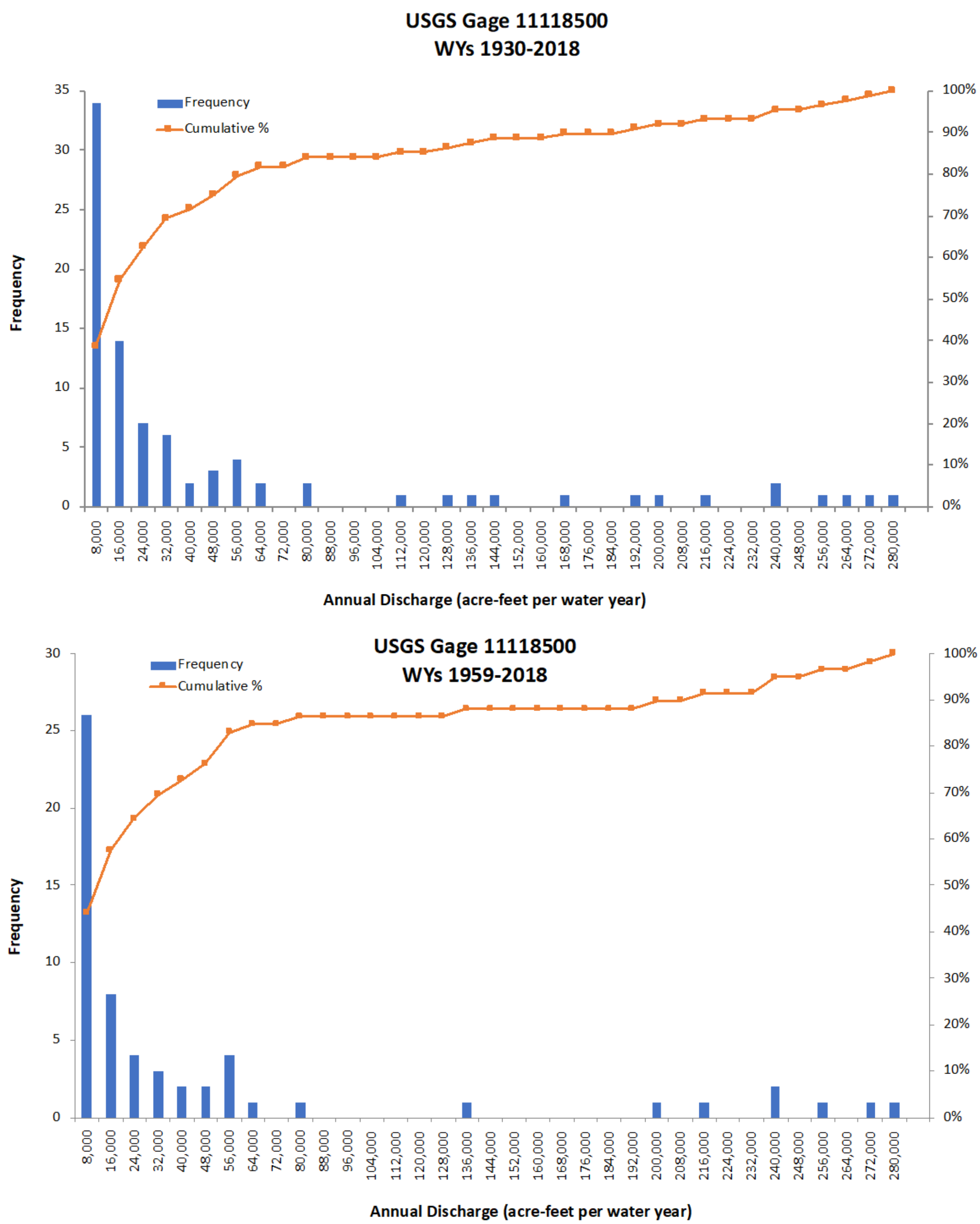

Figure 3.5 Frequency Histograms of Annual Discharge for WYs 1930-2018 (Upper Panel) and WYs 1959-2018 (Lower Panel) at USGS Gage 11118500 at Foster Park Bridge

Table 3.6 Summary Statistics by Month for Pooled Mean Daily Flows (cfs) for WYs 1959-2018 for USGS Gage 11118500

	Algae TMDL Wet Season							Algae TMDL Dry Season				
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
WYs 1959-2018 Without High Flow Years												
Mean	3.69	12.68	21.14	35.19	68.11	44.72	26.09	11.42	6.88	4.06	2.66	2.20
Minimum	0	0	0	0	0	0	0	0	0	0	0	0
25th Percentile	0	0	0.06	0.20	0.56	2.49	3.50	2.41	1.00	0.40	0.10	0
Median	0.7	0.93	2.5	4.3	7.25	12	9.83	5.7	3.7	2	0.92	0.2
75th Percentile	4.62	4.70	8.06	13.00	23.00	32.15	21.00	15.00	8.70	5.37	3.77	2.55
Maximum	340	4,060	5,160	6,340	8,670	6,270	5,930	172	50	38	21	387
High Flow Years Only												
Mean	1.79	1.18	34.35	692.98	1,219.25	825.81	268.54	128.11	55.54	28.52	14.67	11.24
Minimum	0	0	0	0	0.4	6.3	3.7	4.7	3	2.6	0.8	0.2
25th Percentile	0	0	0	3	83	162.5	82.75	44.25	27	17	9.8	6.9
Median	0.075	0.01	0.53	20	201	398.5	179	78	42.3	25.2	13	9.1
75th Percentile	1.26	0.5	1.48	318	809	846	411	149	67.2	35.1	20.5	15
Maximum	190	183	4,480	20,100	22,000	18,500	1,840	904	254	89	40	34

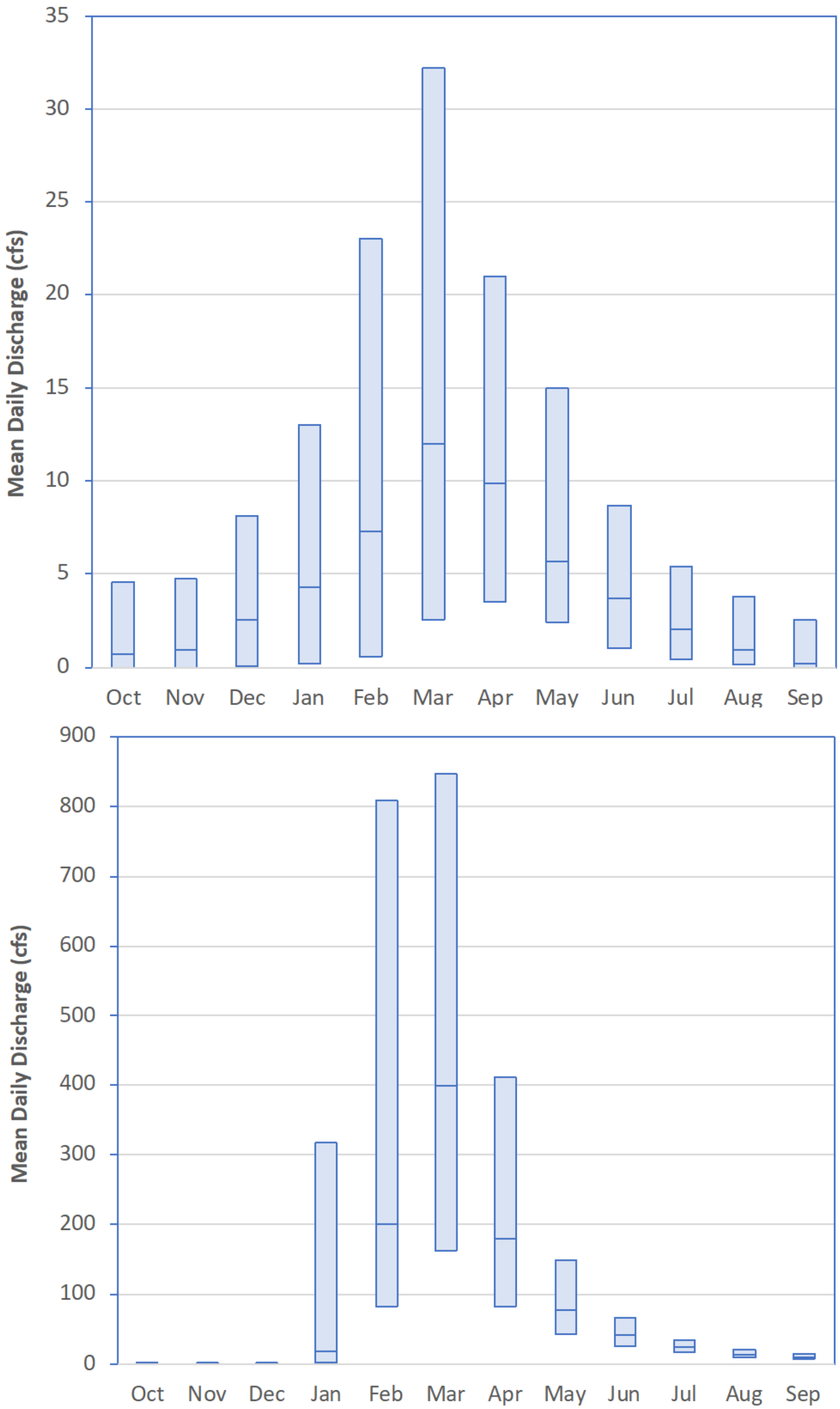

Figure 3.6 Quartiles of Mean Daily Discharge by Month for USGS Gage 11118500. Upper Panel - Pooled Data for WYs 1959-2018 Excluding the 10 High flow years; Lower Panel - Pooled Data for the Ten High Flow Years

3.5.1 Use of Historic Flow Data to Illustrate the OVSD Flow Subsidy

The monthly patterns of flow presented in Table 3.6 were used to illustrate how the "flow subsidy" from the OVSD discharge varies over the course of the year. The flow subsidy was characterized by computing the percentage of total flow in the river at the outfall that comes from effluent. Specifically, OVSD's 2013-2018 average mean daily effluent flow rate (2.391 cfs) was added to the long-term (1959-2018) median mean daily flow for each month measured at the top of Reach 3 (USGS gage at the Foster Park Bridge) to obtain 1) an estimated combined flow at the outfall, and 2) the percent thereof contributed by effluent.[26] Results are presented in Table 3.7. The table illustrates the seasonal variation in the magnitude of the flow subsidy. For example, in the majority of years (i.e., excluding the years with highest total annual discharge), OVSD contributes 92 percent or more of combined flow at the outfall on about half of days during the lowest flow month of September, and 17 percent or less of combined flow on about half of days during the highest flow month of February. Interestingly, at the beginning of very high flow WYs (i.e., in October, November, and December), OVSD's effluent contributes a higher percentage of combined flow than it usually does in those months. This likely reflects the late arrival of major storm systems during very wet years.

Table 3.7 Proportional Contribution of OVSD Effluent to Flows in the Lower Ventura River Based on Historic Flow Data Between 1959-2018

Month	1959-2018 Excluding Highest Flow Years			10 Years of Highest Annual Discharge(1)		
	Median Mean Daily Discharge at Foster Park Bridge (cfs)	Combined Discharge (OVSD + River) (cfs)	Percent of Combined Flow from Effluent	Median Mean Daily Discharge at Foster Park Bridge (cfs)	Combined Discharge (OVSD + River) (cfs)	Percent of Combined Flow from Effluent
Oct	0.70	3.09	77%	0.08	2.5	97%
Nov	0.93	3.32	72%	0.01	2.4	100%
Dec	2.50	4.89	49%	0.53	2.9	82%
Jan	4.30	6.69	36%	20.0	22.4	11%
Feb	7.26	9.65	25%	201.00	203.4	1%
Mar	12.00	14.39	17%	398.50	400.9	1%
Apr	9.83	12.22	20%	179.00	181.4	1%
May	5.70	8.04	29%	78.00	80.3	3%
Jun	3.70	6.04	39%	42.30	44.6	5%
Jul	2.00	4.34	54%	25.20	27.5	8%
Aug	0.92	3.26	72%	13.00	15.3	15%
Sep	0.20	2.54	92%	9.10	11.4	20%

Notes:
(1) The 10 WYs with the highest total annual discharge (acre-feet per WY) between 1959-2018 were 1962, 1969, 1978, 1980, 1983, 1993, 1995, 1998, 2001, and 2005.

[26] This approach assumes that base flow is not added to the river between Foster Park and the outfall. Whether or not this portion of the river is a gaining reach is an unsettled question based on publicly available monitoring data.

3.6 Discussion of Available Evidence for Non-Nutrient Related Contributions to Impairments Addressed by the TMDL

Distinct diurnal variations in pH and DO are typically observed in the lower Ventura River (and elsewhere in the watershed), indicating that submerged primary producer biomass (e.g., benthic algae or submerged macrophytes) strongly affects DO and pH in the river both above and below the OVSD discharge. Abiotic factors that could independently affect DO in the lower river include conductivity, flow (through depth and turbulence - which affect exchange with the atmosphere), and temperature. These factors are not unrelated; for example, low flows can boost water temperature. However, there is no reason for conductivity to vary in a systematic diel fashion in the lower Ventura River, and although water temperature can have a diel cycle, its expected pattern (warmer during the day, cooler at night) would cause diurnal patterns in DO concentrations opposite from those observed in the river because cooler water has a higher capacity to hold oxygen. Diurnal variation in flow can be caused by evapotranspiration of emergent vegetation, but the expected pattern (lower flows during midday, higher flows at night) would not cause pre-dawn DO minima and midday DO maxima.

3.6.1 Effects of Base Flow on Algae-Related Impairments

Although the diurnal variations in DO in the Ventura River indicate that submerged autotrophs (submerged photosynthetic macrophytes or algae) are exerting a strong influence on diurnal patterns of DO, higher base flow can mute the diurnal variations and elevate the average concentrations of DO upon which diurnal patterns are superimposed. The 2009 UCSB Report included use of regression analysis with field data from the Ventura River to develop empirical relationships that estimated the maximum benthic algal biomass (as chl.a/m^2) required to reduce pre-dawn DO to particular concentrations at different stream flows (Q). The equations were as follows:

- Equation 1: Chl.a = 186,000*Q/(minimum DO)$^{4.92}$.
- Equation 2: Chl.a = 4,900*Q$^{0.84}$/10$^{0.35(\text{minimum DO})}$.

In LWA (2009), these equations were used with percentiles of flow derived from the Foster Park gage to evaluate the levels of algal biomass that would be hypothetically be required in Reach 3 to generate pre-dawn exceedances of a DO threshold of 5 mg/L (which is much more lenient standard than the daily minimum DO that was eventually used for the TMDL). Flow values used in the equations were percentiles of pooled mean daily flows for the Foster Park gage for the May-September for WYs 1978-2008.[27] The analysis was repeated considering effluent flow from OVSD as a flow subsidy in the lower reaches of the river.[28] The results of the analysis are reproduced in Table 3.8. The table suggests that - provided mean daily flow between

[27] Mean daily flow (cfs) for all days May-September were pooled for a forty year period (1978-2008). Data for days when flow = 0 cfs at Foster Park were omitted from the analysis on the (reasonable) assumption that when the river channel is dry, algal targets are nonsensical. Then, overall mean and percentile daily flows were calculated.

[28] Annual mean daily flows of effluent discharge were calculated for 2000-2008, which yielded a grand mean daily flow of 3.4 cfs. Using an assumption that no flow is lost between Foster Park and the treatment plant discharge, the grand mean daily effluent flow was added to each daily flow record from Foster Park for May-September (for 1978-2008), and overall mean and percentile daily flows were re-calculated.

May-September is above ≥ 3 cfs - benthic algal biomass could theoretically exceed the TMDL threshold of 150 mg/m^2 chl.a without driving pre-dawn DO below 5 mg/L. When the OVSD discharge is added to Foster Park flows, the equations predict that benthic algal biomass would need to exceed 246 mg/m^2 chl.a to cause DO exceedances (below 5 mg/L) even at the lowest (10th) percentile flow. At least half of the time, flows observed at Foster Park between May-September would be sufficient to maintain pre-dawn DO above 5 mg/L even with benthic algal biomass as high as 400 mg/m^2 chl.a.

The percentiles of flow presented in Table 3.6 for individual months between May-September are lower than those used by LWA in their 2009 evaluation. The 2009 evaluation used a shorter time series of flow (1978-2008) than was used to generate the percentiles in Table 3.6 for individual months (1959-2018). The shorter time series used in 2009 contained eight of the ten "high flow" years defined using a time series for 1959-2019 (see Table 3.5). Use of flow percentiles from the relatively wetter period of 1978-2008 would yield higher estimates of acceptable benthic algal biomass. In addition, the TMDL assigned a much more stringent DO threshold (7 mg/L) than the one used in the 2009 exercise. Nevertheless, the exercise (which uses empirical relationships from the Ventura River) demonstrates the strong potential for flow to ameliorate diurnal excursions of DO caused by benthic algae.

The above discussion also needs to be placed in context with the TMDL-CMP monitoring results for 2015-2017. As shown in Tables 3.2-3.4, exceedances of the TMDL algal biomass target (applied as a *seasonal average*) have been accompanied by excursions of pre-dawn DO below the TMDL target of 7 mg/L. However, the river below the outfall frequently functions as a losing reach, so percentiles of flow at Foster Park would not reflect flow conditions further downstream. It would be interesting to use measured flows from the TMDL-CMP from Reaches 1-2, and the TMDL target of 7 mg/L, in the UCSB equations to predict tolerable maximum chl.a levels in the lower river.

3.6.2 Other Factors Influencing Algae-Related Impairments

Although not unexpected in a Mediterranean climate, extreme inter-annual variability in hydrology may exert as much, or more, control on algal related impairments in the Ventura River watershed than chronic nutrient loading from anthropogenic sources. In addition to affecting the timing and magnitude of algal blooms and nutrient pulses, variability between WYs in the size and duration of winter storms affects the availability of suitable substrate for algal colonization (through scouring) and the multi-year cycle in which aquatic plants, and then riparian shrubs, replace benthic algae as the dominant colonizers of the stream bed between large WYs. Other than an anecdotal report from the 2009 USCB Report (which stated that total and benthic chlorophyll-a levels in the Ventura River watershed were inversely related to riparian canopy cover in sampling conducted in September 2008) there are no data for the Ventura River to evaluate the extent to which algae-related impairments are influenced by riparian habitat and shade. None of the existing monitoring programs report canopy cover or other data regarding riparian habitat structure or channel shading.[29] Prolonged alterations of base flows and suspended sediment have occurred since the Thomas Fire burned most of the watershed in December 2017, and future perturbations of flows and suspended sediment transport are expected if the Matilija Dam is removed in the upper watershed. Because the TMDL CMP is a

[29] Field data forms appended to TMDL CMP reports indicate that spherical densiometers are used by field staff to collect the raw data required to calculate canopy cover, but canopy cover is not reported.

nascent monitoring program, it will take years of data to detect a signal in monitoring data from extreme hydrologic events.

Table 3.8 Threshold Chlorophyll-a Levels Predicted Using May-September Daily Flows at Foster Park, With and Without a Flow Subsidy From the Ojai Valley WWTP(1)

Percentile	Mean Daily Flow (cfs)	Threshold Benthic Algae Chlorophyll-a (mg/m²)			
		Predicted by UCSB Equation 1		Predicted by UCSB Equation 2	
		Using Flow at Foster Park Only	Including the Flow Subsidy from OVSD	Using Flow at Foster Park Only	Including the Flow Subsidy from OVSD
10th	0.6	42	246	56	279
20th	1.8	116	313	142	348
30th	3.0	185	377	216	411
40th	4.2	257	445	289	477
50th	6.3	379	565	409	588
60th	9.1	528	709	567	727
70th	14.6	823	994	831	1000
80th	23.2	1250	1421	1214	1362
90th	45.0	2334	2497	2133	2267
Mean	19.4	1002	1185	911	1089

Notes:
(1) Results reproduced from Table 7 in LWA (2009) *Comments Regarding the UCSB (2009) Numeric Target Recommendation Report.* TM submitted to OVSD, December 15, 2009.

There is evidence that non-algal factors are influencing DO levels in the estuary. The TMDL CMP DO logger data from the estuary consistently reveal a complex pattern that is not easily attributed to *in-situ* algal or macrophyte influence. First, daily DO maxima do not always occur near midday, as is true at the sites upstream in flowing water. DO maxima sometimes occur well after sundown, or even near midnight. Second, intra-day shifts in DO are superimposed over a longer-term pattern of gradually rising and falling mean daily DO such that excursions below the target are only observed during portions of the deployment in many of the quarterly deployments. A good example of this pattern is shown on Figure 3.7. This pattern suggests an influence of the lunar tidal cycle, perhaps related to changes in stage when the estuary berm was open. However, the larger-scale pattern has persisted at times (e.g., in September 2016) when the berm was reported to be closed. The anomalies in the estuary DO data have not been acknowledged in TMDL CMP reports. An effect of the spring/neap tide cycle on pH and DO was observed during continuous logger deployments in Mugu Lagoon during a Calleguas Creek Watershed Nitrogen TMDL Special Study.[30] Sufficient information remains unavailable to explain the causes or significance of the DO data from the estuary at this time.

[30] LWA (2008) Calleguas Creek Nitrogen Compounds and Related Effects TMDL; Results of Special Study on Type and Extent of Algae Impairments in Calleguas Creek and Mugu Lagoon. Submitted to the Los Angeles Regional Water Quality Control Board, July 16, 2008. 93 pp.

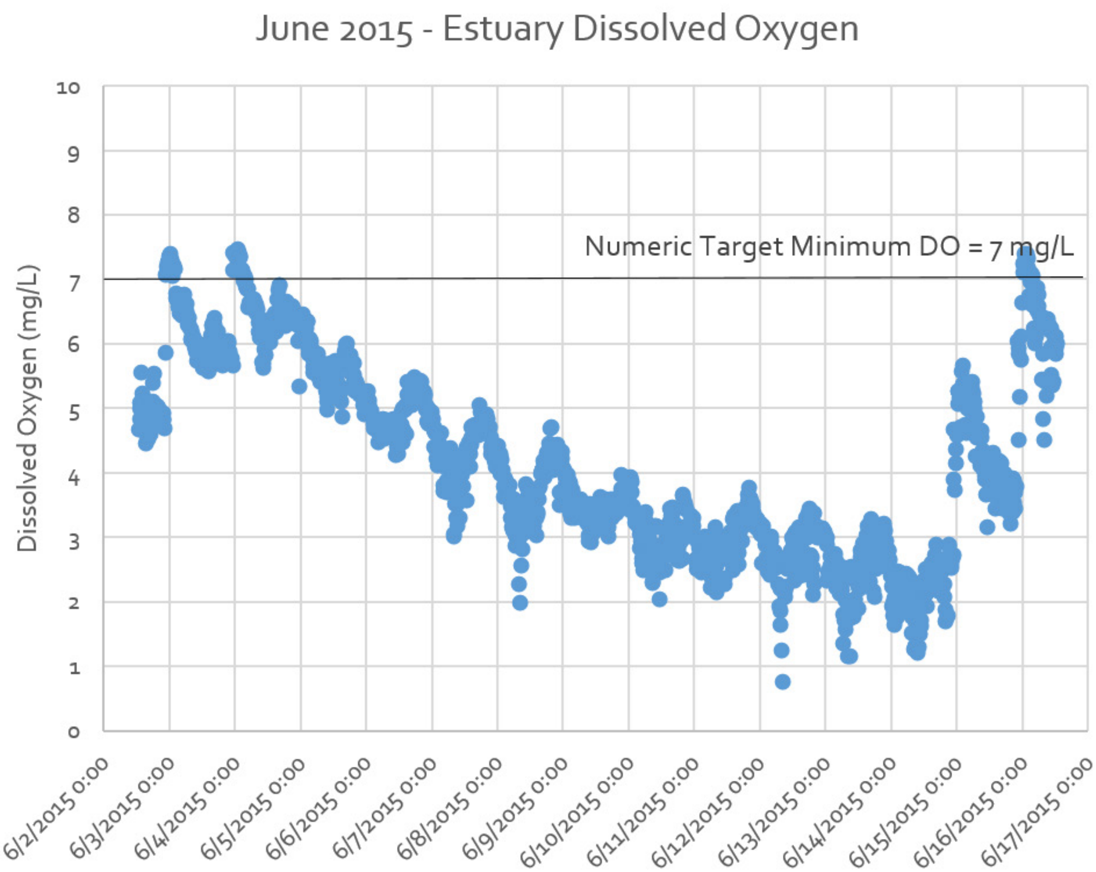

Figure 3.7 Continuous DO Data From the Estuary From the TMDL-CMP Logger Deployment in June 2015

3.7 Discussion of Potential Impacts of Different Forms of Nitrogen on Benthic Algae in the Lower River

Nationwide, the regulatory trend for addressing biostimulatory impairments is to regulate TN and TP. Consistent with this trend, the Algae TMDL established LAs for nitrogen based on TN. TN contains a variety of dissolved and particulate nitrogen forms, not all of which are readily available for uptake by benthic algae or aquatic macrophytes. Dissolved inorganic forms of nitrogen present in oxygenated surface waters are principally ammonium and nitrate (nitrite would only be present in highly reducing environments). There is really no debate that ammonium and nitrate can support algal and aquatic macrophyte growth as soon as they enter surface waters, although autotrophs have taxon-specific preferences and different specific uptake rates for nitrate and ammonium. Principal justifications for limiting discharges of particulate organic nitrogen forms rely on the assumption that particulate organic material in transport in a stream (such as leaf litter, other detritus, suspended algae) can eventually become available for algal or plant growth after participating in biological or biochemical processes. These processes involve time lags and could include 1) processing, ingestion, digestion and eventual excretion of N in particulate organic matter by stream biota (such as invertebrates and fish), 2) lysing of senescent or damaged plant or algal tissue, and 3) decomposition (oxidation) of dead organic matter by stream microbes. Inorganic nitrogen in the particulate fraction generally consists of ammonium or nitrate adsorbed to suspended sediment, e.g., on the ion exchange sites of clay particles. Concentration gradients in the water surrounding such particles will drive desorption of the ions, causing this fraction to serve as a reversible reserve of bioavailable dissolved inorganic nitrogen (DIN).

Spiraling length represents the distance over which the average nutrient atom travels in a river or stream as it completes one cycle of utilization from a dissolved available form, passes through one or more metabolic transformations, and is returned to a dissolved available form. The TMDL Staff Report credits a report by Tetratech for estimates of "nutrient uptake lengths" for TN and TP of 3.6 and 3.7 km, respectively, for the Ventura River. Nutrient uptake lengths are the average distance a nutrient molecule travels before being taken up by biota in a stream, and would be dependent on a variety of physical co-factors such as season, temperature, and flow. The reference for the uptake lengths is improperly cited in the TMDL Staff Report, and the presumably correct reference (found cited elsewhere)[31] is not available on-line. Consequently, the basis for Tetratech's estimated uptake lengths was not possible to review. For perspective, as can be seen from Figure 3.2, the distance between the OVSD outfall and the end of Reach 1 is about 8 km.

Uptake lengths are not the same as spiraling lengths; the latter include the total distance nutrient atoms travel ***before and after*** they are incorporated into organic matter, and ***until they are returned to a dissolved available forms*** after one or more transformations. In other words, uptake lengths would typically be shorter than spiraling lengths. No information is available for nutrient spiraling lengths for the Ventura River. Regardless, spiraling lengths are applicable to nutrient loads that enter the system as bioavailable (or biodegradable) forms. The fate and transport of nitrogen that arrives in surface waters bound to highly refractory compounds will be different.

Although simplistic, an initial evaluation of the bioavailability of TN in the river might start with examination of the amount of N in the following four compartments:

1. Particulate organic N (N in leaf litter, other suspended detritus, and phytoplankton).
2. Dissolved organic N (N in exudates and other organic molecules in suspension).
3. Particulate inorganic N (would consist of ammonium or nitrate adsorbed to suspended sediment).
4. Dissolved inorganic N (DIN; essentially nitrate and ammonium, with occasional trace contributions from nitrite and ammonia).

Some data are available to take a preliminary look at the bioavailability of TN in the lower Ventura River. This is illustrated in Table 3.9, which shows that each monitoring program with sites in the lower Ventura River reports a different suite of nitrogen forms, but only two of them report enough parameters to divide TN into DIN and organic N. None of the programs report parameters that would allow differentiation of organic N into particulate and dissolved fractions.

[31] Tetra Tech. 2012. Ventura River Estuary and Flow Conditions. Prepared for USEPA Region 9 and LARWQCB. June 30, 2012.

Table 3.9 Nitrogen Parameters Reported for Stream Water by Monitoring Entities With Established Sites in the Lower Ventura River

Monitoring Entity	Parameters Reported	Possible to Compute DIN?	Possible to Differentiate Organic-N and DIN?	Possible to Differentiate Particulate and Dissolved N?
Santa Barbara Channelkeeper	Nitrate	No	No	No
Algae TMDL CMP	TN, TDN, NO_3+NO_2, TKN, dissolved TKN	no, because NH_3-N is not reported	no, because NH_3-N is not reported	Yes, but not possible to differentiate the labile and refractory components of the fractions
OVSD	TN, Organic N, NH_3-N, NO_3+NO_2	Yes	Yes	No
VCSQMP	NH_3-N, NO_3+NO_2, TKN	Yes	Yes	No

Data for the nitrogen fractions reported by OVSD for quarterly water samples from receiving water monitoring sites above (site R-3, in Reach 3) and below (sites R-4 and R-5, in Reach 2) the discharge were compiled for the period February 2016- February 2018, and used to partition TN into DIN and organic-N. The results are provided in Table 3.10 and compared to an analogous breakdown for OVSD effluent. The data show a larger variability from month to month in the contributions of DIN and organic N to TN above the discharge compared to below the discharge. For example, above the discharge (at Site R-3) percent DIN varied from 0 percent (in May 2016) to 92 percent (in May 2017). Below the discharge (at Site R-4), percent DIN varied between 70 percent (in August 2016) to 86 percent (in February 2017). Without more information, it is not possible to speculate on how refractory the organic N fraction is in effluent or river water. However, the high percentages of TN accounted for by organic N in Reach 3 suggest that much of the TN naturally in transport above the OVSD outfall would require microbial processing before being eligible to contribute to algal or macrophyte growth.

Table 3.10 Nitrogen Fractions in OVSD Effluent and at Receiving Water Sites Monitoring by OVSD

Quarterly Event	Site(1)	TN (mg/L)	Organic-N (mg/L)	DIN (mg/L)	% Organic N	% DIN
January 2014-March 2018(2)	Effluent	4.7	1.07(2)	3.63	23%(3)	77%
February 2016	R-3	0.83	0.66	0.17	80%	20%
	R-4	4.6	0.99	3.71	22%	81%
	R-5	4.9	1.1	3.95	22%	81%
May 2016	R-3	0.53	0.53	0	100%	0%
	R-4	2.5	1.3	1.2	52%	48%
	R-5	2.7	1.1	1.6	41%	59%

Table 3.10 Nitrogen Fractions in OVSD Effluent and at Receiving Water Sites Monitoring by OVSD (continued)

Quarterly Event	Site(1)	TN (mg/L)	Organic-N (mg/L)	DIN (mg/L)	% Organic N	% DIN
August 2016	R-3	0.88	0.74	0.14	84%	16%
	R-4	4.7	1.4	3.3	30%	70%
	R-5	4.7	1.4	3.3	30%	70%
November 2016	R-3	1.0	0.8	0.13	86%	14%
	R-4	4.6	1.3	3.34	27%	73%
	R-5	4.6	1.4	3.2	30%	70%
February 2017	R-3	5.2	1.3	3.9	25%	75%
	R-4	4.2	0.6	3.6	15%	86%
	R-5	4.5	0.8	3.7	18%	82%
May 2017	R-3	3.7	0.29	3.41	8%	92%
	R-4	2.3	0.5	1.8	21%	78%
	R-5	3.5	0.66	2.8	19%	81%
August 2017	R-3	1.77	0.57	1.2	32%	68%
	R-4	1.97	0.57	1.4	29%	71%
	R-5	2.23	0.41	1.82	18%	82%
November 2017	R-3	1.7	1.1	0.64	63%	37%
	R-4	4.0	1.08	2.92	27%	73%
	R-5	4.1	1.28	2.82	31%	69%
February 2018	R-3	1.01	0.79	0.22	78%	22%
	R-4	3.5	0.97	2.5	28%	72%
	R-5	3.5	1.1	2.4	31%	69%

Notes:
(1) The location of the sampling locations are shown on Figure 3.1.
(2) Values were drawn from OVSD's 2018 Permit, Attachment F, and are for "Highest Average Monthly Discharge".
(3) TKN and Organic N were not reported for effluent by the Regional Board in the 2018 Permit Attachment F. Organic N is best estimated as TKN-NH_3-N. For this table, organic N was estimated as TN-DIN.

3.8 Summary of Key Points

A summary of key points raised in the five sections of the memorandum is provided in the following:

- TMDL Requirements and Assumptions:
  - The selection of a benthic algal biomass target for the Algae TMDL (150 mg/m^2 chl.a) drove the quantification of the required load reductions. The sequence of steps used by Regional Board staff to derive LAs was explained in detail.
  - The benthic algal biomass target is based on literature values that constitute subjective interpretations of how much stream algae is likely to impair recreational uses such as wading and trout fishing, and data sets that include few (usually none) southern California streams or streams from other Mediterranean climates. The target was not based on evidence linking levels of algal biomass to aquatic life

beneficial use impairment (such as low DO or alteration of benthic invertebrate assemblages).

  - A variety of other key assumptions were described that might or might not hold up if the Algae TMDL was re-opened in the future. Among the vulnerable assumptions are 1) that nutrient loading during wet weather does not contribute to algal-related impairments, 2) that existing loading to the estuary is not high enough to cause impairments of beneficial uses, and 3) that nitrate contributions from daylighting groundwater were correctly characterized. The reasons why these particular assumptions are vulnerable to revision in the future was provided in the text. In each case, if the assumption was discarded or revised, future estimates of assimilative capacity could be lower and more stringent load reductions might be required for dischargers.

- TMDL Monitoring Requirements and Results:
  - Diurnal variations in pH and DO are consistently observed in the river, above and below the discharge, and are strong evidence that submerged plants and algae are exerting an influence on DO.
  - Exceedances of the algal biomass and DO targets in the TMDL have been frequent in the lower Ventura River and the Estuary since compliance monitoring began in early 2015.
  - Flow measurements made by different entities consistently provide conflicting evidence regarding whether the river between Foster Park and the OVSD discharge is a gaining or losing reach. Resolution of this discrepancy will be important to correctly understand the nature of the flow subsidy from the OVSD discharge and the relationship between OVSD nutrient discharges and nutrient loads arriving from upstream.
  - Surface flow decreases between the OVSD outfall and the estuary in most months, but the extent to which the loss of surface flow represents direct evaporative losses, uptake and evapotranspiration by aquatic and riparian vegetation, and/or groundwater recharge in Reaches 1-2 is unknown. Understanding groundwater recharge may become important in the future if groundwater quality in the Lower Ventura River basin becomes an issue with the Regional Board.
  - In Spring 2017, significant inputs of water and nutrients occurred below the OVSD discharge that are unrelated to OVSD effluent. Although WY2017 had higher annual discharge than WYs 2015 and 2016, it was still a below-average WY. The source of the water and nutrients is unknown. Compliance monitoring is not designed to elucidate sources of water or nutrients below the OVSD discharge.
  - TN and TP loads are consistently lost from the water column in a non-conservative fashion in the lower river, providing evidence that biological uptake by *stationary* autotrophs (attached algae or rooted plants) are an important sink for nutrients.
  - There is empirical evidence that a non-algal nutrient sink may be important in Reach 2. This will be important to understand if DO exceedances in the estuary drive future changes to the TMDL.

- Existing Flow Conditions in the Lower River:
  - An 89-year record of mean daily flows for USGS Gage 11118500, located at the Foster Park Bridge (top of Reach 3), was used to characterize long-term average patterns of flow for entire WYs and calendar months.

- Wet weather diversions to Lake Casitas (starting in 1959) have had little apparent influence on the long-term patterns of annual discharge at Foster Park.
- The interannual patterns in flow were compared for time series of high flow years and all other years. Results were presented as summary statistics and as box plots of monthly quartiles. In both normal and high flow years, mean daily discharge peaks in March. September and October have the lowest median mean daily flows in normal years, but October and November have the lowest median mean daily flow in high flow years. Mean daily flows of zero cfs have been recorded in every calendar month, except in the high flow years, in which flows of zero have occurred only within the months from October-January.
- Based on long-term median mean daily flows for calendar months (excluding high flow years) the OVSD "flow subsidy" ranges from 17 percent (in March) to 92 percent (in September) of total estimated flow at the outfall. However, because days with zero flow are statistically possible during any month at Foster Park, the flow subsidy can intermittently be much higher.

- Evidence for Non-Nutrient Related Contributions to Impairments Addressed by the Algae TMDL:
  - Temperature, conductivity, and flow could all influence DO in the river, but they are not responsible for the strong diurnal variations in DO and pH that are characteristic for the river. Data that would allow evaluation of the effects of canopy cover or other riparian habitat characteristics on algal-related impairments are not being reported by monitoring entities.
  - Previous empirical relationships between benthic algae biomass, flow and diurnal DO minima from research conducted in the Ventura River in 2008-2009 were used with percentiles of flow at Foster Park to illustrate the extent to which base flow can mitigate DO impairment caused by benthic algae.
  - Extreme channel-changing hydrologic events and perturbations in watershed hydrology and water quality (e.g., from the Thomas Fire) may affect the timing and magnitude of algal blooms over time frames lasting several years. However, it will take many years of monitoring data to characterize the effect of infrequent events of this type and to understand how their effects compare to those of chronic anthropogenic nutrient discharges.
  - There are several lines of evidence that non-algal factors are influencing daily and monthly patterns of DO in the estuary. The lunar tidal cycle, and particularly the spring/neap tidal cycle, may be driving the timing, frequency, and severity of DO impairments in the estuary. Understanding this phenomenon will be important if TMDL target exceedance in the estuary are addressed in a future TMDL.
- Potential Impacts of Different Forms of Nitrogen on Benthic Algae in the Lower River:
  - The Algae TMDL assigned LAs based on TN and TP. TN contains a variety of dissolved and particulate nitrogen forms, which have different bioavailabilities. However, use of TN to address biostimulatory impairments is a nationwide regulatory trend. The ecological justification for this approach was described.
  - The ecological factors affecting the bioavailability of different nitrogen fractions were summarized, and the availability of monitoring data from the lower Ventura

River that can be leveraged to assess (in a crude way) the bioavailability of nitrogen forms was reviewed.

- Receiving water data from OVSD's required monitoring program was used to calculate contributions of organic N and DIN to TN in stream water from Reaches 2 and 3. The evaluation revealed high variability in the magnitude and percent organic N in stream water above the OVSD discharge. Frequent high percentages of organic N above the outfall suggest that much of the TN naturally in transport in the lower river would require microbial processing before being eligible to contribute to algal or macrophyte growth.

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 4
FUTURE REGULATORY REQUIREMENTS

REVISED FINAL | August 2020

LARRY WALKER ASSOCIATES

Science. Policy. Solutions.

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Facilities Master Plan

Technical Memorandum 4
FUTURE REGULATORY REQUIREMENTS

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FUTURE REGULATORY REQUIREMENTS

4.1 Introduction and Purpose

Development of the 20-year Facilities Plan for Ojai Valley Sanitary District (OVSD's) wastewater treatment plant (WWTP) includes "a review of current and future regulations that might impact the operation of the treatment plant, identifying which effluent quality parameters might be impacted and when such regulations may be implemented." Current and reasonably anticipated future regulatory requirements stemming from actions by United States Environmental Protection Agency (USEPA), the State Water Resources Control Board (State Board), and the Los Angeles Regional Water Quality Control Board (Regional Board) were evaluated – with the exception of those that have already been implemented as effluent limitations in OVSD's 2018 National Pollutant Discharge Elimination System (NPDES) permit. Although a wide variety of water quality standards (WQS) and policies are discussed herein, the technical or regulatory merits of the standards and policies are not reviewed in the memorandum, and speculation about potential future revisions or interpretations of the requirements by State Board or Regional Board staff is beyond the scope of this review and is not provided.

The memorandum addresses the implications of three categories of potential or upcoming requirements:

- Potential adoption of new and updated USEPA water quality criteria in the Los Angeles Region Basin Plan (Basin Plan).
- Requirements initiated at the State Board or Regional Board level.
- Future regulation of surface flows in the Ventura River.

The memorandum does not address changes to OVSD's permit limits that would apply in a recycled water permit. Many of the WQS that currently apply to OVSD's discharge are more lenient than those that apply to recycled water, including groundwater recharge and other avenues for indirect potable re-use (such as surface water augmentation).

4.2 Summary of Principal Conclusions

Based on current effluent and receiving water quality, changes to OVSD's permit limits during the Facilities Plan planning period are most likely to occur based on the following three factors:

1. Potential incorporation in the Region 4 Basin Plan of new USEPA human health criteria would trigger reasonable potential, and a need for numeric effluent limits, for seven constituents that are not currently assigned limits in OVSD's NPDES permit. The seven constituents are Dioxin, Benzo(a)pyrene, Benzo(b)flouranthene, Bis(2-ethylhexyl)-Phthalate, Dibenzo(a,h)Anthracene, Dichlorobromomethane, and Indeno(1,2,3-cd)Pyrene. It is not yet known what effluent concentration would be applied.

2. Adoption of new, more stringent, aquatic life criteria for ammonia and selenium into the Region 4 Basin Plan would result in revised permit limits for OVSD, but are unlikely to pose compliance problems.
3. A re-opened Algae total maximum daily loads (TMDLs) or a new Benthic Community Effects TMDL for reaches below OVSD's discharge could result in a reevaluation of OVSD's effluent limits for total nitrogen (TN) and total phosphorus (TP). The potential arises from ongoing exceedances of the numeric targets in the Algae TMDL in the reaches below the OVSD discharge and potential new statewide impairment thresholds for TN and TP (lower than concentrations used for modelling in the Algae TMDL) that may be included in the State Board's upcoming Biostimulatory Substances Amendment to the Water Quality Control Plan for Inland Surface Waters, Enclosed Bays, and Estuaries of California (ISWEBE Plan). However, it is currently unknown how the amendment will be implemented for specific water bodies or publicly owned treatment works (POTWs) in general, and what regulatory off-ramps might be provided to dischargers.

Other more stringent new or updated water quality criteria promulgated by the USEPA or the State Board that could be adopted in Region 4 in the next few years appear to be comfortably met at OVSD's receiving water monitoring stations and in OVSD effluent. Barring changes in effluent and receiving water quality, it is not likely that these other new standards will result in effluent limits for OVSD.

OVSD will need to track whether use of the new Test of Significant Toxicity (TST) test statistic for toxicity tests leads to future exceedances in effluent or receiving water, causing potential for 303(d) listings for toxicity in Reaches 1 or 2, and potentially expand the geographic scope of the expected toxicity TMDL for Reach 3.

OVSD comfortably meets its effluent limits for salt constituents (total dissolved salt (TDS), chloride, sulfate, boron), and receiving water below the outfall comfortably meets the Basin Plan surface water objectives for salt constituents that apply in Reach 2 (there are no surface water objectives for salts that apply to Reach 1). Based on current receiving water quality, 303(d) listings and a TMDL for salts in Reach 2 would not occur unless surface water objectives are changed through a Basin Plan Amendment. A re-evaluation of OVSD's permit limits for salts that was based on protection of *groundwater* quality would likely be accomplished through the salt and nutrient planning process in the Recycled Water Policy, which would be preceded by studies and stakeholder processes, and would also require a Basin Plan Amendment.

Three parallel regulatory processes are underway that directly or indirectly address surface flows in the Ventura River. Regulation of surface flows could affect the ability of OVSD to divert effluent to re-use. Guesswork about whether OVSD's re-use prospects would be positively or negatively affected by these developments is extremely speculative at this time. Some of the rule-making affecting surface flows could occur before the end of the Facilities Plan planning period. A key study by California Department of Fish and Wildlife (CDFW), designed to provide guidance to the State Board on flows required to support Southern California Steelhead habitat and life cycles, may provide the earliest clues about the future status of surface flows in the lower Ventura River.

4.3 Background

The Ventura River Watershed is located in the northwestern portion of Ventura County with a small portion in southeastern Santa Barbara County (Figure 4.1). The watershed drains a fan-shaped area of about 220 square miles ranging in elevation from 6,000 feet to sea level. The Ventura River has several major tributaries, including Matilija Creek, North Fork Matilija Creek, San Antonio Creek, Coyote Creek and Cañada Larga. Approximately 85 percent of the watershed is classified as open space and approximately one half of the watershed lies within the Los Padres National Forest. Agricultural land use occupies about 4.5 percent of the watershed area (LWA 2015)[1]. Urban areas in the watershed include the Cities of Ojai and Ventura, and the communities of Casitas Springs, Foster Park, Oak View, Valley Vista, Mira Monte, Meiners Oaks, Upper Ojai and Live Oak Acres within unincorporated areas of the County of Ventura. High density and low density residential land uses account for 1.9 and 2.9 percent of the watershed area, respectively. Oil production and mining are the predominant industrial land uses and account for 2.1 percent of the watershed area (LWA 2015). The remaining land uses (public facilities, recreation, commercial, education institutions, horse ranch/livestock, transportation, and mixed urban) each account for less than 1 percent of the land use within the watershed. Three state highways (Highways 101, 33, and 150) traverse the watershed.

The Basin Plan assigns beneficial uses (and in some cases, site-specific water quality objectives) to water bodies on the basis of regulatory "reaches". The main stem of the Ventura River is divided into five reaches, shown on Figure 4.1. The estuary at the base of the watershed is regulated somewhat differently than the main stem reaches. OVSD's outfall is situated at the upper end of Reach 2. Consequently, OVSD can be considered a potential contributor to water quality impairments only in Reaches 1-2, and the Ventura River Estuary.

The Regional Board implements regional, state-wide, and USEPA water quality regulations that apply to discharges to surface and ground waters in Region 4 by issuing NPDES permits, waste discharge requirements (WDRs), NPDES permit waivers, and other regulatory devices, depending on the type of discharge. In addition, the Regional Board is responsible for addressing water quality impairments that are related to pollutants by adopting TMDLs which (among other requirements) assign permissible loads of pollutants ("allocations") to individual dischargers or categories of dischargers. TMDL load allocations are implemented as numeric effluent limits for POTWs.

NPDES permits for POTWs are issued on a five year basis. The Regional Board adopted the most recent NPDES permit for OVSD in December 2018 (Order No. R4-2018-0170). The numeric effluent limits contained in the 2018 permit are listed in Table 4.1. Of these, only the effluent limits for TN and TP are related to a TMDL. The TN and TP limits are derived from load allocations assigned to OVSD in the TMDLs for Algae, Eutrophic Conditions, and Nutrients in the Ventura River and its Tributaries (Algae TMDL) that was adopted by the Regional Board in December 2012, and which became effective in July 2013. The development of the requirements from the Algae TMDL are discussed in detail in Technical Memorandum (TM) 3, Total Maximum Daily Load Requirements.

[1] Larry Walker Associates (LWA) (2015) TMDL for Algae, Eutrophic Conditions, and Nutrients in the Ventura River and its Tributaries, Draft Implementation Plan. Prepared by Larry Walker Associates for County of Ventura, Ventura County Watershed Protection District, City of Ojai, City of Ventura, and Caltrans. June 29, 2015.

The effluent limits for TN in Table 4.1 are "final" limits that apply twelve years after the effective date of the Algae TMDL. In order to provide OVSD with time to modify its treatment processes to meet the TN final limits, an interim concentration-based effluent limit for TN of 7.6 milligrams per liter (mg/L) was assigned in the TMDL, and incorporated into the 2018 permit. The interim limit will remain in effect until the final TMDL-based TN effluent limitations become effective in July 2025.

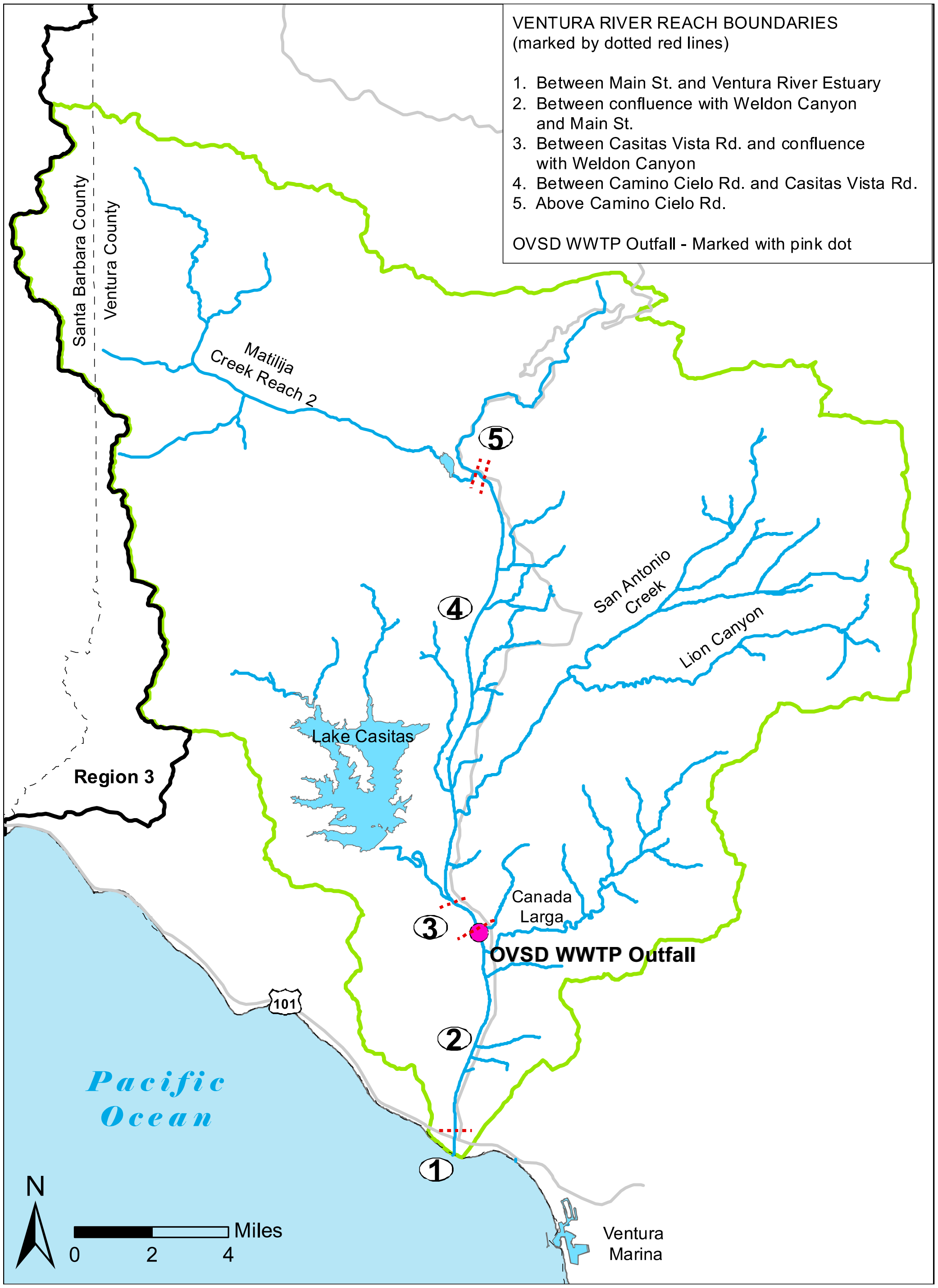

Figure 4.1 Reach Designations for the Main Stem of the Ventura River

Table 4.1 Numeric Effluent Limits in the OVSD's 2018 NPDES Permit

Parameter	Units	Numeric Effluent Limitations					
		Average Monthly	Average Weekly	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum	Average Seasonal
Biochemical Oxygen Demand ($BOD_5/20^{o}C$)	mg/L	10	-	15	-	-	-
	lbs/day	250	-	380	-	-	-
Total Suspended Solids (TSS)	mg/L	10	-	15	-	-	-
	lbs/day	250	-	380	-	-	-
Turbidity	NTU	2 (Ave. Daily)	-	5	-	10	-
pH	standard un4its	-	-	-	6.5	8.5	-
Temperature	Degrees F	-	-	86	-	-	-
Combined Radium-226, 228	pCi/L	5					
Gross Alpha Particle Activity (excluding radon and uranium)	pCi/L	15					
Uranium	pCi/L	20					
Gross Beta/Photon Emitters	millirem/yr	4					
Strontium-90	pCi/L	8					
Tritium	pCi/L	20,000					
Total Coliform	MPN or CFU/100 ml	23	2.2	240			
Removal Efficiency for BOD	%	≤ 85					
Removal Efficiency for TSS	%	≤ 85					
Oil and Grease	mg/L	10		15			
	lbs/day	250		380		EE	
Settleable Solids	ml/L	0.1		0.2			
Total Residual Chlorine	mg/L			0.1			
Total Dissolved Solids (TDS)	mg/L	1,500					
	lbs/day	38,000					
Sulfate	mg/L	500					
	lbs/day	13,000					
Chloride	mg/L	300					
	lbs/day	7,500					
Boron	mg/L	1.5					
	lbs/day	38.0					
MBAS	mg/L	0.5					
	lbs/day	13.0					
Selenium	µg/L	3.4		9.2			
	lbs/day	0.09		0.23			
Ammonia-N	mg/L	1.9		4.6			
	lbs/day	48		120			
Nitrate+Nitrite (as N)	mg/L			10			
	lbs/day			251			
Nitrite (as N)	mg/L			1			
	lbs/day			25			
Total Phosphorus (wet weather)	mg/L			2.6			
Total Phosphorus (dry weather)	lbs/dry weather						5,799
Total Nitrogen (summer season)	lbs/season						8,044
Total Nitrogen (winter season)	mg/L	4.6					
Chronic Toxicity	Pass or Fail, % Effect (TST)	Pass		Pass or Fail % Effect <50			

Notes:
Abbreviations: lbs/day=pounds per day; NTU=nephelometric turbidity unit; F=Fahrenheit; TSS=total suspended solids; pCi/L=picoCuries per liter; MPN=most probable number; CFU=colony forming units; ml/L=milliliter per liter; µg/L=micrograms per liter.

4.4 Potential Adoption of New and Updated USEPA Water Quality Criteria in the Basin Plan

4.4.1 Background

On August 5, 2015, the USEPA (hereinafter, Environmental Protection Agency (EPA)) issued a final rule updating six key areas of the federal WQS regulation which helps implement the Clean Water Act (CWA). The final rule was published in the Federal Register on August 21, 2015 (80 FR 51019). The results are included in 40 CFR 131. The following key program areas were addressed in the final rule: 1) the EPA Administrator's determinations that new or revised WQS are necessary, 2) designated uses for water bodies, 3) triennial reviews of state and tribal WQS, 4) anti-degradation requirements, 5) WQS variances, and 6) provisions authorizing the use of schedules of compliance for water quality-based effluent limits (WQBELs) in NPDES permits. The previous regulation had been in place since 1983.

Among other changes, the final rule amends 40 CFR 131.20(a) to clarify the "applicable water quality standards" that must be reviewed by states or tribes triennially. The final rule also requires that if a state or tribe chooses *not* to adopt new or revised criteria for any parameters for which EPA has published new or updated criteria recommendations under CWA section 304(a), they must explain their decision when reporting the results of their triennial review to EPA under CWA section 303(c)(1) and 40 CFR 131.20(c).

As part of this process, the EPA assembled and published on-line tables of the recommended aquatic life and human health criteria that have been published by the EPA since May 30, 2000. These constitute the standards (in addition to future updated criteria) that states are now required to evaluate as part of their internal triennial reviews. The cut-off date essentially coincides with the promulgation of the California Toxics Rule (CTR) on May 18, 2000. The majority of the constituents for which new or updated federal recommended standards were published since 2000 are priority pollutants that were previously addressed in the CTR. At this point in time, unless the EPA revises the CTR to reflect its updated criteria for priority pollutants, it appears that new EPA criteria for priority pollutants incorporated into regional Basin Plans in California would supersede CTR criteria for many constituents. This outcome would require that EPA approve the Basin Plan changes and depromulgate the CTR criteria.

In May 2018, the Regional Board held a public hearing to consider their priorities for their next (2017-2019) triennial review. In the staff report that accompanied this agenda item,[2] Regional Board staff signaled their intent to move forward with evaluation of the new and revised EPA criteria in the following language:

> *"Accordingly, the main focus of the 2017-2019 triennial review will be the consideration of these CWA section 304(a) recommended criteria for incorporation into the Los Angeles Water Board's Basin Plan. This process will involve evaluating which of the new or revised criteria to consider for adoption and incorporation into the Basin Plan. Where an update or adoption is not recommended, the reasons for this determination will be documented. Following these determinations, staff will proceed with the water quality objective updates. This effort is expected to form the bulk of basin planning work conducted during the 2017-2019 triennial review period. Stakeholders will have*

[2] LARWQB (2018) 2017-2018 Triennial Review: Consideration and Selection of Basin Planning Priority Projects, Revised Draft Staff Report, April 26, 2018.

> *the opportunity to comment on the initial determinations, as well as each of the updates or additions prior to its consideration by the Los Angeles Water Board as part of the public notice and comment process for each individual Basin Plan amendment.*" (LARWQCB 2018, p. 19)

At the hearing, the Regional Board adopted a resolution that (among other projects) prioritized the evaluation of the new and revised CWA section 304(a) recommended water quality criteria for potential incorporation into the Region 4 Basin Plan.

4.4.1.1 General Implication of Updated and New EPA Criteria for Region 4 Dischargers

If adopted by the Regional Board, many of the new or updated EPA criteria would become the most stringent criteria for freshwater bodies in Region 4. In other cases, an existing aquatic life criterion (e.g., one already in the CTR or the Basin Plan), or an existing California "Title 22" drinking water Maximum Concentration Limit (MCL), would remain the most stringent available criterion for a particular contaminant. MCLs are not necessarily applied as applicable WQS when NPDES permits are written. This is described more fully below.

In addition to new and revised federal recommendations for *priority* pollutants, the EPA has published new standards for a number of constituents designated as *non-priority*. By definition, these constituents were not previously addressed in the CTR. However, several of these constituents have no corresponding California (or federal) MCL, so if adopted in the Region 4 Basin Plan (or by State Board action), they would be new water quality objectives for Region 4 with no precedent.

4.4.2 Summary of the New and Updated EPA Water Quality Criteria

The vast majority of the updated federal standards that will be under consideration by the Regional Board are revised (or first-time) *human health* criteria published by the EPA in 2015 for 94 constituents. Updates to federal human health criteria that occurred earlier (i.e., between 2000-2015) were 1) a human health criterion for methylmercury published in 2001 (for consumption of organisms only); 2) human health criteria for nitrosodibutylamine, nitrosodiethylamine, and nitrosopyrrolidine published in 2002 (also for consumption of organisms only); and 3) human health criteria for thallium (for both consumption of water + organism and organism only) published in 2003. With the exception of thallium, the other pre-2015 federal standards listed above are for constituents not previously addressed by the CTR. In addition, four constituents that the EPA has now categorized as Priority Pollutants were assigned human health criteria for the first time in 2015 – there are no corresponding CTR criteria for these constituents either.[3] In addition, in the post-CTR era, the EPA published revised *aquatic life criteria* for four priority pollutants already covered by the CTR (cadmium, in 2001; copper in 2007; ammonia in 2013; and selenium in 2016). Finally, in four cases, the EPA has published aquatic life criteria to constituents for which there were no pre-existing standards, although they are all classified as non-priority pollutants. [4]

Overall, 93 of the new or revised federal standards published since May 2000 are more stringent than pre-existing criteria in either, or both, the CTR and California MCLs – or are new standards with no precedent. Most of the revised human health criteria are significantly lower than existing human health criteria in the CTR – in many cases lower by several orders of magnitude. One

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[3] 1,1,1-Trichloroethane, 1,2,4-Trichlorobenzene, 3-Methyl 4-Chlorophenol, Chloroform.

[4] Carbaryl in 2012; Diazinon and Nonylphenol in 2005; and Tributyltin in 2004

driver for this change was a decision by the EPA to revise the exposure factors upward by increasing the default per-capita daily drinking water rate and the default per-capita daily fish consumption rate. However, the EPA also utilized an updated toxicity database of noncarcinogenic and carcinogenic effects.

In Appendix 4A, the new and updated EPA criteria for priority pollutants that were published by the EPA following the promulgation of the CTR (i.e., since May 30, 2000) are compared to the existing water quality criteria drawn from the CTR or California MCLs. As noted above, in 2015, the EPA published aquatic life and human health criteria for a number of *non*-priority pollutants – it is not clear whether California Regional Boards will also be considering those for incorporation as new Basin Plan objectives. These criteria are provided in Appendix 4B.

The appendices do not include a comparison of existing criteria and revised EPA standards for ammonium and selenium. The derivation of water column concentration standards for these two constituents depends on site-specific factors that require more explanation. In addition, the 2018 OVSD NPDES permit contains numeric effluent limits for both ammonia and selenium (which is not true for the vast majority of priority pollutants), so the eventual decisions by the State Board and Region 4 regarding adoption of revised EPA standards for these two constituents may be impactful for OVSD. Consequently, separate discussions of ammonia and selenium standards are provided below.

4.4.3 Ammonia Criteria Revision

The freshwater ammonia criteria in the Region 4 Basin Plan are based on the EPA 1999 Update of Ambient Water Quality Criteria for Ammonia (EPA-822-R-99-014, December 1999). The *acute* aquatic life criterion (one-hour average concentration) varies with pH; specific concentrations are listed in Table 3-1 of the Region 4 Basin Plan for pH ranging from 6.5-9.0. In addition, different values for the acute criterion apply depending on whether or not a water body is assigned the beneficial uses COLD and/or MIGR. The *chronic* aquatic life criterion (30-day average concentration) is both pH and temperature dependent, and also dependent on whether the "early life stages" of fish are present. Tables 3-2 and 3-3 in the Region 4 Basin Plan are matrices of specific criterion values for combinations of pH (ranging from 6.5-9.0) and temperature (ranging up to 30 degrees C); Table 3-2 is used when early life stages of fish are present, and Table 3-3 is used when early life stages are absent. Regulators have the option of selecting criteria values from the tables (e.g., by rounding off effluent or receiving water values to those in the tables), or using more precise pH and temperature values in formulae provided in the Basin Plan.

The 1999, the EPA recommended aquatic life criteria for ammonia were based on the most sensitive endpoints known at the time: the acute criterion was based primarily on effects on salmonids (where present) or other fish, and the chronic criterion was based primarily on reproductive effects on the benthic invertebrate *Hyalella* or on survival and growth of the early life stages of fish (when present), depending on temperature and season.

In 2004, the EPA published a Federal Register Notice indicating its intent to re-evaluate the freshwater ammonia criteria based primarily on new information suggesting that mussels in the family Unionidae ('unionid mussels') were highly sensitive to ammonia. In their subsequent updated dataset, freshwater bivalve mollusks and snails were the predominant groups of genera in the most sensitive quartile of genus mean acute values, and freshwater mussels were two of the four most sensitive species in the chronic dataset. Ultimately, in 2009, the EPA published *draft* revised criteria that recommended two sets of acute and chronic criteria, applying to waters with or without freshwater unionid mussels. Significantly, the EPA noted in the 2009 draft ammonia criteria document that available data indicated that another freshwater mollusk taxon, non-pulmonate (gill-bearing) snails, are also sensitive to the effects of ammonia.

In its eventual final 2013 update (EPA 822-R-18-002, April 2013), the EPA ended up recommending that a single national acute and a single national chronic criterion be applied to all waters, rather than different criteria based on the presence or absence of freshwater mussels. This was based on their reasoning that 1) while unionid mussel species are not prevalent in some waters, such as in the arid west, non-pulmonate snails are broadly distributed across the United States; 2) freshwater unionid mussels are among the most sensitive genera in the dataset; and 3) that all states have at least one freshwater unionid mussel or bivalve mollusk, or non-pulmonate snail species, native or present in at least some of their waters.

The current Region 4 Basin Plan and 2013 EPA ammonia criteria are compared in Table 4.2. The Basin Plan acute criterion for waters with the COLD and/or MIGR beneficial uses are provided, because those apply to the Ventura River. At representative pH 7 and temperature 20 degrees Celsius (C), the 2013 acute criterion is 1.4-fold lower than the 1999 acute criterion, and the 2013 chronic criterion is 2.4-fold lower than the 1999 chronic criterion. Site-specific objectives for the Ventura River would depend on the representative pH and temperature values selected.

<u>It's important to note that the values in Table 4.2 are not equivalent to effluent permit limits that would be assigned to OVSD in either case.</u> For derivation of effluent limits for ammonia in NPDES permits, Regional Board staff use effluent-specific values for pH and temperature to derive acute and chronic objectives, which are in turn subjected to a series of adjustment factors (related to variability and frequency of effluent data) to calculate effluent limits. This is discussed more fully below in Section 4.4.5.1.

Table 4.2 Comparison of Current Freshwater Ammonia Criteria in the Basin Plan with the Recommended Criteria in the 2013 EPA Update at pH 7.0 and 20 degrees C

Acute Criterion (mg/L)		Chronic Criterion (mg/L)	
Basin Plan for Waters Designated COLD and/or MIGR	2013 Final EPA Update *Oncorhynchus* spp. Present	Basin Plan for "Early Life Stages Present"	2013 Final EPA Update
24.1	17	4.15	1.9

As part of the 2013 Update, the EPA published guidelines providing flexibility for states to develop site-specific objectives for ammonia when the aquatic fauna present in a water body do not align with the fauna in the acute and chronic datasets underlying the updated criteria.[5] For cases in which a state can demonstrate that mussels are not present on a site-specific basis, the EPA specified a Recalculation Procedure to remove the mussel species from the national criteria dataset to better represent the species present at the site. Starting with the draft update in

[5] Flexibilities for States Applying EPA's Ammonia Criteria Recommendations, EPA-820-F-13-001, April 2013.

2009, there has been much debate within the regulated community regarding whether unionid mussels are present in one location or another in California, and some regional studies have been undertaken. The issue is currently unsettled in Region 4.

4.4.4 Selenium Criteria Revisions

The current selenium criterion that applies in Region 4 is the chronic aquatic life criterion from the 2000 CTR; the CTR does not contain a freshwater acute criterion nor human health criteria for selenium. The EPA published a new national chronic aquatic life selenium criterion in 2016 (EPA-822-R-16-006, June 2016), after a long process that began with publication of first draft updated criteria in 2004. Instead of a single default water column value, the 2016 update introduced four "elements": 1) a fish egg-ovary element; 2) a fish whole-body and/or muscle element; 3) a water column element (one value for lentic and one value for lotic aquatic systems); and 4) a water column "intermittent" element to account for potential chronic effects from short-term exposures (one value for lentic and one value for lotic aquatic systems). The intermittent exposure scenario was developed by the EPA to address short-term exposures that contribute to chronic effects through selenium bioaccumulation (e.g., releases from storage ponds or other intermittent releases). The EPA derived the values for the water-column criterion elements from the egg-ovary element by assessing food-chain bioaccumulation based on available data collected at lentic and lotic systems in the continental United States. Thus, all four criterion elements are based on reproductive effects in freshwater fish. The EPA recommended that states and tribes adopt all four elements as WQS.

Subsequently, in 2018, in response to a lawsuit and a consent decree, the EPA published proposed selenium criteria for Aquatic Life and Aquatic-Dependent Wildlife *for California freshwaters*. The California-specific proposed criteria are comprised of "criterion elements" of fish tissue and bird eggs, and a performance-based approach for translating the bird egg and fish tissue elements into site-specific water column criteria. Specifically, for California, the EPA altered its recommended 2016 national selenium criteria for freshwater with 1) the addition of a bird egg criterion element, and 2) the replacement of the 2016 selenium monthly average exposure water column criterion elements with a performance-based approach. California regulators would have to translate the tissue criterion elements in the 2018 proposed rule into site-specific water column concentrations using one of the following two approaches:

1. A mechanistic model approach in which trophic transfer factors, enrichment factors, and conversion factors (from the EPA documentation or other literature or research) are used with fish tissue or bird egg data in proscribed formulae to derive target water column values, or
2. An empirical approach in which co-located field datasets for water column concentrations and tissue concentrations are used to derive bioaccumulation factors (BAFs) for one tissue "element" or another, which are then used as the denominators in ratios with the published "tissue element" criteria to derive target water column values.

The selenium criteria that currently apply in Region 4, the 2016 national, and 2018 draft California criteria are compared in Table 4.3.

Table 4.3 Comparison of Selenium Freshwater Aquatic Life Criteria Currently Applicable in Region 4 With Those Proposed by EPA for California

Criterion	Tissue (mg/kg dry weight)				Water Column (µg/L)		
	Bird Egg	Fish			Chronic		Short Term/ Intermittent Exposure[1]
		Egg-Ovary	Muscle	Whole Body	Lentic	Lotic	
Current Region 4 (based on CTR)	N/A	N/A	N/A	N/A	5 (4-day average)	5 (4-day average)	N/A
2016 National Recommended Criteria	N/A	15.1	11.3	8.5	1.5 (30-day average)	3.1 (30-day average)	$\frac{WQC - C(1-f)}{f}$
2018 Proposed California Criteria (Rule)	11.2	15.1	11.3	8.5	Site-specific derived from bird egg or fish tissue data using mechanistic model or empirical approach		$\frac{WQC - C(1-f)}{f}$

Notes:
(1) Where WQC is the water column 30-day average for the water column, C is the average background selenium concentration, and f is the fraction of any 30-day period during which elevated selenium concentrations occur, with f assigned a value of ≥0.033 (corresponding to 1 day).

4.4.5 Potential Significance of New and Updated EPA Criteria to OVSD

New criteria can be impactful for WWTP NPDES Permittees in several ways. If a new criterion is lower than existing water quality criteria in the pertinent region's Basin Plan, it could trigger a new 303(d) listing, and subsequent development of a new TMDL for the affected water body. Depending on the Source Assessment and other evaluations conducted during TMDL development, a new TMDL may result in new or more stringent interim and/or final load allocations for the WWTP discharge. The latter will be reflected by effluent limits in NPDES permits issued after the TMDL effective date. In the absence of new 303(d) listings, new or lower water quality criteria can affect effluent limits by triggering "Reasonable Potential." Reasonable Potential Analysis (RPA) is performed by Regional Board staff during permit writing to determine whether effluent quality, or ambient receiving water quality, (or both), have the potential to exceed applicable water quality criteria. For constituents deemed to have Reasonable Potential, WQBELs will be calculated and included in the NPDES permit.

RPA is only performed for priority pollutants detected in effluent that don't have TMDL-related waste load allocations. RPA is performed using the most stringent water quality criterion that applies to the beneficial uses assigned to the pertinent receiving water. In other words, depending on the constituent, the most stringent water quality criterion could be a TMDL load allocation (if one exists), an aquatic life or human health criterion promulgated through the CTR, a statewide water quality criterion adopted by the State Board, a California Title 22 drinking water MCL, or a Regional Board-specific Basin Plan Objective.

In accordance with the CTR, the human health criteria for "consumption of water + organism" only applies when the Municipal and Domestic Supply (MUN) beneficial use is assigned to a water body. In the Basin Plan, the MUN beneficial use is assigned to the Ventura River as a *potential* use, not an *existing* use. The 2018 OVSD Permit provides the following detail regarding the Regional Board's treatment of the potential MUN designation:

> *"Consistent with Regional Water Board Resolution No. 89-03 and State Water Board Resolution No. 88-63, in 1994 the Regional Water Board conditionally designated all inland surface waters in Table 2-1 of the 1994 Basin Plan as existing, intermittent, or potential for Municipal and Domestic Supply (MUN). However, the conditional designation in the 1994 Basin Plan included the following implementation provision: "no new effluent limitations will be placed in Waste Discharge Requirements as a result of these [potential MUN designations made pursuant to the SODW policy and the Regional Water Board's enabling resolution] until the Regional Water Board adopts [a special Basin Plan Amendment that incorporates a detailed review of the waters in the Region that should be exempted from the potential MUN designations arising from SODW policy and the Regional Water Board's enabling resolution]." On February 15, 2002, the USEPA clarified its partial approval (May 26, 2000) of the 1994 Basin Plan amendments and acknowledged that the conditional designations do not currently have a legal effect, do not reflect new water quality standards subject to USEPA review, and do not support new effluent limitations based on the conditional designations stemming from the SODW Policy until a subsequent review by the Regional Water Board finalizes the designations for these waters. This permit is designed to be consistent with the existing Basin Plan*." (Order R4-2018-0170, at F-17; emphasis added)

California MCLs and the Human Health criteria for consumption of water + organisms were *not* utilized in the RPA conducted for OVSD's permit. It's worth noting that if the MUN beneficial use was assigned as an *existing* use in the future for the Ventura River, human health criteria for consumption of water + organisms (which are lower than human health criteria for consumption of organisms only), and possibly California Title 22 MCLs, would be used in RPA, and could result in additional constituents receiving numeric effluent limits.

The data used by Regional Board staff to conduct the RPA in 2018 for the priority pollutants detected in OVSD effluent is placed in context with the most stringent applicable post-CTR updated EPA criteria in Table 4.4. Reasonable potential was triggered only for selenium in the RPA conducted during the 2018 permit renewal, and the trigger was ambient receiving water concentrations, not OVSD effluent concentrations.

Table 4.4 Data Used for RPA for the 2018 OVSD Permit Renewal Compared to the Most Stringent Post-CTR EPA Criteria

CTR #	Constituent	Applicable Water Quality Criterion used in RPA for 2018 Permit (µg/L)	Maximum OVSD Effluent Conc. (µg/L)	Maximum Receiving Water Concentration (µg/L)	RPA Triggered for 2018 Permit?	Most Stringent Applicable Post-CTR EPA Criterion(1) (µg/L)
1	Antimony	4300	0.98	2.9	No	640 (HH-organism only; 2015)
2	Arsenic	150	2.2	3.3	No	N/A
4	Cadmium	4.39(2)	0.11	0.18	No	0.75(3) (Freshwater CCC; 2016)
5a	Chromium III	378.6(2)	1.63	0.33	No	none
5b	Chromium IV	11	1.7	2.2	No	none
6	Copper	18(2)	9.6	9.7	No	(Freshwater CCC; 2007)(4)

Table 4.4 Data Used for RPA for the 2018 OVSD Permit Renewal Compared to the Most Stringent Post-CTR EPA Criteria (continued)

CTR #	Constituent	Applicable Water Quality Criterion used in RPA for 2018 Permit (µg/L)	Maximum OVSD Effluent Conc. (µg/L)	Maximum Receiving Water Concentration (µg/L)	RPA Triggered for 2018 Permit?	Most Stringent Applicable Post-CTR EPA Criterion(1) (µg/L)
7	Lead	166(2)	0.8	5.4	No	none
8	Mercury	0.051	0.003	ND	No	none
9	Nickel	97(2)	10.6	5.5	No	none
10	**Selenium**	**5**	**2.2**	**5.8**	**Yes**	**3.1 (Freshwater CCC for lentic water bodies; 2016)(5)**
11	Silver	10(2)	0.03	0.04	No	none
12	Zinc	187(2)	77.8	124	No	26,000 (HH-organism only; 2002)
14	Cyanide	5.2	2.4	3.1	No	400 (HH-organism only; 2015)
16	2,3,7,8-TCDD (Dioxin)	$1.4x10^{-8}$	$4.1x10^{-7}$	ND	No	$5.1x10^{-9}$ (HH-organism only; 2002)
20	Bromoform	360	1.2	ND	No	120 (HH-organism only; 2015)
21	Carbon Tetrachloride	4.4	0.2	ND	No	5 (HH-organism only; 2015)
23	Dibromochloro-methane	34	11.7	0.22	No	21 (HH-organism only; 2015)
27	Dichlorobromo-methane	46	29	3.7	No	27 (HH-organism only; 2015)
35	Methyl chloride	none	29	ND	No	none
36	Methylene chloride	1,600	1.0	0.2	No	1,000 (HH-organism only; 2015)
38	Tetrachloroethylene	8.85	4.3	ND	No	29 (HH-organism only; 2015)
39	Toluene	200,000	1.0	ND	No	520 (HH-organism only; 2015)
47	2,4-Dimethylphenol	2,300	0.2	ND	No	3,000 (HH-organism only; 2015)
54	Phenol	4,600,000	3.7	ND	No	300,000 (HH-organism only; 2015)
55	2,4,6-Trichlorophenol	6.5	0.4	ND	No	2.8 (HH-organism only; 2015)
61	Benzo(a)pyrene	0.049	0.01	ND	No	0.00013 (HH-organism only; 2015)
62	Benzo(b)flouranthene	0.049	0.01	ND	No	0.0013 (HH-organism only; 2015)
64	Benzo(k)flouranthene	0.049	0.01	ND	No	0.013 (HH-organism only; 2015)
68	Bis(2-ehtylhexyl)-Phthalate	4(6)	1.7	ND	No	0.37 (HH-organism only; 2015)
73	Chrysene	0.049	0.01	ND	No	0.13 (HH-organism only; 2015)
74	Dibenzo(a,h)Anthracene	0.049	0.03	ND	No	0.00013 (HH-organism only; 2015)
76	1,3-Dichlorobenzene	2,600	0.3	ND	No	10 (HH-organism only; 2015)
77	1,4-Dichlorobenzene	2,600	0.1	ND	No	900 (HH-organism only; 2015)

Table 4.4 Data Used for RPA for the 2018 OVSD Permit Renewal Compared to the Most Stringent Post-CTR EPA Criteria (continued)

CTR #	Constituent	Applicable Water Quality Criterion used in RPA for 2018 Permit (µg/L)	Maximum OVSD Effluent Conc. (µg/L)	Maximum Receiving Water Concentration (µg/L)	RPA Triggered for 2018 Permit?	Most Stringent Applicable Post-CTR EPA Criterion(1) (µg/L)
79	Diethyl Phthalate	120,000	0.3	ND	No	600 (HH-organism only; 2015)
92	Indeno(1,2,3-cd)Pyrene	0.049	0.02	ND	No	0.0013 (HH-organism only; 2015)
96	N-Nitrosodimethylamine	8.1	1.3	0.02	No	3.0 (HH-organism only; 2002)
105	Gamma-HCH (lindane)	0.063	0.01	ND	No	4.4 (HH-organism only; 2002)

Notes:

(1) HH indicates a human health criterion. Because MUN is not an *existing* beneficial use for the Ventura River in the Region 4 Basin Plan, CA-MCLs and USEPA updated human health criteria for consumption of water+ organism were not considered applicable for this evaluation. Regional Board staff did not employ human health criteria for organism + water, or California MCLs, in their RPA for priority pollutants when developing OVSD's 2018 permit.

(2) Applicable freshwater acute and chronic criteria for this constituent vary depending on the hardness value. The applicable water quality criterion used by the Regional Board in the RPA for this constituent may not match the analogous values in Appendix 4A, the latter of which correspond to a representative hardness of 100 mg/L. OVSD's 2018 permit did not specify the hardness value used by the Regional Board for RPA.

(3) USEPA criterion shown corresponds to a hardness of 100 mg/L.

(4) Updated USEPA freshwater aquatic life criteria for copper are calculated using site-specific data in a biotic ligand model (BLM), so a comparative value is not available for the table.

(5) The USEPA published proposed *California-specific* selenium criteria in 2018. The proposed rule does not include a default water column concentration criterion. Instead, regulators would have to derive site-specific water column criteria from fish tissue data (egg/ovaries, muscle, whole fish) or bird egg data using mechanistic or empirical models.

(6) The value does not match the CTR HH-organism value, but matches the California MCL for this constituent. MCLs were not used to trigger reasonable potential for other priority pollutants in the RPA, so this might be a typo in the 2018 OVSD Permit, Attachment F.

Table 4.5 lists the seven cases in which a post-CTR updated EPA criterion listed in Table 4.4 is lower than the maximum OVSD effluent concentration for that constituent between 1/2014-3/2018. These are cases which would have technically triggered reasonable potential in 2018, and a need for numeric effluent limits, if the associated updated EPA criteria were already effective in Region 4. As stated above, reasonable potential was already triggered for selenium in the 2018 permit renewal cycle based on ambient *receiving water* concentrations, and OVSD received a numeric effluent limit for selenium for the first time in 2018. It is not yet known what water column concentration for selenium will apply in the Ventura River if the proposed 2018 EPA chronic freshwater criterion for California is adopted in Region 4 (see Table 4.3). Although future permits for OVSD will contain selenium limits, the value assigned to effluent might change if the applicable water quality criterion becomes lower than that used during the 2018 permit renewal. The implications of new and proposed selenium criteria for OVSD are discussed more fully below, along with those for new criteria for ammonia, cadmium, and copper.

Table 4.5 Cases in Which Updated EPA Criteria Would Have Triggered Reasonable Potential for OVSD During the 2018 Permit Renewal

Constituent	Limiting Post CTR EPA Updated Criterion (µg/L)		Maximum OVSD Effluent Concentration (Jan. 2014- Mar. 2018) (µg/L)
Dioxin	2015 Human Health Organism Only	$5.1\text{x}10^{-9}$	4.1×10^{-7}
Benzo(a)pyrene	2015 Human Health Organism Only	0.00013	0.01
Benzo(b)flouranthene	2015 Human Health Organism Only	0.0013	0.01
Bis(2-ethylhexyl)-Phthalate	2015 Human Health Organism Only	0.37	1.7
Dibenzo(a,h)Anthracene	2015 Human Health Organism Only	0.00013	0.03
Dichlorobromo-methane	2015 Human Health Organism Only	27	29
Indeno(1,2,3-cd)Pyrene	2015 Human Health Organism Only	0.0013	0.02

4.4.5.1 Implications of 2013 Updated Ammonia Criteria for OVSD

Numeric effluent limits for ammonia are not set equal to computed water quality criteria. Instead, following procedures laid out in the Basin Plan, numeric limits are calculated by subjecting pH- and temperature-specific water quality objectives to a number of adjustment factors (related to effluent monitoring data variability and frequency) to derive effluent limits. Tentative ammonia limits for the 2018 permit were calculated using the procedures in the Basin Plan, which are pH- and temperature dependent. As described in Attachment F – Fact Sheet of the 2018 permit, Regional Board staff used a 90th percentile effluent pH of 7.8 to calculate a tentative one-hour average (acute freshwater) objective, and a 50th percentile effluent pH of 7.7 and temperature of 21.7 degrees C to calculate a tentative 30-day (chronic freshwater) objective corresponding to the "early-life stages present" condition. This exercise yielded tentative values of 8.11 mg/L and 2.28 mg/L total ammonia-N (TAN) for the one-hour and 30-day objectives, respectively. Conversion of these objectives to effluent limits yielded 8.1 mg/L TAN for the maximum daily limit and 1.6 mg/L TAN for the average monthly limit. These tentative effluent limits would have been more lenient than those in the preceding 2013 permit. Consequently, in keeping with anti-backsliding policy, the numeric limits in the 2018 permit were set equal to those in the 2013 permit, which were 4.6 and 1.9 mg/L TAN for the maximum daily and monthly average limits, respectively.

In order to estimate whether use of the *2013 EPA ammonia criteria* would present a problem for OVSD *in the future*, an effluent pH of 7.8 and temperature of 22 degrees C were applied to Table 5a (for "*Oncorhynchus* spp. present") in the EPA 2013 ammonia criteria document to identify a hypothetical *future* acute criterion of 2.28 mg/L TAN, and to Table 6 in the document to identify a hypothetical *future* chronic criterion of 0.89 mg/L TAN. This exercise assumes that the Regional Board determines that freshwater mussels are present in the Ventura River, or otherwise declines to generate site-specific (more lenient) ammonia criteria for a "mussels absent" condition for the Ventura River.

The tentative criteria derived for OVSD by Regional Board staff in 2018 using current effluent temperature and pH, and hypothetical criteria based on the 2013 EPA criteria (explained above), are compared to effluent monitoring data in Table 4.6. The water quality objectives based on the revised EPA criteria are much lower than those based on the current Basin Plan criteria. However, the monitoring data in the table indicate that ammonia levels in OVSD effluent would not currently exceed the more stringent objectives based on the 2013 EPA criteria for the 'mussels present' condition, and would not trigger reasonable potential in an RPA.

Table 4.6 Comparison of Acute and Chronic Water Quality Objectives for TAN Based on OVSD-Specific pH and DO, Existing Region 4 Basin Plan Criteria, and 2013 Revised USEPA Ammonia Criteria

Computed using Basin Plan objectives and recent OVSD effluent pH (7.8) and temp. (21.7 degrees C)(1) (mg/L)		Extracted from USEPA 2013 Revised Ammonia Criteria tables using pH 7.8 and temp. 22 degrees C (mg/L)		Effluent Monitoring Data for Jan. 2014-Mar. 2018 (mg/L)	
Acute (one-hour)	Chronic (30-day)	Acute (one-hour)(2)	Chronic (30-day)(3)	Highest Daily Effluent Concentration	Highest average monthly effluent concentration
8.11	4.8	2.28	0.89	1.5	0.13

Notes:

(1) Basin Plan objectives are based on the 1999 USEPA update of ammonia criteria.

(2) Drawn from Table 5a in the 2013 USEPA criterion document for "*Oncorhynchus* spp. present".

(3) Drawn from Table 6 in the 2013 USEPA criterion document.

4.4.5.2 Implications of 2018 Proposed California Selenium Criteria for OVSD

The potential significance of the EPA's 2018 proposed selenium criteria for California to OVSD's future permit limits is not possible to evaluate at this time. The proposed rule does not contain a default water column concentration to compare to recent effluent or receiving water data. As explained above, the proposed California rule would require use of site-specific bird egg or fish tissue data to derive site-specific water column criteria. If the final EPA rule is published without default water column numbers, it would probably be some time before Regional Board staff react by undertaking site-specific calculations for water column criteria using available bird egg or fish tissue data. The EPA's 2016 *national* selenium criteria included a default chronic water column criterion (for lotic water bodies) of 3.1 µg/L. This is lower than the current CTR freshwater chronic criterion of 5 µg/L that was used to perform RPA for OVSD's 2018 permit. However, OVSD's recent maximum effluent concentration (2.2 µg/L) would not exceed the more stringent 2016 EPA national water column criterion. It remains to be seen whether the Regional Board will adopt the recommended *national* water column concentration in a future Basin Plan amendment in the absence of the required site-specific fish tissue data needed to derive water column objectives consistent with procedures in the EPA's proposed California selenium rule. Regardless, according to the RPA conducted for OVSD's 2018 permit, selenium concentrations in receiving water have ranged up to 5.8 mg/L, which, if observed in the future, would trigger reasonable potential and ongoing need for numeric effluent limits in future permits.

4.4.5.3 Implications of the 2016 Revised Cadmium Criteria for OVSD

There are no human health criteria for cadmium for consumption of organisms only. The EPA published the original national recommended cadmium aquatic life criteria in 1980 with subsequent revisions in 1985, 1990, 1996, 2001, and 2016. The updated 2016 criteria account for many new laboratory aquatic toxicity tests with cadmium published since the EPA's 2001 criteria document. In addition, the effect of total hardness on cadmium toxicity was also revised using the newly acquired data.

Although cadmium is detected in OVSD effluent, monitoring data did not trigger reasonable potential during the 2018 permit renewal process. The hardness value used by Regional Board staff to calculate the applicable chronic life objective for hardness-dependent metals criteria in OVSD's RPA was not specified in permit Attachment F. However, during an analogous evaluation by LWA in 2018, a hardness of 400 µg/L was used, based on minimum effluent and receiving water (station R-3) hardness of 211 and 383 mg/L, respectively, and a median ambient hardness of 533 mg/L (using monitoring data between Jan. 2013-Mar. 2018).[6] *Applying a representative hardness value of 400 mg/L to the 2016 EPA updated cadmium criteria* yields a freshwater acute criterion of 6.54 µg/L and a freshwater chronic criterion of 2.03 µg/L. Neither value is exceeded by current OVSD effluent or receiving water concentrations. Barring changes in OVSD effluent or receiving water, effluent limits for cadmium are not expected to be imposed on OVSD during the planning period for the Facilities Plan.

4.4.5.4 Implications of the 2007 Revised Copper Criteria for OVSD

In 2007, the EPA issued revised national recommended freshwater aquatic life criteria for copper. The previous (1984) EPA copper criteria were adjusted for hardness (as is the case for most metals). The update was based on new data for copper toxicity and its effects on aquatic life. The new criteria introduced the use of the Biotic Ligand Model (BLM) – a metal bioavailability model that uses receiving water body characteristics to develop site-specific water quality criteria. The BLM requires ten input parameters to calculate a freshwater copper criterion (a saltwater BLM is not yet available): temperature, pH, dissolved organic carbon, calcium, magnesium, sodium, potassium, sulfate, chloride, and alkalinity. The BLM is used to derive the criteria rather than used as a post-derivation adjustment (as was the case with the 1984 hardness-based criteria). This allows the BLM-based criteria to be customized to the particular water under consideration.

Reaction from many potentially affected POTWs in southern California suggests that in many cases application of the BLM results in criteria values that are more lenient than the 1984 criteria. Application of the BLM to site-specific data for the Ventura River was beyond the scope of this review, but could be performed at a later date to investigate the potential effect of the updated criterion on future OVSD effluent limits.

4.5 Requirements Initiated at the State Board or Regional Board

4.5.1 Biostimulatory Substances Amendment to the Water Quality Control Plan for ISWEBE Plan

After a lengthy multi-year process previously involving separate tracks for development of statewide Biological Objectives (related to community composition of invertebrates, algae, etc.) and statewide Nutrient Objectives ("nutrient numeric endpoints") for wadeable streams, the State Board combined the two regulatory processes into a single process that (among other things) is using technical approaches to translate biological goals (measured using

[6] LWA (2018) Ojai Valley Sanitary District RPA, Technical Memorandum submitted to Ojai Valley Sanitary District, June 5, 2018.

bioassessment indices for invertebrates and algae) into biostimulatory objectives.[7] The State Board now proposes to adopt a statewide narrative water quality objective for "biostimulatory substances" (i.e., nutrients) with numeric translators, along with a program of implementation (with various regulatory control options for point and non-point sources) as an amendment (Biostimulatory Substances Amendment) to the ISWEBE Plan.

The stakeholder and science advisory elements of the previous two tracks have been carried over into the Biostimulatory project, and the Southern California Coastal Water Research Project (SCCWRP) has remained the State Board's chief technical consultant. The technical team from SCCWRP has produced a large number of reports and scientific journal articles to date that (among other outcomes) 1) describe the development and validation of new statewide benthic macroinvertebrate and algal assessment indices;[8] 2) use a variety of statistical and conceptual approaches to propose thresholds for community composition that demonstrate a deviation from reference condition; 3) recommend additional biostimulatory indicators such as algal biomass or ash-free dry weight, or nutrient concentrations; and 4) evaluate the technical merit of differential treatment of channels in human modified landscapes.

Starting in 2019, the project is transitioning from the principal stakeholder engagement and technical phase into policy development. Management is reviewing a work plan for policy development, the Science Advisory Panel is finalizing its work, and the SCCWRP consultant team will be revising technical reports, but not starting new work products at this time. The target date for draft policy provisions is in Spring 2020, with public review in Spring-Summer 2021, and potential adoption of the Amendment in Fall 2021.

The magnitude of the TN and TP concentrations that have been presented in a number of work products as potential numeric thresholds (using a variety of approaches) are much lower than the N and P numbers used to date in southern California TMDLs, *landing well below 1.0 mg/L TN and 0.1 mg/L TP*. It is not clear whether the State Board will adopt values this low as the default statewide nutrient targets for perennial streams and how much discretion will be granted to regional boards to implement the new requirements. However, both the biointegrity components of the plan (the new macroinvertebrate and algal indices) and the numeric nutrient translators will be available for use by the Regional Board if they reopen the Algae TMDL, or when they develop a new Benthic Community Effects TMDL for the lower Ventura River (see below). Either action could lead to assignment of very low N and P targets for receiving water (no targets for N and P were assigned to receiving waters in the Algae TMDL), less generous estimates of assimilative capacity, and potentially lower N and P allocations for dischargers than were included in the Algae TMDL. There would need to be a review period that assesses not just nutrient concentrations, but also algal biomass, DO, and potentially other indicators (like benthic community) to determine if there is still a problem before the TMDL values would change.

4.5.2 Potential Algae TMDL Reopener

TMDL implementation schedules usually include target dates for Regional Board reconsideration of targets and allocations (termed "reopeners"). The timing often corresponds to the due dates for required or optional Special Studies, or other actions that could provide a basis for

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[7] As of February 2019, the State Board portal to present and past documentation is at https://www.waterboards.ca.gov/water_issues/programs/biostimulatory_substances_biointegrity/.

[8] The California Stream Condition Index (CSCI) for macroinvertebrates, and the Algal Stream Condition Index (ASCI) for diatoms and (currently) soft-bodied algae.

re-evaluation. According to the implementation schedule in the Algae TMDL, the Regional Board was scheduled to reconsider the TMDL sometime in 2018 (i.e., five years after the effective date of the TMDL). The reopener was to follow the submission in 2017 of optional special studies. None of the optional special studies included in the TMDL were carried out by responsible parties or other stakeholders, and the Regional Board has not publicly signaled their intent to reopen the TMDL at this time. However, the Regional Board is funding development of a nutrient component to accompany the State Board's integrated hydrologic surface water-groundwater model for the Ventura River Watershed (being developed as part of the minimum flows project, see Section 4.6 below). It is expected that Regional Board staff will utilize this model to reexamine the source assessment from the Algae TMDL, and possibly revise nutrient allocations, if the TMDL is reopened in the future. Should the TMDL be reopened, the implementation schedule in the TMDL addresses a potential extension of the current 12-year deadline for OVSD to meet its dry-weather allocations:

> "*If TMDL reconsideration results in more stringent WLAs, then the implementation schedule for OVSD may be extended, if necessary, by only the amount of time required to upgrade treatment processes to meet the more stringent WLAs*." (LARWQCB R12-011, footnote to Table 7-35.2.)

4.5.3 New 303(d) listings for the Ventura River

Section 303(d) of the federal CWA requires states to identify waters where WQS are exceeded and beneficial uses are not attained. These waters are compiled into the Section 303(d) list of impaired waters. This includes waters impaired as a result of non-point source, point source discharges or combined point source and non-point source contributions including natural sources. The term "303(d) list" is short for a state's list of impaired and threatened waters (e.g., stream/river segments, lakes).

States are required to submit their 303(d) list for EPA approval every two years. For each water on the list, the state identifies the pollutant causing the impairment, when known. In addition, the state assigns a priority for development of TMDL based on the severity of the pollution and the sensitivity of the uses to be made of the waters, among other factors. Along with 303(d) lists of impaired waters, states are required to submit section 305(b) water quality reports to the EPA. Section 305(b) reports provide information on the water quality status of all waters in the state, whereas section 303(d) lists are a subset of these waters – those that are impaired by a pollutant and in need of a TMDL. Given that both the 305(b) report and the 303(d) lists are due at the same time (April 1 of every even numbered year), the EPA recommends that states combine them into a single "Integrated Report."

Each Regional Board in California prepares its own 303(d) listing recommendations, which are reviewed and sometimes modified by the State Board. California has now staggered its listing process so each region will go through a listing process once every six years. For the 2014 and 2016 listing cycles, the State Board produced a single 2014 and 2016 California Integrated Report. The California 303(d) List that was part of the 2014/2016 Integrated Report was approved by the EPA on April 6, 2018.

Table 4.7 compares the listings from the 2012 and the 2014/2016 303(d) lists for the lower reaches of the Ventura River (i.e., reaches below Foster Park; see Figure 4.1 for reach designations). The 2014/2016 list contains two new listings in the lower portion of the watershed: 1) a new toxicity listing for Reach 3, and 2) a new Benthic Community Effects listings for Reaches 1-2. The fact sheet for the toxicity listing (Decision 63974) indicates that OVSD monitoring data for receiving water station R-3 was the basis for the listing (8 out of 43 samples they looked at were exceedances). The fact sheet for the Benthic Community Effects listing indicates that the assessment relied on two samples with Index of Biotic Integrity (IBI) scores below 40 from the Main Street Bridge site in September 2006 and 2007 from Ventura County Watershed Protection District bioassessment reports. Benthic Community Effects listings had also been initially proposed for Reaches 3 and 4 in the 2014/2016 cycle, but the State Board dissented during their review of the proposed 303(d) list for Region 4, and the final listing decisions for Reaches 3 and 4 were "Do Not List".

4.5.3.1 New Toxicity Listing for Reach 3

OVSD effluent is discharged downstream of Reach 3 and would not be considered a potential source in a toxicity TMDL that addressed only Reach 3, or other upstream reaches. Through an amendment to the ISWEBE Plan, the State Board will be implementing new Toxicity Provisions in the near future. The Toxicity Provisions were originally scheduled for adoption in early 2019, but action has been delayed by the State Board and no revised schedule has been released publicly. As part of the provisions, a new approach for analyzing toxicity test data (the TST) will serve as the basis for statewide numeric objectives for acute and chronic toxicity. In advance of adoption of the provisions at the State Board level, Regional Board staff have been expressing toxicity limits in Region 4 NPDES permits using the TST for several years. In accordance, toxicity limits (for chronic toxicity only) using the TST were included in OVSD's 2018 permit.

The new metric raises the possibility that future TST test results for receiving water may diverge from historic data reported using the previous metric ("TUa" and "TUc", for acute and chronic tests, respectively), potentially giving rise to toxicity listings in other reaches in the lower Ventura River. In 2018, LWA translated a series of previous acute and chronic toxicity test results from OVSD into TST units and compared them to the original test results reported in TUa and TUc.[9] The comparison was performed for tests from 2017 and early 2018 using fathead minnow and *Ceriodaphnia dubia* in effluent and receiving water samples from stations R-3 and R-4. The comparison showed that TST outcomes tracked those obtained using the standard TU units, providing no *initial* indication that use of the TST will lead to more frequent toxicity hits for OVSD effluent or for receiving water immediately above or below the OVSD discharge.

4.5.3.2 Benthic Community Effects Listing for Reaches 1-2

Given that there are ongoing exceedances of the algal-biomass-related targets from the Algae TMDL in the lower river (see TM 3 for the Facilities Plan), the Causal Assessment step during development of a TMDL for Benthic Community Effects in Reaches 1-2 would almost certainly include nutrient loading and associated biostimulatory effects as one of the potential causes of low invertebrate index scores. Existence of the 2013 Algae TMDL will not preclude the Regional Board from examining nutrient loading in this new context, possibly establishing TN and/or TP targets for the receiving water (for the first time), and potentially assigning new allocations for TN or TP for dischargers. Even if the targets for algal biomass, pH and DO from the Algae TMDL *were* being met at that time, the Regional Board could decide that the requirements in the Algae TMDL are not protective of benthic community composition. Non-nutrient factors (such as toxicity, suspended sediment, hydromodification) would likely also be considered in a Causal Assessment.

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[9] LWA (2018) Evaluation of Recent Toxicity Data using the Test of Significant Toxicity (TST). Technical Memorandum submitted to Ojai Valley Sanitary District, May 24, 2018.

The 303(d) list assigned a "due date" for the Benthic Community Effects TMDL of 2029. This time frame makes it likely that the provisions of the Biostimulatory Substances Amendment will govern the approach for setting nutrient targets and for providing any regulatory "off ramps" in an implementation plan. Because the State Board is on the path toward adopting the statewide macroinvertebrate index (California Stream Condition Index (CSCI)) and a new statewide algal index (Algal Stream Condition Index (ASCI)) as the appropriate metrics for bioassessment (see below), it's possible that future data for the Ventura River obtained using the new metrics could affect the status of the Benthic Community Effects listing in future listing cycles (either confirming or refuting the listing) – or could affect whether other reaches become listed in addition to Reaches 1-2.

Significance of the Biostimulatory Substances Amendment for OVSD

It is currently unknown how the Biostimulatory Substances Amendment will be implemented for specific water bodies or POTWs in general. OVSD would be affected by a Benthic Community Effects TMDL in the future if the Regional Board decided to impose effluent limits (concentrations or seasonal loads) for TN and TP lower than the final limits established by the Algae TMDL. Accordingly, since the Facility Plan planning horizon stretches 20 years to 2038, it would be prudent to evaluate whether effluent TN and TP concentrations lower than those required by the Algae TMDL could be achieved without resorting to reverse osmosis.

Table 4.7 Impairments Requiring TMDLs on the 2012 and 2014-2016 303(d) lists for Reaches in the Lower Ventura River[1]

Reaches in Lower Ventura River	Pollutant Category	Pollutant	on 2012 303(d) List?	on 2016 303(d) List?	TMDL status as of 2016 303(d) List
Estuary	Nutrients	Algae	X	X	Algae TMDL approved Jun. 2013
		Eutrophic	X	X	Algae TMDL approved Jun. 2013
	Fecal Indicator Bacteria[2]	Total Coliform	X		
		Indicator Bacteria		X	TMDL expected 2019
	Trash	Trash	X	X	Trash TMDL approved Feb. 2008
Reach 1 & 2	Nutrients	Algae	X	X	Algae TMDL approved Jun. 2013
	Miscellaneous	**Benthic Community Effects[3]**		**X**	**TMDL expected 2029**

Table 4.7 Impairments Requiring TMDLs on the 2012 and 2014-2016 303(d) lists for Reaches in the Lower Ventura River(1) (continued)

	Pollutant Category	Pollutant	on 2012 303(d) List?	on 2016 303(d) List?	TMDL status as of 2016 303(d) List
Reach 3	Fecal Indicator Bacteria	Indicator Bacteria	X	X	TMDL expected 2021
	Toxicity	Toxicity		X	TMDL expected 2027
	Hydromodification	Pumping, Water Diversion	X		removed from 303(d) list during 2014-2016 listing cycle

Notes:
(1) Based on the Final 2014 and 2016 Integrated Report Dated October 3, 2017 at https://www.waterboards.ca.gov/water_issues/programs/tmdl/2014_16state_ir_reports/category5_report.shtml, accessed via https://www.waterboards.ca.gov/water_issues/programs/tmdl/integrated2014_2016.shtml.
(2) Comment included on 303(d) List: "Stables and horse property may be the sources."
(3) Based on 2 IBI scores <40 near the Main Street Bridge, from September 2006 and September 2007.
(4) Comment included on 303(d) List: "Horse stables, land use, cattle, and wildlife may be sources".

4.5.4 2017 Mercury Provisions and New Tribal and Cultural Beneficial Uses

In Region 4, the historically applicable mercury objectives are human health criteria from the CTR, which are water column concentrations of total mercury of 0.050 µg/L (human health – organism + fish) and 0.051 µg/L (human health – organism only). In 2001, the EPA issued a new national fish tissue criterion for *methylmercury*. This objective was included in Appendix 4-A as one of the new or updated EPA criteria issued since May 2000, and thus would ordinarily be one of the EPA standards under review for adoption in Region 4 (and elsewhere in California) based on the triennial review process described above. However, on May 2, 2017, the State Board adopted Resolution 2017-0027, which approved "Part 2 of the Water Quality Control Plan for ISWEBE Plan —Tribal and Subsistence Fishing Beneficial Uses and Mercury Provisions." Through this amendment to the ISWEBE Plan, the State Board designated three new beneficial uses and new methylmercury objectives (expressed both as fish tissue limits and translated into water column concentrations of total mercury). The new beneficial uses are as follows:

- Tribal Tradition and Culture (CUL).
- Tribal Subsistence Fishing (T-SUB).
- Subsistence Fishing (SUB).

Applicability of the new fish tissue objectives is governed not only by existence of one (or more) or the new beneficial uses in a particular water body, but also by the existence of one or more of eight beneficial uses already employed throughout the state (COMM, WILD, MAR, RARE, WARM, COLD, EST, and SAL). The five new fish-tissue based objectives are compared to the 2001 EPA criterion in Table 4.8.

Table 4.8 Comparison of 2001 EPA and 2017 California Fish-Tissue Objectives for Methylmercury

Water Quality Objective	Applicable Beneficial Uses	Fish Tissue Objective (mg/kg)
2001 EPA Human Health – consumption of organism only	N/A	0.3
2017 California Mercury Provisions Fish Tissue Objectives		
Sport Fish	COMM, CUL, WILD, MAR, WARM, COLD, EST, SAL	0.2[2]
Tribal Subsistence Fishing	T-SUB	0.04[3]
Subsistence Fishing	SUB	Region- or site-specific
Prey Fish	WILD or MAR	0.05
California Least Tern	WILD, MAR, WARM, COLD, EST, SAL, or RARE only where least tern or least tern habitat exists[1]	0.03

Notes:
(1) A list of water bodies in which the California Least Tern objective applies is provided in Attachment D of Appendix A: Final Staff Report: Part 2 of the Water Quality Control Plan for ISWEBE Plan – Tribal and Subsistence Fish Beneficial Uses and Mercury Provisions (ISWEBE Plan).
(2) When WILD or MAR uses exist, objective must be applied to Trophic Level 4 Fish.
(3) Skinless fillets of a mixture of 70% Trophic Level 3 and 30% Trophic Level 4 Fish.

The ISWEBE Plan amendment also included a translation of fish tissue objectives into water column concentrations of total mercury for use in RPA and development of effluent limits. Regional boards may use the default water column concentration pertinent to a water body, or develop site-specific water column concentrations using special studies to derive BAFs or models. The default water column concentrations are provided in Table 4.9 and compared to the CTR human health criteria.

Table 4.9 Water Column Concentrations for Total Mercury Translated from 2017 California Fish Tissue Objectives Compared to CTR Human Health Criteria

Beneficial Uses	2017 California Mercury Provisions Water Column Translators (µg/L)							CTR Human Health Criteria (µg/L)
	COMM, CUL, WILD, MAR, WARM, COLD, EST, SAL, RARE		COMM, CUL, T-SUB, WILD, MAR, WARM, COLD, RARE	T-SUB		SUB	MUN	not MUN
	Flowing Water	Slow Water	Lakes & Reservoirs	Flowing Water	Slow Water	Any	Organism + Water	Organism Only
Water Column Concen-tration	0.012	0.004	Case by Case	0.004	0.001	Case by Case	0.050	0.051

Significance of the 2017 California Mercury Provisions to OVSD

Unless, in the future, the Regional Board assigns the T-SUB or SUB beneficial use to the Ventura River or its Estuary (which seems unlikely as fishing is prohibited in the watershed to protect endangered Southern California Steelhead), the practical significance of the new mercury provisions for OVSD hinges on the fact that the beneficial uses WILD, RARE, WARM, and COLD already apply to the Ventura River, and that the beneficial uses WILD, RARE, EST, and MAR already apply to the Ventura River Estuary. Consequently, the "Flowing Water" total mercury concentration of 0.012 µg/L would probably apply to Reaches 1-4 of the river, and the concentration of 0.004 µg/L to the estuary. During the 2018 OVSD permit renewal, Regional Board staff performed RPA for mercury using the CTR human health organism-only objective for total mercury as the applicable water quality objective and a maximum effluent concentration of 0.003 µg/L total mercury. Mercury was not detected in receiving water. Based on these data for 2014-2018, reasonable potential would not have been triggered for the OVSD discharge even if the new mercury provisions had been applied during permit renewal in 2018.

Based on the above evaluation, and barring changes in effluent or receiving water quality, it is not expected that effluent limits for mercury would be included in future OVSD permits.

4.5.5 Recycled Water Policy re. Salt and Nutrient Management Plans

OVSD currently comfortably meets its numeric concentration-based effluent limits for salts (TDS, sulfate, chloride, and boron), which are set equal to the Basin Plan surface water objectives for Reach 2, as follows:[10]

- TDS1500 mg/L.
- Chloride300 mg/L.
- Sulfate500 mg/L.
- Boron1.5 mg/L.

No surface water objectives for salts apply to Reach 1 or the estuary. Of the four salt constituents, beneficial uses of surface water in Ventura County are usually considered most sensitive to chloride levels, owing to the agricultural (AGR) beneficial use and cultivation of chloride sensitive row and tree crops. Table 4.10 provides a summary of recent chloride data for OVSD effluent and receiving water monitoring sites.

Table 4.10 Recent Chloride Concentrations in OVSD Effluent and Receiving Water Stations

Location		Average 2016-2017 Chloride Concentration (mg/L)
Above Discharge	RSW-003	71
Below Discharge	RSW-004	104
	RSW-005	109
OVSD Effluent		148

The surface water objective for chloride in the reach OVSD discharges to (300 mg/L, for Reach 2) is much higher than those assigned to the reaches above them (50-60 mg/L). For context, the highest surface water objective for chloride in neighboring watersheds is 150 mg/L - and this level may be considered too high by agricultural stakeholders for chloride sensitive crops. The

[10] The salts objectives that apply to Reach 2 are described in Basin Plan Table 3-10 as applicable "between confluence with Weldon Canyon and Main Street."

surface water objective for Reach 2 is an order of magnitude higher than the groundwater objective for the Lower Ventura River groundwater basin (30 mg/L chloride).

The necessary research to discover the basis for the surface and groundwater chloride objectives for the Ventura River watershed was not performed for this memorandum. However, based on information for other Ventura County water bodies, the objectives were likely developed by applying anti-degradation policy to decades-old ranges of historic concentrations, rather than through consideration of concentrations needed to meet existing beneficial uses.

None of the main stem reaches of the Ventura River are currently listed as impaired by salts, so no TMDL for salts is imminent that could trigger changes to the salts effluent limits for OVSD. However, OVSD's load-based limit for salts could come under scrutiny if a Salt and Nutrient Management Plan (SNMP) was developed for the lower Ventura River groundwater basin in the future and a fate-and-transport analysis demonstrates that recharge from the lower river is impairing the beneficial uses of groundwater in the Lower Ventura River Basin.

SNMPs are not a direct vehicle for changes to NPDES permits; the plans provide opportunities to develop site-specific objectives that are better linked to existing beneficial uses in basins and allow development of implementation measures and projects to manage salts in ways that may not affect NPDES permit limits. Any site-specific objectives identified in a SNMP would require a Basin Plan Amendment before they were effective and could be used for modified effluent limitations.

The original (2013) State Recycled Water Policy required development of SNMPs for all groundwater basins in the state. However, in their 2018 amendment to the Recycled Water Policy, the State Board no longer has this blanket requirement, but does require regional boards to identify groundwater basins where SNMP have not yet been developed, but are still needed to achieve water quality objectives for salts and nutrients in the long-term. The amendment allows for regional boards to prioritize basins, as follows:

> "*6.1.3. Basin evaluation. To sustain the ongoing development of salt and nutrient management plans in basins where plans are needed and to clarify where salt and nutrient management planning is not needed, each regional water board shall evaluate each basin or subbasin in its region within two years of [effective date of the amendment] and identify basins where salts and/or nutrients are a threat to water quality and therefore need salt and nutrient management planning to achieve water quality objectives in the long term.....Each regional water board shall update this evaluation at least every 10 years to consider any changes in these factors that have occurred that would change the findings from the initial evaluation....Regional water boards may consider the following factors in this determination, as well as any additional region-specific factors:*
>
> - *Magnitude of and trends in the concentrations of salts and nutrients in groundwater.*
> - *Contribution of imported water and recycled water to the basin water supply.*
> - *Reliance on groundwater to supply the basin or subbasin.*
> - *Population.*
> - *Number and density of on-site wastewater treatment systems.*
> - *Other sources of salts and nutrients including irrigated agriculture and confined animal facilities.*" (Draft Amendment to the Recycled Water Policy, 5/9/2018).

Since the Facility Plan covers a 20-year planning window, it is considered prudent to perform an evaluation of what would need to be done at the OVSD WWTP to reduce chloride concentrations. It is also in OVSD's interest to remain aware of available data for salt concentrations from wells screened in the Lower Ventura River Basin.

4.5.6 Onsite Wastewater Treatment Systems (OWTS) Policy

Future enhanced regulation of Onsite Wastewater Treatment Systems (OWTS) in the Ventura River Watershed may lead to pressure for OVSD to provide connections to some fraction of the currently unsewered households situated near reaches of the main stem of the Ventura River and San Antonio Creek. The regulatory triggers for the enhanced regulation are described below.

4.5.6.1 OWTS Requirements Triggered by the Algae TMDL

The Algae TMDL established a load allocation for OWTS of 7,478 pounds TN per year, based on a required 50 percent reduction in loading. This implies a requirement to *reduce* OWTS loading by the same amount. The load allocation applies in dry and wet weather. No load allocation was assigned to OWTS for TP.

Essentially concurrently with the adoption of the Algae TMDL, the State Board adopted the Water Quality Control Policy for Siting, Design, Operation, and Maintenance of Onsite Wastewater Treatment Systems (OWTS Policy). This Policy established a statewide, risk-based, tiered approach for the regulation and management of OWTS installations and replacements and set the level of performance and protection expected from OWTS. Among other objectives, the OWTS Policy includes requirements for OWTS near waters on the 303(d) list for nutrients or pathogens and authorizes local agencies to implement corrective actions, require monitoring, establish exemption criteria, and to determine when existing OWTS are subject to major repair.

Owing to the inclusion of OWTS as a source of nutrient loading in the Algae TMDL, the OWTS in the Ventura River watershed fall under "Tier 3" of the OWTS policy by default, which has the effect of establishing an "Advanced Protection Management Program" for the watershed. The schedule in the Algae TMDL provided a three-year window after the TMDL implementation date for the following task: "Regional Board staff and Ventura County will work to determine areas of OWTS to be included in an Advanced Protection Management Program area and a plan for a 50 percent reduction of loading from OWTS in these areas." Toward this end, the Regional Board sponsored a consultant to perform a study for the Ventura County Environmental Health Division to identify the areas in the watershed in which OWTS were likely to be contributing to nutrient loading in the Ventura River.[11] The report was reviewed internally by a technical advisory team and is now under review by Regional Board staff. Ultimately the consultant mapped out "high risk" and "low risk" zones as follows:

- Low density OWTS (within 2,000 ft buffer of impaired reaches) or not within 2,000 feet buffer of impaired reaches = Low risk of surface water contamination.
- Medium and high density OWTS (within 2,000 ft buffer of impaired reaches) = high risk or potential risk of surface water contamination based on downgradient surface water nitrate levels observed in the study and historically.

[11] Geosyntec (2018) Technical Report for the Study of Water Quality Impairments Attributable to Onsite Wastewater Treatment Systems (OWTS) in the Ventura River Watershed. Prepared for County of Ventura Environmental Health Division, September 2018.

Out of an estimated 2,874 OWTS in the watershed, the consultant concluded that 42 were located in the "high risk" area and 760 were in the "potential risk" area (i.e., 28 percent of all OWTS in the Ventura River Watershed were classified as high or potential risk). It remains to be seen how the study will be received by the Regional Board, and how it will be translated into an approved Local Agency Management Program. As a member of the technical advisory team, OVSD submitted detailed comments about the draft report, among which were 1) that a whole unsewered neighborhood within the limits of the City of Ojai was left out of the analysis, 2) that stream monitoring data utilized were a fraction of the publicly available data, were unsuitable for estimating downgradient effects for a variety of reasons, and were not reflective of groundwater/surface water interactions, 3) direct sources of information about failing septic tanks available from public agencies (including OVSD) was not utilized to map risk of surface water contamination, and 4) avenues for transport of OWTS leachate during wet weather or wet years were not considered in the analysis.

At some point in the future, OVSD will likely be called upon by the County Environmental Health Division and/or the Regional Board to consider new connections for suitably located high-risk OWTS. The TMDL required a 50 percent reduction in loading from OWTS, equal to 7,478 pounds TN per year. If one assumes a residential sewage output of 200 gallons/household/day and an average OVSD influent concentration of 36 mg/L TN,[12] a household load of 0.06 lb TN/day can be estimated, which extrapolates to 22 lbs TN per household per year. Based on this estimate, the TMDL-required OWTS load reduction equates to complete elimination of N loading from 340 unsewered households, a number that is an order of magnitude higher than the 42 high-risk OWTS that were identified in the Regional Board sponsored OWTS study.

As required by the Algae TMDL, the final summer (May-September) effluent limit for OVSD is expressed as a load of 8,044 lbs TN/season. Assuming a constant residential sewage output over the course of a year, the TMDL-required annual OWTS load reduction of 7,478 lb/year corresponds to a May-September load reduction of 3,116 lbs. If OVSD accommodated enough unsewered households to meet the entire TMDL-specified OWTS load reduction, its influent load during the May-September period would hypothetically increase by 3,116 lbs. Currently, OVSD's treatment processes remove about 87 percent of TN. A new influent load of 3,116 lbs over the course of May-September would lead to an estimated additional discharge of 413 lbs of TN in effluent. Assuming current treatment nitrogen removal efficiencies, the hypothetical new OWTS load would consume about 413/8044, or 5 percent of OVSD's summer load allocation. OVSD will need to evaluate whether it can accommodate additional connections and also meet the stringent final effluent limit for TN imposed by the TMDL. This aspect of facility planning is discussed in more detail in TM 5, TMDL Implementation and Facilities Upgrades, which presents treatment process alternatives to meet the TMDL.

4.5.6.2 OWTS Requirements Potentially Triggered by the Reach 3 Pathogen Listing

In 2018, the State Board updated the OWTS Policy, in part by revising tables that identify impaired water bodies where: 1) it is likely that operating OWTS will subsequently be determined to be a contributing source of pathogens or nitrogen and therefore it is anticipated that OWTS would receive a loading reduction, and 2) it is likely that new OWTS installations discharging within 600 feet of the water body would contribute to the impairment. Per the

[12] Both values from LWA (2011) Corrected Source Assessment Report: Nitrogen and Phosphorus in the Ventura River Watershed, report submitted to the Ojai Valley Sanitary District, August 9, 2011.

OWTS Policy (Tier 3, Section 10) the Regional Water Boards must adopt TMDLs by the dates specified in the table.

The tables in the Policy Update include Reach 3 of the Ventura River, and assign a Pathogen TMDL completion date of 2024. The estimated TMDL completion date in the 2014/2016 303(d) list for the "fecal indicator bacteria" listing in Reach 3 is 2021. *The pathogen-related nexus to the OWTS Policy provides a potential regulatory pathway independent of the Algae TMDL to require septic tank upgrades or new connections to OVSD for unsewered properties situated near Reach 3*. The outcome will depend on whether OWTS are given a load allocation in a future pathogen TMDL.

Although unrelated to OVSD's numeric effluent limits, there is another potential impact to OVSD from a pathogen TMDL for Reach 3. When the pathogen TMDL is initiated, it is likely that human sources of pathogens will be investigated. Should the collection system or other wastewater infrastructure be identified as a source of pathogens (e.g., exfiltration), it is possible that OVSD could be assigned an allocation in the pathogen TMDL that could require collection system upgrades unrelated to effluent limits.

4.6 Future Regulation of Surface Flow in the Ventura River

OVSD does not receive dilution credits from instream flows in the Ventura River. Thus the amount of flow in the Ventura River does not have a *direct* effect on OVSD effluent limits. However, instream flows can have an *indirect* effect on OVSD's permit limits in several ways. First, the amount of base flow in the river affects instream concentrations of pollutants that are introduced as point and non-point sources. Instream concentrations of pollutants are one of the factors used in RPA during permit writing. If pollutants are diluted by chronically higher base flows, the chances that receiving water concentrations sampled above the OVSD discharge will exceed WQS is reduced, which in turn lowers the chances that receiving water concentrations will trigger a need for numeric effluent limits. Second, base flow ameliorates the effect of some pollutants. For example, higher base flows can decrease the extent to which benthic algae and other submerged aquatic respirants cause nocturnal excursions of DO below aquatic life criteria. This can happen as a function of higher reaeration rates in turbulent subhabitats such as riffles, indirectly through maintenance of lower water temperatures with higher dissolved oxygen solubility, and also because at higher flows, the collective respiratory demand of the submerged biota is exerting its effect on a larger volume of water. Third, higher base flows can reduce residence time and the extent to which labile dissolved or particulate matter participates in biogeochemical cycles after it enters a stream. In the case of nutrients, higher base flows may reduce the cumulative effect of a given nutrient load by increasing what are referred to as "spiraling lengths." Spiraling length represents the distance over which the average nutrient atom travels in a river or stream as it completes one cycle of utilization from a dissolved available form, passes through one or more metabolic transformations, and is returned to a dissolved available form. Finally, in the TMDL context, in cases in which nutrients are regulated using water column concentrations (as opposed to loads), higher base flows would increase the estimated assimilative capacity of the water body, and could lead to more favorable concentration-based load allocations for dischargers.

At least three regulatory processes are currently underway in the Ventura River Watershed that may affect the magnitude of base flows in the future: 1) Sustainable Groundwater Management Act (SGMA) related regulation of groundwater pumping in the basins underlying the Ventura

River Watershed, 2) the State Board's current project to establish minimum flow requirements in the Ventura River to protect Southern California Steelhead, and 3) the City of Ventura's lawsuit seeking an adjudication of all surface and groundwater diversions in the watershed.

4.6.1 SGMA

On September 16, 2014, Governor Jerry Brown signed into law a three-bill legislative package, composed of AB 1739 (Dickinson), SB 1168 (Pavley), and SB 1319 (Pavley), collectively known as SGMA. SGMA requires governments and water agencies of high and medium priority basins to halt overdraft and bring groundwater basins into balanced levels of pumping and recharge. For each of these basins, SGMA requires that either a Groundwater Sustainability Plan (GSP) be submitted that demonstrates how a basin will reach sustainability within 20 years of implementation, or that a qualifying local agency submit an "Alternative" that demonstrates from the outset that a basin has been operated within its sustainable yield over a period of at least 10 years. Management by a water master pursuant to an adjudication qualifies as an Alternative.

Four groundwater basins underlie portions of the Ventura River Watershed:

- Upper Ojai Valley.
- Ojai Valley Basin.
- Upper Ventura River Basin.
- Lower Ventura River Basin.

The basins are shown on Figure 4.2. Of these, the Ojai Valley Basin was designated by DWR as a High Priority Basin, and the Upper Ventura River Basin was designated as a Medium Priority Basin. As of the final 2018 Basin Prioritization (released by DWR in January 2019), the other two basins are designated as Very Low Priority Basins; GSAs are not required to form for these basins and GSPs are not required to be written.

The Ojai Basin Groundwater Management Agency (OBGMA), established in 1991 through enabling legislation CA SB 534, qualified as a local agency eligible to submit an Alternative for DWR review for the Ojai Valley groundwater basin. OBGMA submitted the required documentation in December 2016. DWR has yet to release its determinations regarding whether Alternatives that were submitted for numerous basins throughout the state are sufficient to serve as alternatives to, and in lieu of preparing, a GSP.

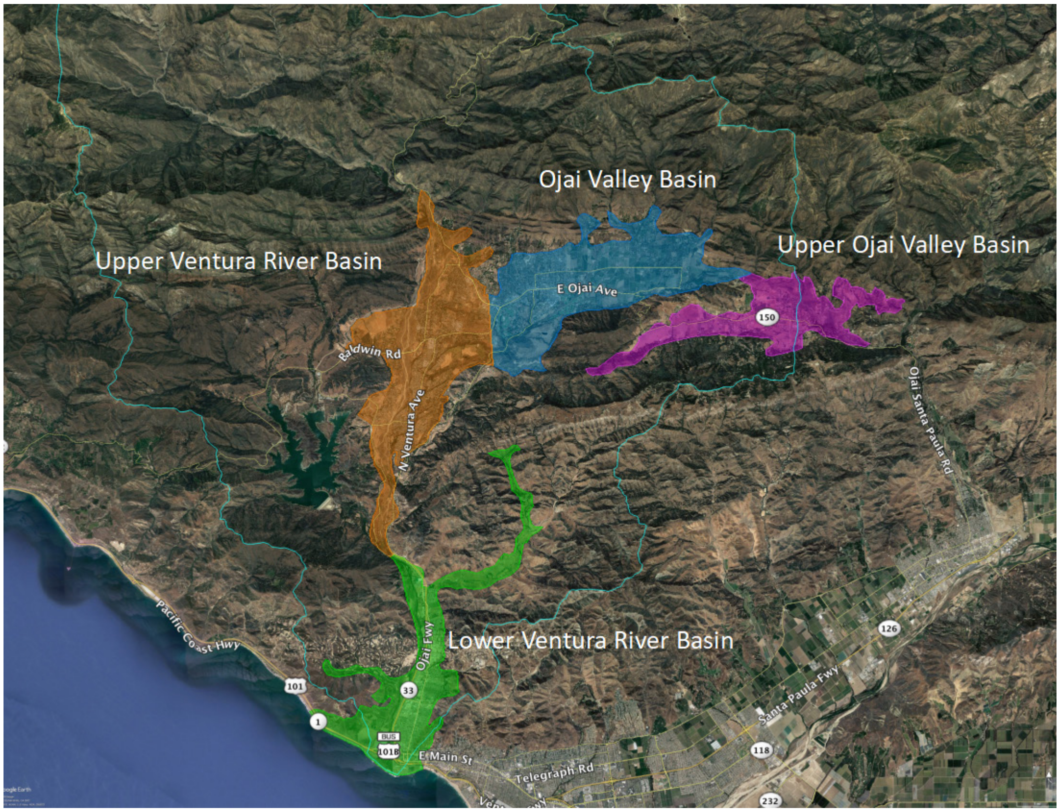

Figure 4.2 Groundwater Basins Underlying the Ventura River Watershed

The Upper Ventura River Groundwater Agency (UVRGA) was officially formed when a Joint Exercise of Powers Agreement (JPA) was executed in December 2016 by five member agencies:

- Casitas Municipal Water District.
- County of Ventura.
- City of Ventura (Ventura Water).
- Meiners Oaks Water District.
- Ventura River Water District.

The UVRGA received a cost-share grant from DWR in 2018 and is in the early stages of conducting the work to generate a GSP for the Upper Ventura River Basin by the required deadline of January 31, 2022.

A GSP must demonstrate how the following six "undesirable effects" will be avoided: 1) chronic lowering of groundwater levels indicating a significant and unreasonable depletion of supply if continued over the planning and implementation horizon, 2) significant and unreasonable reduction of groundwater storage, 3) significant and unreasonable seawater intrusion, 4) significant and unreasonable degraded water quality, including the migration of contaminant plumes that impair water supplies, 5) significant and unreasonable land subsidence that substantially interferes with surface land uses, and 6) depletions of interconnected surface water that have significant and unreasonable adverse impacts on beneficial uses of the surface water. It is expected that the quantification of measurable objectives and minimum thresholds (parameters to be determined (TBD)) addressing the sixth undesirable effect listed above will be a significant driver during modeling of sustainable yield and the identification of management measures and projects for the Upper Ventura River basin. This SGMA requirement provides the most direct nexus between the authorities granted to the UVRGA to manage groundwater pumping, and the other two parallel processes underway that aim to manage surface water and/or groundwater use.

4.6.2 Adjudication of Surface, Subsurface, and Groundwater Diversions

The adjudication suit arises as part of the ongoing court case *Santa Barbara Channelkeeper v. State Water Resources Control Board and City of San Buenaventura* (California Superior Court case CPF-14-513875) that started in September 2014. In January 2018, the first District Court of Appeals (San Francisco) upheld the City of Ventura's previously denied cross complaint that named several other distinct parties alleged to affect flow in the Ventura River, including Casitas Municipal Water District, other local water providers, and unnamed cross defendants ("Does 1-400") operating wells or surface water diversions within the watershed. A comprehensive adjudication of surface, subsurface and groundwater affecting the Ventura River was one of the nine claims of relief in a second amended cross complaint filed on September 24, 2018. In November 2018, the Ventura River Water District and Meiners Oaks Water District filed a motion for stay pending completion of the GSP for the Upper Ventura River Groundwater Basin. A hearing on the stay was removed from the calendar following a November 13, 2018 court order granting Complex Case Designation. The venue for the case has now been shifted to Los Angeles County Superior Court. Unless stayed in the near future, the adjudication case would proceed in parallel with GSP development by the UVRGA.

4.6.3 Development of Minimum Flow Requirements by the State Board

Both the above court case and the activities of the UVRGA are occurring in parallel with the State Board's development of minimum flow requirements for the Ventura River by virtue of its selection as one of the five priority stream systems being addressed through the California Water Action Plan. As part of this process, the CDFW is conducting a study to determine the flows they believe are required to 1) maintain hydrologic connectivity for steelhead life stages throughout the mainstem of the Ventura River, and 2) to support survival, movement, and productive riffle habitat for steelhead in San Antonio Creek. In parallel, the Instream Flow Unit of the State Board's Division of Water Rights has funded a consultant to develop an integrated hydrologic surface water-groundwater model that will be used (among other tools) by the State Board to develop its management plan to achieve reasonable minimum flows in the Ventura River. Originally, this model was expected to be available for use by the UVRGA to satisfy DWR's modeling requirements for GSPs. However, the UVRGA is now contemplating development of other modeling tools given their recent recognition that the State Board's model will not be released in time to be used for the Upper Ventura River basin GSP. Meanwhile, the UVRGA is conducting monitoring and data evaluation independent of CDFW's in-stream flow study. It remains unclear (even to State Board and DWR representatives) how conflicting conclusions about minimum flows and permissible levels of groundwater pumping arising from GSPs, adjudication, and the State Board's instream flows project will be resolved.

4.6.4 Significance of Flow Regulation for OVSD

The State Board's determination of required minimum flows in various reaches of the river will provide an authoritative basis for evaluating the role of OVSD effluent in maintaining habitat downstream of the outfall. Small changes in base flow resulting from regulation of surface flows and/or groundwater pumping could be significant compared to OVSD's effluent flow in certain months. Historic data for flow in the lower Ventura River, and the average of the OVSD "flow subsidy," was examined in detail in TM 3.

As explained at the beginning of this section, the amount of base flow in the lower Ventura River can affect the attainment of WQS and TMDL targets, and can affect whether reasonable potential (and the need for new numeric effluent limits) is triggered by receiving water concentrations. However, the direct (or indirect) regulation of surface flows (through one or more of the regulatory processes described above) will also potentially set limits on whether, how much, or at what times of the year OVSD might be allowed to remove effluent from the river in the future (e.g., for recycling or other re-use). Rough time frames for the three regulatory processes to conclude are compared in Table 4.11. Based on the comparison, the UVRGA's identification of safe yield, and associated management actions, will probably be the first "tranche" of policy affecting expectations for surface flows reaching Foster Park. However, actions by the UVRGA could theoretically be trumped by State Board minimum flow requirements after the UVRGA submits its first GSP. Furthermore, adjudication (which would unlikely conclude before the State Board determines its policy for the Ventura River) could theoretically overturn policy developed by both the UVRGA and the State Board.

Regardless of the timing or the vehicle for flow regulation, it is not necessarily true that OVSD will bear the brunt of the responsibility to maintain flows in the lower Ventura River, even during the drier months. As the analysis of flow in TM 3 showed, OVSD effluent makes a large contribution to instream flows during August, September, October, and November. However, a future obligation of surface water diverters (and possibly groundwater diverters) to support higher base flows in the reaches above OVSD's discharge could theoretically lead to a surplus of flow in Reaches 1-2, providing greater leeway for OVSD to remove some (if perhaps not all) of its discharge from the river. Any predictions are highly speculative until, at a minimum, the CDFW renders its conclusions about flows needed to support steelhead (see above) as part of the State Board process described above.

Table 4.11 General Time Frames for Regulatory Processes That Could Alter Surface Flows in the Lower Ventura River

Process	Likely Time Frame
Regulation of Pumping by the UVRGA	Post-2022(1)
State Board Establishment of Required Minimum Flows	TBD(2)
Physical Solution Imposed through Adjudication of Surface and Groundwater Diversions	Probably no earlier than 2029

Notes:
(1) Date represents the deadline for the submission of the first GSP for the Upper Ventura River Groundwater Basin to DWR.
(2) LWA has contacted the State Board Instream Flows supervisor for a time frame estimate, but the value was not available for the current draft.

4.7 Conclusions

Key conclusions from the review are briefly outlined below:

- Adoption of new or updated EPA or State Board water quality criteria:
  - Adoption of new EPA human health criteria would trigger reasonable potential for the seven constituents in Table 4.5 for which OVSD does not currently have numeric effluent limits.

  - New, more stringent, aquatic life criteria for ammonia, and selenium would result in revised permit limits for OVSD, but are unlikely to pose compliance problems.
  - Calculations using the BLM would be needed to conclude whether new EPA copper criteria would be met by OVSD effluent.
  - Other more stringent new or updated USEPA or State Board water quality criteria that could be adopted in Region 4 in the next few years appear to be comfortably met at OVSD's receiving water monitoring stations and in OVSD effluent. Barring changes in effluent and receiving water quality, it is not likely that the new standards will result in numeric effluent limits for OVSD.
- State Board Biostimulatory Substances Amendment:
  - A reopened Algae TMDL or a new Benthic Effects TMDL for reaches below the OVSD discharge may lead to re-evaluation of OVSD's discharges of TN and TP. The possibility hinges on ongoing exceedances of the algal biomass and DO targets from the Algae TMDL in the lower river, and explicit and stringent expectations for N and P concentrations that may be included in the State Board's Biostimulatory Substances Amendment to the ISWEBE Plan.
- Implementation of the Advanced Protection Management Program for OWTS:
  - The required reduction in OWTS loading in the Algae TMDL equates to a cessation of discharge from roughly 340 unsewered households.
  - Based on current nitrogen removal efficiencies, additional influent from 340 new residential connections would use up about 5 percent of the May-September TN load assigned to OVSD effluent in the Algae TMDL.
  - Although implementation of the OWTS policy through an Advanced Protection Management Program for the Ventura River Watershed would not change OVSD's effluent limits for TN and TP, OVSD's ability to accept new connections from currently unsewered households will need to be evaluated in light of its ability to meet upcoming final waste load allocations in the Algae TMDL with a sufficient margin of safety.
- Regulation of surface flows:
  - It's difficult to predict whether regulation of surface flows in reaches above the OVSD outfall will provide leeway for OVSD to petition the State Board to remove some of its discharge from the river.
  - It's theoretically possible that adjudication could lead to decreases in base flow in the Ventura River upstream of the OVSD discharge. If so, receiving water concentrations of some pollutants might increase. However, inspection of the data in Table 4.4 suggests that with the exception of selenium, priority pollutants detected in receiving water are well below existing Region 4 criteria or new EPA criteria that might be adopted into the Basin Plan in the next several years.
- Salts:
  - Receiving water and OVSD effluent currently easily meet the surface water objectives for salt constituents below the discharge, but the objective for chloride in the lower river is high compared to other reaches in Ventura County and higher than values usually cited appropriate for salt sensitive crops.
  - Unless there was a Basin Plan Amendment to change the surface water objectives for salts in Reach 2, 303(d) listings for salt constituents for Reach 2 (and thus a Salts TMDL) are unlikely.

  - Measures to address salt loading to groundwater in an SNMP for the Lower Ventura River Basin would also require a Basin Plan Amendment and be preceded by a series of actions and studies providing OVSD with abundant opportunities to comment or participate as a stakeholder.
- State Board Toxicity Provisions:
  - Going forward, OVSD will need to track outcomes of using the new TST metric for toxicity tests conducted by OVSD at R-4 and R-5, and those conducted by other agencies below the discharge, in case new data sets provide impetus for adding Reaches 1-2 to a future toxicity TMDL for Reach 3.
  - Wasteload allocations in a toxicity TMDL would not necessarily rely on OVSD's existing TST-based effluent limit, but could implicate specific priority pollutants detected in OVSD effluent.

Appendix 4A

COMPARISON OF NEW AND UPDATED USEPA WATER QUALITY CRITERIA FOR PRIORITY POLLUTANTS WITH WATER QUALITY CRITERIA USED IN REGION 4

Table 4.A New or Updated Aquatic Life and Human Health Water Quality Criteria published by USEPA starting in 2000, and corresponding CTR and California Primary MCLs. All criteria are expressed as µg/L unless otherwise noted. Updated USEPA Ammonia and Selenium criteria are omitted from the table, but discussed in the text

Constituent	California Toxics Rule[1]						New or Updated USEPA Criteria – Published after May 2000							CA Primary MCLs[2]
	Aquatic Life				Human Health for Consumption of		Aquatic Life				Human Health for Consumption of		Publ. Year	
	Freshwater		Saltwater				Freshwater		Saltwater					
	CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)	Water + Organism	Organism Only	CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)	Water + Organism	Organism Only		
1,1-Dichloroethylene	-	-	-	-	0.057	3.2	-	-	-	-	300	20,000	2015	0.6
1,1,1-Trichloroethane	-	-	-	-			-	-	-	-	10,000	20,000	2015	200
1,1,2-Trichloroethane	-	-	-	-	0.6	42	-	-	-	-	0.55[3]	8.9[3]	2015	5
1,1,2,2-Tetrachloroethane	-	-	-	-	0.17	11	-	-	-	-	0.2[3]	3[3]	2015	1
1,2-Dichlorobenzene	-	-	-	-	2,700	17,000	-	-	-	-	1,000	3,000	2015	600
1,2-Dichloroethane	-	-	-	-	0.38	99	-	-	-	-	9.9[3]	650[3]	2015	0.5
1,2-Dichloropropane	-	-	-	-	0.52	39	-	-	-	-	0.90[3]	31[3]	2015	5
1,2-Diphenylhydrazine	-	-	-	-	0.04	0.54	-	-	-	-	0.03	0.2	2015	-
1,2-Trans-Dichloroethylene	-	-	-	-	700	140,000	-	-	-	-	100	4,000	2015	10
1,2,4-Trichlorobenzene	-	-	-	-			-	-	-	-	0.71	0.076	2015	5
1,3-Dichlorobenzene	-	-	-	-	400	2,600	-	-	-	-	7	10	2015	-
1,3-Dichloropropylene	-	-	-	-	10	1,700	-	-	-	-	0.27[3]	12[3]	2015	0.5
1,4-Dichlorobenzene	-	-	-	-	400	2,600	-	-	-	-	300	900	2015	5
2-Chloronaphthalene	-	-	-	-	1,700	4,300	-	-	-	-	800	1,000	2015	-
2-Chlorophenol	-	-	-	-	120	400	-	-	-	-	30[4]	800[4]	2015	-
2-Methyl-4,6-Dinitrophenol	-	-	-	-	13.4	765	-	-	-	-	2	30	2015	-
2,3,7,8 TCDD (Dioxin)	-	-	-	-	0.013 pg/L	0.014 pg/L	-	-	-	-	0.005 pg/L	0.051 pg/L	2002	30 pg/L
2,4-Dichlorophenol	-	-	-	-	93	790	-	-	-	-	10[4]	60[4]	2015	-
2,4-Dimethylphenol	-	-	-	-	540	2,300	-	-	-	-	100[4]	3,000[4]	2015	-
2,4-Dinitrophenol	-	-	-	-	70	14,000	-	-	-	-	10	300	2015	-
2,4-Dinitrotoluene	-	-	-	-	0.11	9.1	-	-	-	-	0.049[3]	1.7[3]	2015	-
2,4,6-Trichlorophenol	-	-	-	-	2.1	6.5	-	-	-	-	1.5[3][4]	2.8[3][4]	2015	-
3-Methyl 4-Chlorophenol	-	-	-	-			-	-	-	-	500[4]	2,000[4]	2015	-
3,3 Dichlorobenzidine	-	-	-	-	0.4	0.077	-	-	-	-	0.049[3]	0.15[3]	2015	-
4,4'-DDD	-	-	-	-	0.00083	0.00084	-	-	-	-	0.00012[3]	0.00012[3]	2015	-
4,4'-DDE	-	-	-	-	0.00059	0.00059	-	-	-	-	0.000018[3]	0.000018[3]	2015	-
4,4'-DDT	1.1	0.001	0.13	0.001	0.00059	0.00059	-	-	-	-	0.000030[3]	0.000030[3]	2015	-
Acenaphthene	-	-	-	-	1,200	2,700	-	-	-	-	70[4]	90[4]	2015	-
Acrolein	-	-	-	-	320	780	3	3	-	-	3	400	2015[5]	-
Acrylonitrile	-	-	-	-	0.059	0.66	-	-	-	-	0.061[3]	7.0[3]	2015	-
Aldrin	3	-	1.3	-	0.00013	0.00014	-	-	-	-	0.00000077[3]	0.00000077[3]	2015	-

Table 4.A New or Updated Aquatic Life and Human Health Water Quality Criteria published by USEPA starting in 2000, and corresponding CTR and California Primary MCLs. All criteria are expressed as µg/L unless otherwise noted. Updated USEPA Ammonia and Selenium criteria are omitted from the table, but discussed in the text (continued)

Constituent	California Toxics Rule[1]						New or Updated USEPA Criteria – Published after May 2000							CA Primary MCLs[2]
	Aquatic Life				Human Health for Consumption of		Aquatic Life				Human Health for Consumption of		Publ. Year	
	Freshwater		Saltwater		Water + Organism	Organism Only	Freshwater		Saltwater		Water + Organism	Organism Only		
	CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)			CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)				
alpha HCH	-	-	-	-	0.0039	0.013	-	-	-	-	0.00036[3]	0.00039[3]	2015	-
alpha-Endosulfan	0.22	0.056	0.034	0.0087	100	240	-	-	-	-	20	30	2015	-
Anthracene	-	-	-	-	9,600	110,00	-	-	-	-	300	400	2015	-
Antimony, TR	-	-	-	-	14	4,300	-	-	-	-	5.6 9[6]	640[6]	2015	6
Benzene	-	-	-	-	1.2	71	-	-	-	-	0.58-2.1[3]	16-58[3]	2015	1
Benzidine	-	-	-	-	0.00012	0.00054	-	-	-	-	0.00014[3]	0.011[3]	2015	-
Benzo(a)Anthracene	-	-	-	-	0.0044	0.049	-	-	-	-	0.0012[3]	0.0013[3]	2015	-
Benzo(a)Pyrene	-	-	-	-	0.0044	0.049	-	-	-	-	0.00012[3]	0.00013[3]	2015	0.2
Benzo(b)Fluoranthene	-	-	-	-	0.0044	0.049	-	-	-	-	0.0012[3]	0.0013[3]	2015	-
Benzo(k)Fluoranthene	-	-	-	-	0.0044	0.049	-	-	-	-	0.012[3]	0.013[3]	2015	-
beta HCH	-	-	-	-	0.014	0.046	-	-	-	-	0.0080[3]	0.014[3]	2015	-
beta-Endolsulfan	0.22	0.056	0.034	0.0087	110	240	-	-	-	-	20	40	2015	-
Bis(2-Chloroethyl)Ether	-	-	-	-	0.031	1.4	-	-	-	-	0.030[3]	2.2[3]	2015	-
Bis(2-Chloroisopropyl)Ether	-	-	-	-	1,400	170,000	-	-	-	-	200	4,000	2015	-
Bis(2-Ethylhexyl)Phthalate	-	-	-	-	1.8	5.9	-	-	-	-	0.32[3]	0.37[3]	2015	4
Bromoform	-	-	-	-	4.3	360	-	-	-	-	7.0[3]	120[3]	2015	-
Butylbenzyl Phthalate	-	-	-	-	3,000	5,200	-	-	-	-	0.10[3]	0.10[3]	2015	-
Cadmium, TR[7]	4.5[8]	2.5[8]	42.3	9.36	-	-	1.8[8]	0.72[8]	33	7.9	2	-	2001 HH 2016 FW AL	5
Carbon Tetrachloride	-	-	-	-	0.25	4.4	-	-	-	-	0.4[3]	5[3]	2015	0.5
Chlordane	2.4	0.0043	0.09	0.004	0.00057	0.00059	-	-	-	-	0.00031[3]	0.00032[3]	2015	0.1
Chlorobenzene	-	-	-	-	680	21,000	-	-	-	-	100[4]	800[4]	2015	-
Chlorodibromomethane	-	-	-	-	0.41	34	-	-	-	-	0.80[3]	21[3]	2015	-
Chloroform	-	-	-	-	-	-	-	-	-	-	60	2,000	2015	-
Chrysene	-	-	-	-	0.0044	0.049	-	-	-	-	0.12[3]	0.13[3]	2015	-
Copper, TR[7]	14[8]	9.329[8]	5.8	3.73	1300	-	[9]	[9]	4.8[3]	3.1[3]	-	-	2007	1,300
Cyanide	22	5.2	1	1	700	220,000	-	-	-	-	4	400	2015	150
Di-n-Butyl Phthalate	-	-	-	-	2,700	12,000	-	-	-	-	20	30	2015	-
Dibenzo(a,h)Anthracene	-	-	-	-	0.0044	0.049	-	-	-	-	0.00012[3]	0.00013[3]	2015	-
Dichlorobromomethane	-	-	-	-	0.56	46	-	-	-	-	0.95[3]	27[3]	2015	-

Table 4.A New or Updated Aquatic Life and Human Health Water Quality Criteria published by USEPA starting in 2000, and corresponding CTR and California Primary MCLs. All criteria are expressed as µg/L unless otherwise noted. Updated USEPA Ammonia and Selenium criteria are omitted from the table, but discussed in the text (continued)

Constituent	California Toxics Rule[1]						New or Updated USEPA Criteria – Published after May 2000							CA Primary MCLs[2]
	Aquatic Life				Human Health for Consumption of		Aquatic Life				Human Health for Consumption of		Publ. Year	
	Freshwater		Saltwater		Water + Organism	Organism Only	Freshwater		Saltwater		Water + Organism	Organism Only		
	CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)			CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)				
Dieldrin	0.24	0.056	0.71	0.0019	0.00014	0.00014	-	-	-	-	0.0000012[3]	0.0000012[3]	2015	-
Diethyl Phthalate	-	-	-	-	23,000	120,000	-	-	-	-	600	600	2015	-
Dimethyl Phthalate	-	-	-	-	313,000	2,900,000	-	-	-	-	2,000	2,000	2015	-
Endosulfan Sulfate	-	-	-	-	110	240	-	-	-	-	20	40	2015	-
Endrin	0.086	0.036	0.037	0.0023	0.76	0.81	-	-	-	-	0.03	0.03	2015	2
Endrin Aldehyde	-	-	-	-	0.76	0.81	-	-	-	-	1	1	2015	-
Ethylbenzene	-	-	-	-	3,100	29,000	-	-	-	-	68	130	2015	300
Fluoranthene	-	-	-	-	300	370	-	-	-	-	20	20	2015	-
Fluorene	-	-	-	-	1,300	14,000	-	-	-	-	50	70	2015	-
gamma HCH (lindane)	0.95	-	0.16	-	0.019	0.063	-	-	-	-	4.2	4.4	2015	0.2
Heptachlor	0.52	0.0038	0.053	0.0036	0.00021	0.00021	-	-	-	-	0.0000059[3]	0.0000059[3]	2015	0.01
Heptachlor Epoxide	0.52	0.0038	0.053	0.0036	0.0001	0.00011	-	-	-	-	0.000032[3]	0.000032[3]	2015	0.01
Hexachlorobenzene	-	-	-	-	0.00075	0.00077	-	-	-	-	0.000079[3]	0.000079[3]	2015	1
Hexachlorobutadiene	-	-	-	-	0.44	50	-	-	-	-	0.01[3]	0.01[3]	2015	-
Hexachlorocyclopentadiene	-	-	-	-	240	17,000	-	-	-	-	4[4]	4[4]	2015	50
Hexachloroethane	-	-	-	-	1.9	8.9	-	-	-	-	0.1[3]	0.1[3]	2015	-
Indeno(1,2,3-cd)Pyrene	-	-	-	-	0.0044	0.049	-	-	-	-	0.0012[3]	0.0013[3]	2015	-
Isophorone	-	-	-	-	8.4	600	-	-	-	-	34[3]	1,800[3]	2015	-
Methyl Bromide	-	-	-	-	48	4,000	-	-	-	-	100	10,000	2015	-
Methylmercury												0.3 mg/kg	2001(12)	
Methylene Chloride	-	-	-	-	4.7	1,600	-	-	-	-	20[3]	1,000[3]	2015	-
N-Nitrosodi-n-Propylamine	-	-	-	-	0.005	1.4	-	-	-	-	0.0050[3]	0.51[3]	2002	-
N-Nitrosodimethylamine	-	-	-	-	0.00069	8.1	-	-	-	-	0.00069[3]	3.0[3]	2002	-
N-Nitrosodiphenylamine	-	-	-	-	5	16	-	-	-	-	3.3[3]	6.0[3]	2002	-
Nitrobenzene	-	-	-	-	17	1,900	-	-	-	-	10[4]	600[4]	2015	-
Pentachlorophenol	23.83(10)	18.28(10)	13	7.9	0.28	8.2	-	-	-	-	0.03[3][4]	0.04[3][4]	2015	1
Phenol	-	-	-	-	21,000	4,600,000	-	-	-	-	4,000[4]	300,000[4]	2015	-
Polychlorinated biphenyls	-	0.014	-	0.03	0.00017	0.00017	-	-	-	-	0.000064[3](11)	0.000064[3](11)	2002	0.5
Pyrene	-	-	-	-	960	11,000	-	-	-	-	20	30	2015	-
Tetrachloroethylene	-	-	-	-	0.8	8.85	-	-	-	-	10[3]	29[3]	2015	5

Table 4.A New or Updated Aquatic Life and Human Health Water Quality Criteria published by USEPA starting in 2000, and corresponding CTR and California Primary MCLs. All criteria are expressed as µg/L unless otherwise noted. Updated USEPA Ammonia and Selenium criteria are omitted from the table, but discussed in the text (continued)

Constituent	California Toxics Rule[1]						New or Updated USEPA Criteria – Published after May 2000							CA Primary MCLs[2]
	Aquatic Life				Human Health for Consumption of		Aquatic Life				Human Health for Consumption of		Publ. Year	
	Freshwater		Saltwater		Water + Organism	Organism Only	Freshwater		Saltwater		Water + Organism	Organism Only		
	CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)			CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)				
Thallium, TR	-	-	-	-	1.7	6.3	-	-	-	-	0.24	0.47	2003	2
Toluene	-	-	-	-	6,800	200,000	-	-	-	-	57	520	2015	150
Toxaphene	0.73	0.0002	0.21	0.0002	0.00073	0.00075	-	-	-	-	0.00070[3]	0.00071[3]	2015	3
Trichloroethylene	-	-	-	-	2.7	81	-	-	-	-	0.6[3]	7[3]	2015	5
Vinyl Chloride	-	-	-	-	2	525	-	-	-	-	0.022[3]	1.6[3]	2015	0.5
Zinc, TR	119.82[8]	120[8]	95	85.6	-	-	-	-	-	-	7,400	26,000	2002	-

Notes:
(1) Table only lists CTR Aquatic Life or Human Health criteria for which new or updated USEPA criteria have been published starting in 2000. Thus, the table does not list all CTR criteria. Updated USEPA criteria for ammonia and selenium are omitted from the table, but discussed in the text.
(2) California MCLs are only listed for constituents that have new or updated USEPA aquatic life or human health criteria. Table does not reflect all California MCLs.
(3) This criterion is based on carcinogenicity of 10-6 risk.
(4) This chemical has a criterion for organoleptic (taste and order) effects. In some cases, the organoleptic criterion may be more stringent.
(5) Publication year for the new USEPA criteria was 2009.
(6) This criterion was revised to reflect EPA's q1* or RfD as contained in the Integrated Risk Information System (IRIS) as of May 17, 2002. The fish tissue bioconcentration factor (BCF) is from the 1980 Ambient Water Quality Criteria document.
(7) Freshwater and saltwater criteria for metals are expressed in terms of the dissolved metal in the water column.
(8) The freshwater criterion for this metal is expressed as a function of hardness (mg/L) in the water column. The value given here corresponds to a hardness of 100 mg/L.
(9) Freshwater Copper Criteria calculated using the BLM.
(10) The freshwater criterion for this chemical is pH and temperature dependent. Value give here corresponds to pH=8.0, temp 20 degrees C.
(11) This criterion applies to total PCBs (e.g., the sum of all congener or all isomer or homolog or Aroclor analyses).
(12) The fish tissue residue criterion for methylmercury is based on a total fish consumption rate of 0.0175 kg/day.

Appendix 4B

USEPA WATER QUALITY CRITERIA FOR NON - PRIORITY POLLUTANTS PUBLISHED AFTER 1999

Table 4.B USEPA Aquatic Life and Human Health Water Quality Criteria published after 1999 for Non-Priority Pollutants, and corresponding California Primary MCLs. All criteria are expressed as µg/L unless otherwise noted

Constituent	New or Updated USEPA Criteria - Published 2000 onward							CA Primary MCL[(2)]
	Aquatic Life[(1)]				Human Health for Consumption of		Publ. Year	
	Freshwater		Saltwater		Water + Organism	Organism Only		
	CMC (acute)	CCC (chronic)	CMC (acute)	CCC (chronic)				
1,2,4,5-Tetrachlorobenzene	-	-	-	-	0.03	0.03	2015	-
2,4,5-Trichlorophenol	-	-	-	-	300[(3)]	600[(3)]	2015	-
Bis(Chloromethyl) Ether	-	-	-	-	0.00015[(4)]	0.017[(4)]	2015	-
Chlorophenoxy Herbicide (2,4- D)	-	-	-	-	1,300	12,000	2015	70
Chlorophenoxy Herbicide (2,4,5-TP) [Silvex]	-	-	-	-	100	400	2015	50
Dinitrophenols	-	-	-	-	10	1,000	2015	-
Hexachlorocyclohexane (HCH) - Technical	-	-	-	-	0.0066[(4)]	0.010[(4)]	2015	30
Methoxychlor	-	-	-	-	0.02	0.02	2015	30
Nitrosodibutylamine, N	-	-	-	-	0.0008[(4)]	1.24[(4)]	2002	-
Nitrosodiethylamine, N	-	-	-	-	0.016[(4)]	34[(4)]	2002	-
Nitrosopyrrolidine, N	-	-	-	-	0.016[(4)]	34[(4)]	2002	-
Pentachlorobenzene	-	-	-	-	0.1	0.1	2015	-
Carbaryl	2.1	2.1	1.6	-	-	-	2012	-
Diazinon	0.17	0.17	0.82	0.82	-	-	2005	-
Nonylphenol	28	6.6	7	1.7	-	-	2005	-
Tributyltin (TBT)	0.46	0.072	0.42	0.0074	-	-	2004	-

Notes:

(1) Table only lists Aquatic Life Criteria for non-Priority pollutants for which new or updated USEPA Criteria have been published starting in 2000.

(2) California MCLs are only listed for constituents that have new or updated USEPA aquatic life or human health criteria. Table does not reflect all California MCLs.

(3) This chemical has a criterion for organoleptic (taste and order) effects. In some cases, the organoleptic criterion may be more stringent.

(4) This criterion is based on carcinogenicity of 10-6 risk.

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 5
TMDL IMPLEMENTATION AND FACILITIES UPGRADES

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Ojai Valley Sanitary District
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TMDL IMPLEMENTATION AND FACILITIES UPGRADES

5.1 Introduction

This technical memorandum (TM) is the fifth in a series of six TMs that will form the basis of the 20-year Facilities Plan for Ojai Valley Sanitary District (OVSD). This TM includes development and evaluation of alternatives to address both the short-term and long-term treatment objectives at the wastewater treatment plant (WWTP). The planning horizon for the Facilities Plan is 2039.

Short-term objectives focus on the need to address modifications driven by OVSD's 2018 National Pollutant Discharge Elimination System (NPDES) Permit. Although the WWTP routinely meets the numeric concentration limits in the NPDES Permit, the permit also includes a Total Maximum Daily Load (TMDL) for nitrogen and phosphorus removal, which comes into effect in June 2025. Short-term objectives also include improving the energy efficiency of the plant. Four process upgrade alternatives were developed and evaluated to address the short-term objectives.

Long-term objectives for the WWTP focus on future regulatory considerations that could potentially be implemented within the planning horizon of the Facilities Plan. Long-term objectives include meeting any new requirements arising from a TMDL reopener or State-wide nutrient limits, effluent salinity reduction, and implementation of a recycled water program.

5.2 Key Findings and Recommendations

The key findings and recommendations are:

- Five treatment alternatives were developed for meeting the short-term 2025 TMDL. Four of these include use of one or both of the existing oxidation ditches. Due to the concerns with the condition of the concrete in the oxidation ditches and whether the structures will last another 20 years, a fifth alternative was developed, which would construct a new aeration basin designed to meet current and future operating conditions and the effluent limits required in the TMDL.
- Each alternative was evaluated for its ability to meet the TMDL limits, as well as deal with higher flow and loads during high flow events in the winter. Each configuration was stressed to determine its performance for treating a flow of 6 mgd for six consecutive days, at loads 50-percent greater than average conditions. These conditions were expected to simulate a large winter storm event.
- Alternative 1 (combining both existing ditches into a single 5-stage Bardenpho system) had the lowest estimated 20-year life-cycle costs, of $14.4 million, and an estimated construction cost of $8.4 million. However, this alternative lacked redundancy, making impractical as a long-term solution. Accordingly, a hybrid of Alternatives 1 and 2, called

Alternative 1A, was developed, to include the benefits of Alternative 1 and the redundancy features of Alternative 2.

- Alternative 1A includes using both ditches in series during the winter months in a 5-stage Bardenpho configuration, with the flexibility to take one ditch out of service in the summer months. When one ditch is out of service, the required TMDL limits would be achieved via polishing in denitrification filters with a capacity of 2 mgd. This would provide a high level of flexibility to plant operations.
- Because of its ability to makes use of all existing facilities and incorporate good process redundancy and operational flexibility, Alternative 1A, with an estimated construction cost of $16.1 million, was selected as the preferred alternative.
- In order to assess site space considerations for the long term, out to 2039, three future regulatory scenarios were considered:
  - Future State-wide discharge limits for nitrogen and phosphorous and/or reasonably anticipated future limits arising from a TMDL reopener driven by the Los Angeles Regional Board.
  - Effluent salinity reduction needs for chloride.
  - Implementation of a recycled water program.
- Facilities requiring the largest footprint would be those associated with an indirect potable reuse (IPR) project incorporating full advanced treatment (FAT) and brine concentration.
- Future site layouts were developed for implementing a future IPR program using the four treatment alternatives to meet the TMDL as the base. The layouts were compared to assess ease of operation, siting constraints, ease of maintenance, logical flow, etc.

5.3 Flow and Load Projections

Three growth scenarios were used to develop flow and load projections. Scenarios are:

- Base Case - 0.3 percent annual growth in population equivalents or sewer connections.
- 500 Septic Conversions - 0.3 percent annual growth and 100 septic conversions a year for five years.
- 1,000 Septic Conversions - 0.3 percent annual growth and 100 septic conversions a year for 10 years.

Since the 2018 annual dry-weather flow (ADWF) was 1.6 mgd and there were 12,175 sewer connections, it is estimated that each connection generates 131 gallons ADWF. Table 5.1 summarizes the flow projections for the different scenarios.

Table 5.1 ADWF Projections for Planning Period

Year	Base Case(1)	500 Septic Conversions(1)	1,000 Septic Conversions(1)
2018	1.61	1.61	1.61
2023	1.63	1.69	1.69
2028	1.66	1.72	1.78
2033	1.68	1.75	1.81
2038(2)	1.70	1.77	1.83

Notes:
(1) Based on 131 gallons per sewer connection and septic conversion.
(2) Buildout for Facilities Plan.

The difference between the base case and 1,000 septic conversions is 0.13 mgd, or approximately 7 percent of the buildout ADWF. To ensure the full impact of potential septic conversions is captured, the evaluation is based on the 1,000 septic conversion scenario, with a buildout ADWF of 1.83 mgd.

Table 5.2 summarizes the flow and load projections for buildout ADWF, and maximum month (MM) conditions. It was assumed the wastewater characteristics at ADWF and MM would be the same as what was observed in 2018. The projections in Table 5.2 will be used for evaluating the alternatives in this TM.

Table 5.2 Flow and Load Projections for Planning Period Buildout

Parameter	Units	ADWF Condition(1)	MM Condition(2)
Flow	mgd	1.83	2.01
BOD_5	mg/L	359	450
	lbs/day	5,500	7,540
COD	mg/L	854	1070
	lbs/day	13,000	17,930
TSS	mg/L	393	450
	lbs/day	6,000	8,740
TKN	mg/L	53.6	61.6
	lbs/day	820	1,030
Ammonia	mg/L	34.5	39.7
	lbs/day	530	670
Total P	mg/L	7.6	8.9
	lbs/day	120	150
Soluble P	mg/L	3.8	4.5
	lbs/day	60	70

Notes:
(1) Based on 2018 average dry weather wastewater characteristics.
(2) Based on 2018 90th percentile concentrations; reference Table 1.1 in TM 1.
(3) Concentrations based on the load during peak wet weather being the same as average dry weather.

5.4 Short-Term Objectives

This section summarizes the alternatives development and evaluation to achieve the short-term treatment objectives of the WWTP. Key objectives are to meet the requirements in the 2018 NPDES Permit, which include TMDL requirements for total nitrogen (TN) and total phosphorus (TP) in the plant discharge coming into effect in June 2025. Another objective is to improve the energy efficiency of the plant. Construction cost estimates for the alternatives were developed, as well as life-cycle operating and maintenance costs. The evaluation of alternatives considered meeting the TMDL requirements both during dry-weather and summer flow conditions and during conditions of high flow. In addition, "stress" tests to evaluate performance during extended storm periods with elevated loads are also considered. This evaluation will help OVSD with selecting a preferred alternative.

5.4.1 Short-Term Effluent Requirements

OVSD's 2018 NPDES Permit includes numeric concentration limits and a TMDL for nitrogen and phosphorus removal, which comes into effect in June 2025. The numeric concentration limits range from 1.9 to 10 mg/L for various nitrogen species and 2.6 mg/L for phosphorus. However, the effluent concentrations required to achieve the TMDL limits are lower and will establish the nitrogen removal requirements in the short term. The TMDL requirements are 5,799 pounds per day (lbs/day) of TP in dry weather and 8,044 lbs/day of TN in the summer season. Table 5.3 summarizes the effluent concentrations required to achieve TMDL limits for the three growth scenarios in Table 5.1. Note that the calculations are based on the flow projections in Table 5.1 and include a 15-percent safety factor.

Table 5.3 Effluent Nutrient Requirements for TMDL

Year	Base Case(1)	500 Septic Conversions(1)	1,000 Septic Conversions(1)
Total Phosphorus, mg/L(1)	0.96	0.92	0.89
Total Nitrogen, mg/L(2)	3.15	3.03	2.93

Notes:
(1) Based on 5,799 lbs/day in dry weather (lowest flow for three consecutive months).
(2) Based on 8,044 lbs/day in summer season (May through September).

Although the different growth scenarios result in slightly different requirements, the WWTP will need to meet TN and TP limits of 3 and 0.9 mg/L, respectively. This is considered to be the limit of technology for biological treatment.

5.4.2 Short-Term Alternative Description

This section provides a brief description, schematic, and site plan of the short-term alternatives. Initially, four alternatives were developed, which are discussed below. After the initial comparison of the alternatives, a fifth hybrid alternative (Alternative 1A) was developed. This alternative is discussed in Section 5.4.4.

5.4.2.1 Alternative 1 – 5-Stage Bardenpho in Combined Ditch Configuration

Alternative 1 consists of converting the existing treatment process from a 3- to 5-stage Bardenpho process. This is accomplished by adding a post-anoxic and aerobic zone at the end of a 3-stage Bardenpho. This process performs better than the 3-stage Bardenpho because the post-anoxic and aerobic zone are able to achieve additional nitrogen removal and meet the desired target of 3 mg/L. Since there is little soluble carbon remaining by the time the mixed liquor reaches the post-anoxic (or second anoxic zone), supplemental carbon such as Micro C would be needed in that zone. To avoid constructing new structures, Oxidation Ditch No. 1 (OD 1) will be converted to the post-anoxic and aerobic zones by installing interconnecting piping so mixed liquor from the aerobic zone in Oxidation Ditch No. 2 (OD 2) would flow to the post-anoxic zone in OD 1. Instead of operating the oxidation ditches in parallel as OVSD currently does, the process will now be configured as one train. The ditches would continue to be mixed and aerated with similar equipment in the racetrack configuration. However, the condition assessment (see TM 6) indicated that all mechanical aerators would need to be replaced. For configurations involving the ditches it was assumed that the aerators would be replaced with units that include variable frequency drives (VFDs), which would improve the energy efficiency. Figures 5.1 and 5.2 are a schematic and site plan of this alternative, respectively.

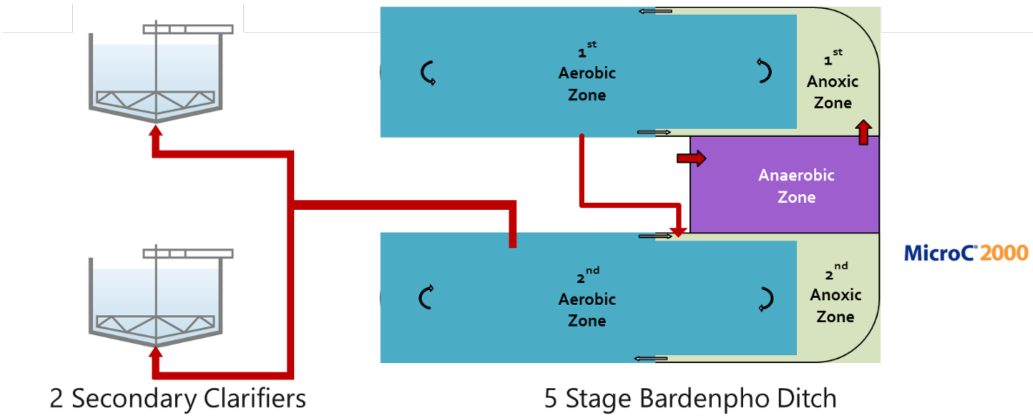

Figure 5.1 Alternative 1 Schematic

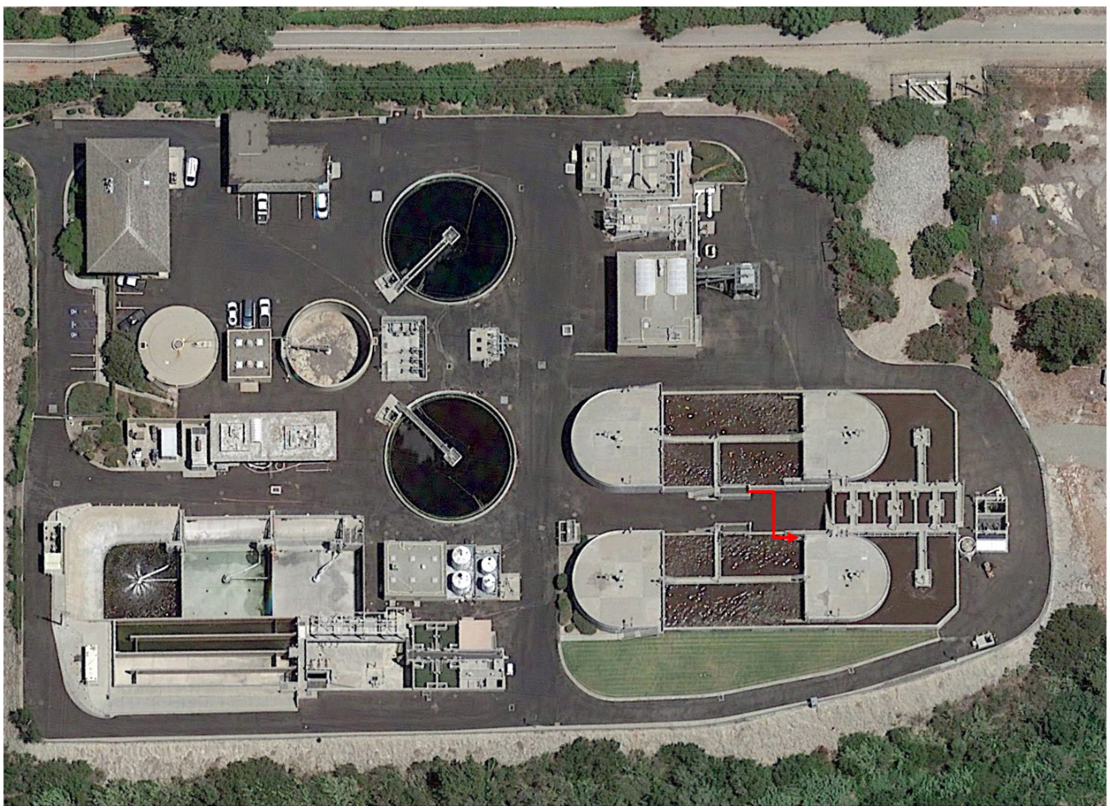

Figure 5.2 Alternative 1 Site Plan

5.4.2.2 Alternative 2 – 3-Stage Bardenpho Ditches + Denitrification Filters

Alternative 2 consists of continuing to operate the existing 3-stage Bardenpho secondary process and adding post denitrification filters to achieve additional nitrogen removal. Although there are different configurations, denitrification filters are typically deep-bed media filters designed to promote biological growth in anoxic conditions. Since there is little soluble carbon in the secondary effluent, supplemental carbon will be needed at the filters. The filtered effluent will be of sufficient quality to meet NPDES permit limits, and some filter equipment manufacturers are certified for producing effluent of a quality that meets Title 22 Recycled Water. It is anticipated that the denitrification filters may be able to replace the existing filters, which could then be abandoned or demolished. Depending on the filter manufacturer selected, the performance of the denitrification filters would have to be demonstrated to meet Title 22 before the existing filters could be shut down. For the purposes of this planning document, it has been assumed that both sets of filters would be needed, at least in the short term. The denitrification filters could be constructed where the unused anaerobic digesters are located. Figures 5.3 and 5.4 are a schematic and site plan of this alternative, respectively.

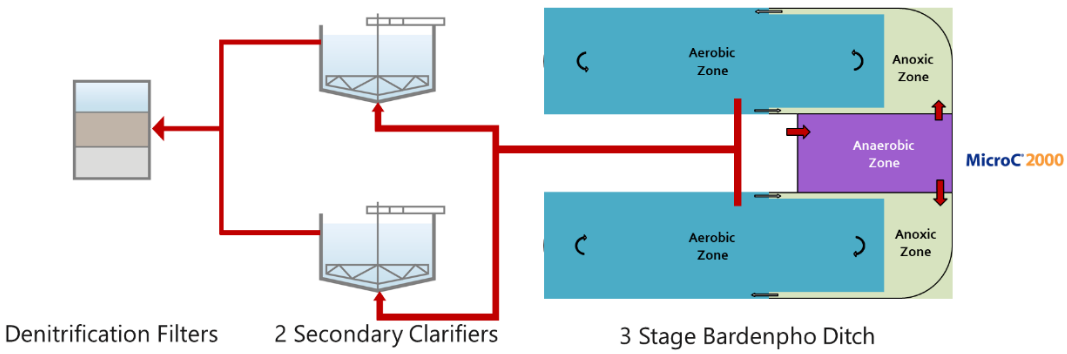

Figure 5.3 Alternative 2 Schematic

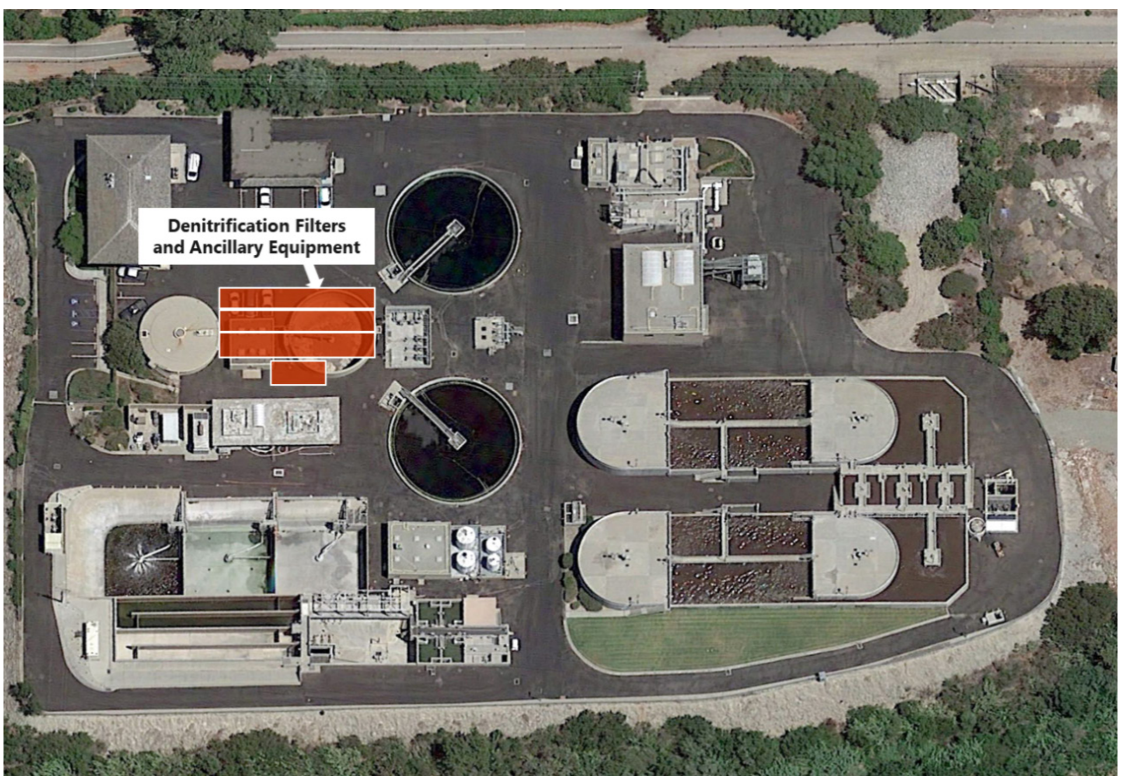

Figure 5.4 Alternative 2 Site Plan

5.4.2.3 Alternative 3 – 5-Stage Bardenpho in Aeration Basin Configuration

Alternative 3 consists of converting OD 1 from a 3- to 5-stage Bardenpho process. Same as in Alternative 1, this is accomplished by adding a post-anoxic and aerobic zone at the end of a 3-stage Bardenpho. This will be incorporated into OD 1 by converting the existing "race-track" ditch to a plug flow aeration basin by adding baffles and mixers. The existing aerators/mixers will also be replaced with fine-bubble diffusers and mixed-liquor recirculation pumps. The treatment is more efficient in the plug flow configuration, which means the process can operate at a lower solids retention time (SRT) than what is needed for a racetrack configuration. As a result, OD 2 would no longer be needed (or used), and the WWTP would only be operating one process train. A disadvantage of operating with only one of the ditches, however, is that the mixed-liquor concentrations will be higher than Alternatives 1 and 2, and one more secondary clarifier with return activated sludge/waste activated sludge (RAS/WAS) pumping is needed to handle wet-weather flows. High-efficiency turbo-style blowers would supply air for the fine-bubble diffusers, and they would be housed in a new blower building. Although the existing mechanical aerators are effective, fine bubble diffusers are more efficient and use significantly less power. Figures 5.5 and 5.6 show a schematic and site plan of this alternative, respectively.

Note that since OD 1 has been identified to have concrete structural issues (see TM 6), this alternative could be accomplished by converting OD 2, rather than OD 1. The required modifications and outcome would be the same.

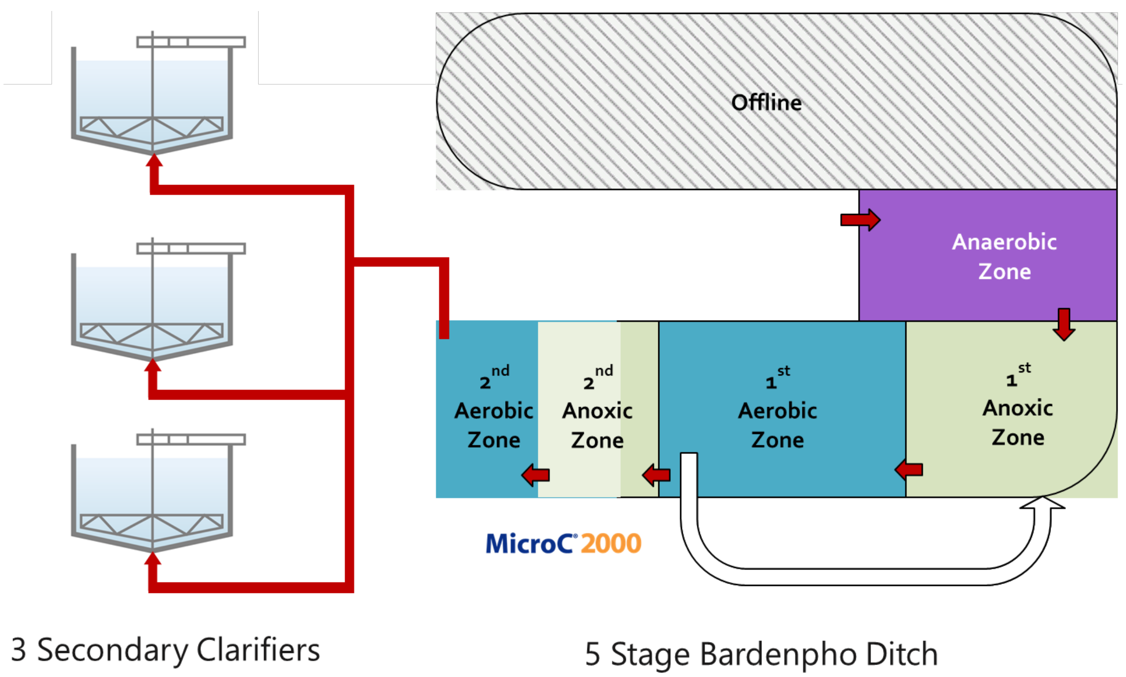

Figure 5.5 Alternative 3 Schematic

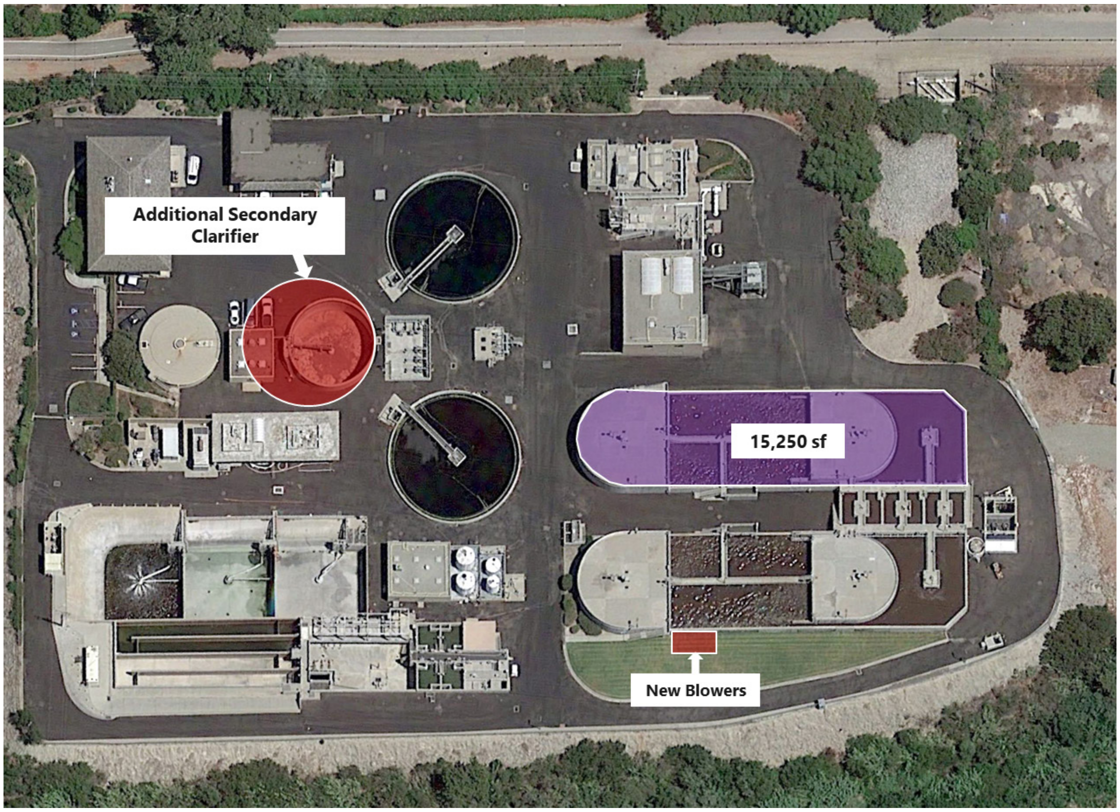

Figure 5.6 Alternative 3 Site Plan

5.4.2.4 Alternative 4 – New 5-Stage Bardenpho Process

Alternative 4 consists of decommissioning the existing ditches and constructing a new 5-stage Bardenpho process. It would be a single new aeration basin with fine-bubble diffusers. The new aeration basin would be large enough so that the existing secondary clarifiers would not be overloaded during wet-weather conditions, and the basin would be designed so that one zone could be taken out of service during dry weather while the rest of the basin remains in operation. Because construction of a new aeration basin is likely to take more than one year, it would not be possible to demolish one of the ditches and construct the new basin in its place. Thus, it would need to be constructed in a different location to the ditches. Figures 5.7 and 5.8 are a schematic and site plan of this alternative, respectively. The site plan shows that the new aeration basin would be constructed in the area currently occupied by the odor control biofilter. The biofilter would be moved to a new location, as shown on Figure 5.8.

This configuration would free up a significant amount of the site for future facilities, such as a new headworks or equalization basins, etc.

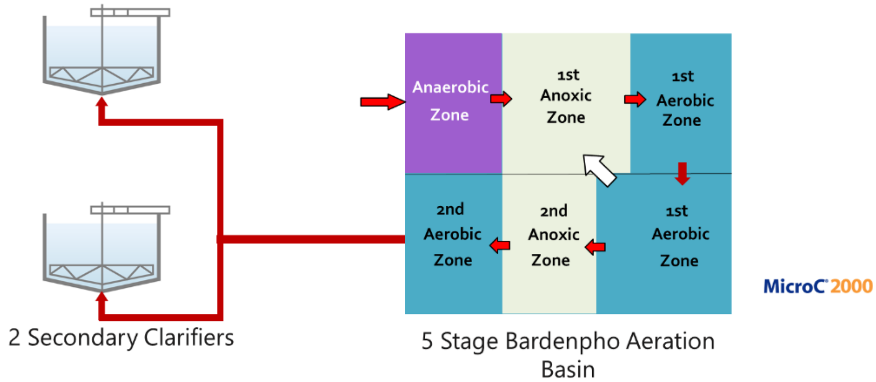

Figure 5.7 Alternative 4 Schematic

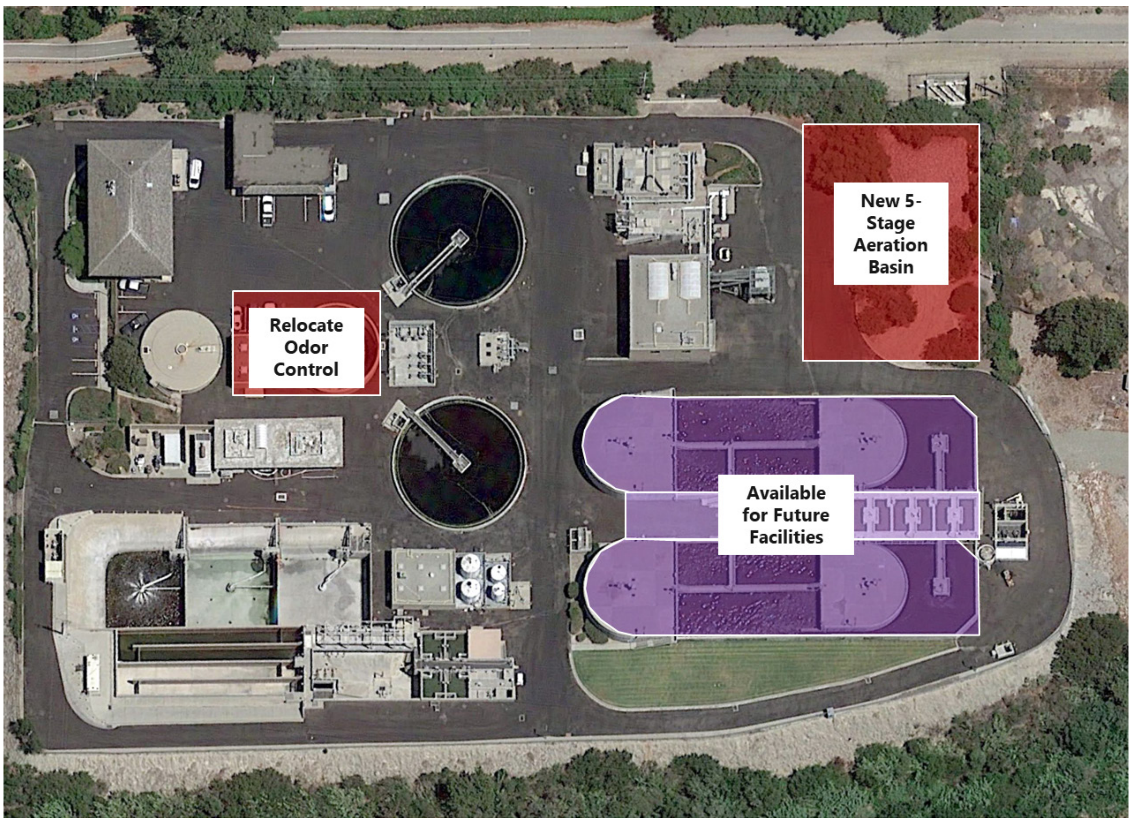

Figure 5.8 Alternative 4 Site Plan

5.4.2.5 Alternative Criteria Summary at Buildout

Influent characteristics used for the modeling are summarized in Table 5.2. Table 5.4 summarizes the alternative design criteria and performance at buildout.

Table 5.4 Short-Term Alternative Design Criteria at Buildout

Item	Units	Alternative 1 5-Stage Bardenpho in Combined Ditch Configuration	Alternative 2 3-Stage Bardenpho Ditches + Denitrification Filters	Alternative 3 5-Stage Bardenpho in Aeration Basin Configuration	Alternative 4 New 5-Stage Bardenpho Process
Bioreactors					
Numbers	-	1	2	1(1)	1(1)
Volume, Each	MG	3.5	1.75	1.87	2.3
Total Volume in service	MG	3.5	3.5	1.87	2.30
Aeration Type	-	Surface Aerators	Surface Aerators	Fine-Bubble Diffusers	Fine-Bubble Diffusers
Oxygen Demand	lbs/hr (field)	338	330	283	316
Average Power Usage	kWh/day	3,000	3,000	1,900	2,000
Target SRT at MM Load	days	22	22	11	14
MLSS at MM Design Condition	mg/L	3,100	3,100	3,900	3,000
Secondary Clarifiers					
Number	-	2	2	3	2
Size, each	ft	85	85a	85	85
Solids Loading Rate at PWWF(2)	lbs/sf/d	21	21	17	20
Denitrification Filters					
Number	-	-	2+2	-	
Size, each	ft x ft	-	11.67 x 100.0	-	
Area, total	sf		2,334 (duty) + 2,334 (standby)		
Duty Filter Loading at ADWF	gpm/sf		0.5		
Chemical Usage					
Bioreactors	gal/day	25	-	60	40
Denitrification Filters	gal/day	-	150	-	-
Total	gal/day	25	150	60	40

Notes:
Abbreviations: ft=feet, gal/day=gallons per day, kWh/day=kilowatt hour per day, lbs/hr=pounds per hour, lbs/sf/d=pounds per square foot per day, MG=million gallons, PWWF=peak wet-weather flow
(1) Designed with ability to take zones out of service.
(2) Assumes 50-percent RAS at PWWF.
(3) Assumes SRT control and stable SVI averaging at approximately 130 mL/g.

5.4.2.6 Stress Test Results

In addition to simulating planning period buildout conditions summarized in Table 5.2, the process model was used to simulate extended storm periods (one week) with elevated loads, assumed to be 50-percent greater than average day loads. This was performed as OVSD has experienced such events in recent history. A conservative approach to evaluating this is to model the peak flow of 6 mgd under steady-state conditions and determine how much load each alternative could accommodate before one of the following occurs:

- Effluent TN or TP exceeds 3 or 1 mg/L, respectively.
- Mixed-liquor concentration is too high for secondary clarifiers to accommodate at 6 mgd.

A solids-flux state point analysis was used to determine how high mixed-liquor concentrations could be. A reasonable worst-case sludge volume index (SVI) of 130 milliliters per gram (mL/g) was used, which reflects the average of historical data from 2018. The solids flux analysis results in mixed-liquor concentrations ranging from 4,000 to 5,300 mg/L, which is higher than recommended for normal, sustained operation. However, for a one-week period, it may be acceptable.

It was also assumed that the SRT could be reduced during this period down to 10 days for all alternatives. While not ideal, 10 days should provide sufficient inventory to ensure effluent TN and TP concentrations stay within desired target of 3 and 1 mg/L, respectively.

The approach is conservative in some ways because the steady-state model predicts results assuming the system reaches equilibrium with the input parameters selected. However, if the storm takes places for one week, the mixed liquor concentration will not reach equilibrium within that time, which means the mixed liquor concentration will not be as high as stated here. Conversely, the process performance may not be as good as is predicted with a steady-state model as temporary steps up in loads usually result in temporary bleed through of some parameters. This effect could also be exacerbated if the SRT is being reduced during this time from the normal range of 14 to 22 days down to 10 days. To more accurately predict performance, dynamic modeling should be performed during the design phase of the project.

Upstream flow equalization would help to even out the load and reduce the stress on the process, but given the high volume it would be challenging to accommodate full equalization on the plant site so this was not evaluated. Without flow equalization, the stress test represents a conservative assessment of the anticipated performance of the alternatives.

Results of the stress test simulations are summarized in Table 5.5.

Table 5.5 Stress Test Simulation Summary(1)

Item	Units	Alternative 1 5-Stage Bardenpho in Combined Ditch Configuration	Alternative 2 3-Stage Bardenpho Ditches + Denitrification Filters	Alternative 3 5-Stage Bardenpho in Aeration Basin Configuration	Alternative 4 New 5-Stage Bardenpho Process
Target SRT	days	10	10	10	10
Aeration Basin Volume in Service	Million gallons	3.5	3.5	1.87	2.30
Number of 85-Foot Diameter Secondary Clarifiers in Service	--	2	2	3	2
Allowable MLSS at Peak Flow(2)	mg/L	4,000	4,000	5,300	4,000
Solids Loading Rate at Peak Flow(3)	lbs/sf/d	27	27	23	27
Effluent Quality					
Ammonia	mg/L	0.24	0.73	1.55	0.3
TN	mg/L	2.02	2.93	2.94	2.68
TP	mg/L	0.87	0.72	0.16	0.82
Peak Load					
BOD_5	lbs/day	12,600	12,600	8,200	8,200
Ammonia	lbs/day	1,400	1,400	800	800
Peak Load	% of ADW Load	230%	230%	150%	150%

Notes:
(1) Stress test conditions at 150% of average loads and 6 mgd flow.
(2) Based on reasonable worst-case SVI of 130 mL/g, 6-mgd flow, and number of secondary clarifiers in service.
(3) Assumes 50-percent RAS at PWWF.

All of the alternatives were able to accommodate at least 150 percent of the average dry-weather load. Alternatives 1 and 2, which have more bioreactor volume in service were predictably able to accommodate even higher loads at 230 percent of the average dry-weather load. Although all alternatives were able to meet effluent quality requirements, Alternative 3 is showing signs of ammonia breakthrough, as the effluent ammonia is higher than predicted for the other alternatives. This is because Alternative 3 has the least amount of bioreactor volume.

It should be noted that the nitrogen removal achieved with the alternatives is close to the limit of technology for biological nutrient removal. Further reduction in nitrogen would require advanced technologies such as reverse osmosis.

5.4.3 Alternative Comparison

5.4.3.1 Capital and Life-Cycle Costs

Capital, operation and maintenance (O&M), and life-cycle costs were developed for each alternative. Costs shown in this TM do not include capital or O&M costs that are common to all alternatives, and do not include elements that may be captured as part of the Facilities Plan condition assessment. The purpose of this estimate is so that alternative costs can be evaluated and compared to each other.

Costs are presented in 2019 dollars and are not escalated to future years. Costs were prepared in accordance with the guidelines of Association for the Advancement of Cost Engineering International (AACE International) 18R-97 for a Class 5 estimate.

Construction cost estimates include direct and indirect costs. Direct costs include materials, labor, construction equipment required for installation, and subcontractor costs. Indirect costs include contractor general conditions, contractor overhead and profit, sales tax, and an estimating contingency of 25 percent.

Direct construction costs were estimated from various references. Where possible, the costs from design estimates or construction bid tabs were used and converted to current dollars. Other cost sources included Carollo Engineers, Inc. (Carollo) reference projects, the R.S. Means price catalog, Carollo's Unit Price catalog, and vendor quotes for major pieces of equipment. The total project capital cost was estimated as the total construction cost plus an additional allowance of 35 percent for engineering, legal, administration, and permitting cost.

Table 5.6 summarizes the alternative cost comparison. Appendix 5A includes detailed cost estimates.

Table 5.6 Alternative Cost Comparison

Description	**Alternative 1** 5-Stage Bardenpho in Combined Ditch Configuration	**Alternative 2** 3-Stage Bardenpho Ditches + Denitrification Filters	**Alternative 3** 5-Stage Bardenpho in Aeration Basin Configuration	**Alternative 4** New 5-Stage Bardenpho Process
Preliminary Construction Cost Estimate	**$8,400,000**	**$17,190,000**	**$17,700,000**	**13,000,000**
Allowance for Engineering, Legal, Admin etc. (35%)	**$2,940,000**	**$6,017,000**	**$6,195,000**	**$4,550,000**
Total Annual Power and Supplemental Carbon Cost(1)	**$183,000**	**$275,000**	**$126,000**	**132,000**
Annual Power Cost(2)	$165,000	$165,000	$104,000	110,000
Supplemental Carbon(3)	$18,000	$110,000	$22,000	22,000

Description	**Alternative 1** 5-Stage Bardenpho in Combined Ditch Configuration	**Alternative 2** 3-Stage Bardenpho Ditches + Denitrification Filters	**Alternative 3** 5-Stage Bardenpho in Aeration Basin Configuration	**Alternative 4** New 5-Stage Bardenpho Process
Present Worth of Power and Carbon Cost(4)	**$3,090,000**	**$4,600,000**	**$2,100,000**	**$2,200,000**
Total Present Worth(5)	**$14,430,000**	**$27,807,000**	**$25,995,000**	**$19,750,000**

Notes:
(1) Only considered power and supplemental carbon as anticipate equipment replacement and labor to be similar amongst all alternatives.
(2) Based on $0.15 per kilowatt hour (kWh).
(3) Assumed Micro-C at $1.97 per gallon.
(4) Present Worth Power and Carbon calculated using total annual cost, 20-year analysis period, and 6-percent discount rate and 3-percent inflation.
(5) Total Present Worth = Preliminary Capital Cost Estimate + Present Worth Power and Carbon.

For calculating the energy costs associated with each alternative, it was assumed for Alternatives 1 and 2 that the existing dual-speed mechanical aerators would be replaced with new VFD units, which would improve their energy efficiency. The present-worth analysis in Table 5.6 shows that Alternative 1 is expected to have the lowest 20-year life-cycle cost, followed by Alternative 4. Alternatives 2 and 3 are expected to have the highest life-cycle costs.

5.4.3.2 Non-Economic Factors

All four process configurations would be able to achieve the desired effluent quality goals in order to meet the TMDL. Other non-economic factors that can be used to compare the alternatives are shown in Table 5.7. Relative energy consumption and O&M costs have also been included in the table for completeness. The comparison is shown in terms of +/-/0 rankings. "+" generally indicates a good quality relative to the particular comparison criteria, "-" generally indicates a poorer result, and "0" represents a neutral condition. One factor considered was the ability to address the alkali silica reactivity (ASR) that has been identified in some of the structures, including both ditches, which needs to be addressed. More details are presented in TM 6.

Table 5.7 Comparison of Alternatives

Comparison Criteria	**Alternative 1** 5-Stage Bardenpho in Combined Ditch Configuration	**Alternative 2** 3-Stage Bardenpho Ditches + Denitrification Filters	**Alternative 3** 5-Stage Bardenpho in Aeration Basin Configuration	**Alternative 4** New 5-Stage Bardenpho Process
Energy Consumption	-	-	0	0
Chemical Usage	0	-	0	0
Address ASR Issue(1)	+	+	+	++
Process Robustness	+	+	+	+

Comparison Criteria	**Alternative 1** 5-Stage Bardenpho in Combined Ditch Configuration	**Alternative 2** 3-Stage Bardenpho Ditches + Denitrification Filters	**Alternative 3** 5-Stage Bardenpho in Aeration Basin Configuration	**Alternative 4** New 5-Stage Bardenpho Process
Process Redundancy	-	++	+	+
Frees Up Space for Future Facilities	-	-	+	++
Sized to Meet Future Average Conditions	-	-	0	+
O&M Cost	-	--	0	+

Notes:
(1) See TM 6 for details of the ASR issues that impact some concrete structures built in 1994.

Overall, this analysis indicates that the alternative with the most preferred ranking would be Alternative 4, followed by Alternative 3. Alternatives 1 and 2 had the lowest ranking. A major concern regarding Alternative 1 is its lack of process redundancy, since it is not possible to take one of the ditches out of service without losing significant treatment capacity. Redundancy would be provided in Alternatives 3 and 4 through the provision of zones that can be taken out of service.

Following discussion of the above results with OVSD in May 2019, it was decided that rather than construct a new basin dedicated to the current TMDL (Alternative 4), it would be preferable to make use of the existing facilities as far as possible. Accordingly, a fifth alternative, Alternative 1A, was developed and is discussed in the following section.

5.4.4 Development of Hybrid Alternative 1A

Alternative 1 was the lowest cost alternative, based on the evaluation presented above, which makes use of most of the existing infrastructure. However, it lacked redundancy because if one of the two ditches is taken out of service, it would not be possible to meet the TMDL limits. This represented a fatal flaw for Alternative 1. On the other hand, Alternative 2, which also uses most of the existing infrastructure and includes permanent denitrification filters and good process redundancy, would be the most costly alternative.

A hybrid alternative that incorporates the benefits of Alternatives 1 and 2 might provide the best approach. Accordingly, Alternative 1A was developed and evaluated, and is discussed in this section.

Alternative 1A would operate as Alternative 1 during the wet season. That is, the ditches would be operated in series as a combined 5-stage Bardenpho process to achieve the TMDL limits even during high flow events. Smaller capacity denitrification filters would be provided to allow one ditch to be taken out of service during the summer months. In this configuration, the operating ditch would become a 3-stage Bardenpho ditch, as it is today, and the denitrification filters would provide polishing to remove residual nitrate. Process modeling has shown that this configuration will achieve the TMDL limits during the summer months. Figure 5.9 shows a schematic of Alternative 1A. The dotted line indicates the flow configuration to the denitrification filters when one ditch is out of service.

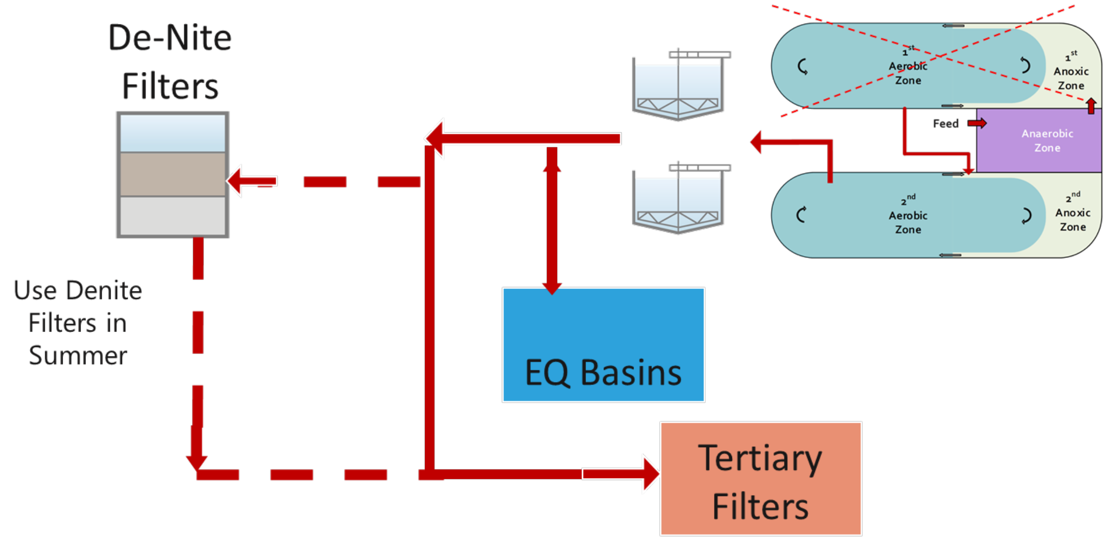

Figure 5.9 Schematic of Hybrid Alternative 1A

A preliminary site layout for Alternative 1A is shown on Figure 5.10. As indicated, the denitrification filters are located to the west of the westerly ditch on the existing grass area. The capacity of the filters would be 2 mgd, to meet the anticipated maximum daily flow conditions during summer. The layout also shows new chemical storage facilities to the south of the filters. These would replace the existing temporary Micro-C storage and dosing system located south of the anaerobic zone.

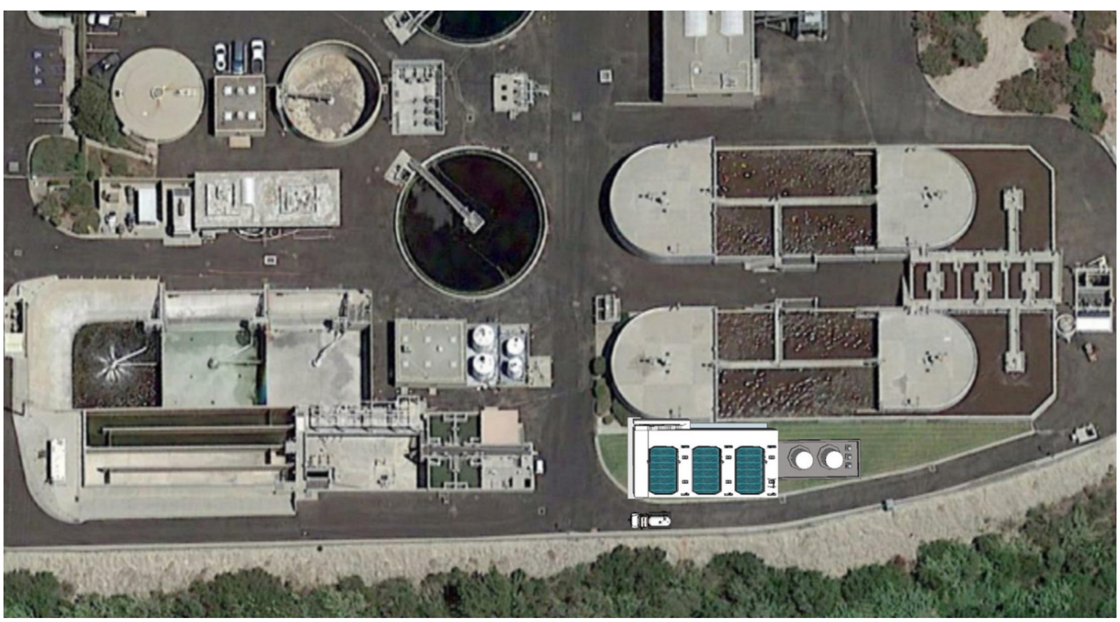

Figure 5.10 Preliminary Layout for Hybrid Alternative 1A

5.4.4.1 Proposed Piping Configuration for Alternative 1A

OVSD would like to retain maximum flexibility for operation of the denitrification filters. That is, the flexibility for the flow from the denitrification filters to be re-filtered through the tertiary filters, or be bypassed directly to the ultraviolet (UV) system. The existing tertiary filter influent pump station pumps secondary effluent from a wet well to the tertiary filters, from where flow gravitates to the open-channel UV disinfection system. The flowrate through the filters and UV is limited to about 4.3 mgd.

Current Approach to Dealing with Flow Downstream of the Clarifiers

For average day conditions, the plant typically operates one of the three vertical turbine pumps to transfer flow to the tertiary filters, and the pumps are set to deliver 1.8 mgd. The pump station wet well is hydraulically connected to the three equalization (EQ) basins. Excess flow beyond the pump setpoint flows into the EQ basins and returns by gravity when the secondary effluent flow drops.

During winter, when the average daily flows are higher, a second pump is brought into service as needed, to control the wet well level. The normal operating condition is to have one of the four tertiary filters off line. Under these conditions the maximum flowrate is 4.3 mgd. However, during wet weather events, the average flows to the plant could be 7-mgd or higher. If the plant brings the fourth filter on line and operates all three pumps, the plant reports that it can handle a flow of between 7.2 and 7.4 mgd. At this point the post-chlorination system would be brought into service. If the flow goes beyond 7.2 to 7.4 mgd then a hose is dropped into the filter feed pump wet well and a portable pump is used to transfer flow to a pipe that directs the flow across the gulley north of the Operations Building and into the sludge drying/composting area. If this addition transfer is not able to cope with the incoming flow another hose is dropped into one of the EQ basins and a second portable pump is operated to also transfer flow to the sludge drying/composting area. Up to 3 MG can be stored in the solids area. Any water that is pumped into this area is able to gravitate back to the headworks. With this configuration the plant has been able to handle a peak of up to about 10 mgd. No modifications to this arrangement are planned.

Proposed Denitrification Filter Configuration

The proposed layout shown on Figure 5.10 incorporates three 25 ft long filters (the shortest available from the manufacturer), each able to treat about 1 mgd. This provides for 2-mgd of average summer time flow, with one filter spare. For the purposes of the Facilities Plan and the cost estimate, three filters will be provided. During preliminary design, it can be decided whether two denitrification filters will be adequate.

Hydraulics

The hydraulics of existing facilities downstream of the tertiary filter pump station were checked to confirm the capacity of the system. The hydraulics were then re-run with the denitrification filters in place to confirm that the desired flow flexibility can be achieved. A schematic of the system arrangement is shown on Figure 5.11.

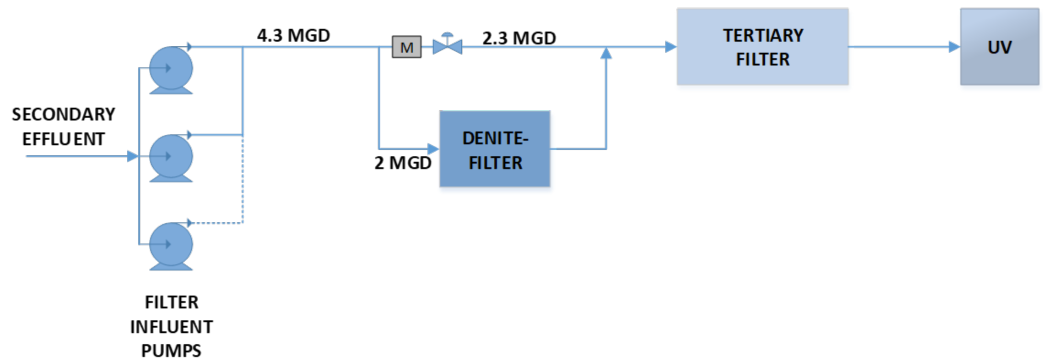

Figure 5.11 Schematic of Facility Arrangement to Accommodate Denitrification Filters for Alternative 1A

Hydraulically, the most challenging condition will be to have both the tertiary filters and the denitrification filters operating and the tertiary filters treating all the denitrification filter effluent, as illustrated on Figure 5.11. Figure 5.12 shows the partial hydraulic profile for this condition, and indicates that this is achievable if the weir on the outlet from the denitrification filters is set not lower than 205.5 feet. This would put the top of the concrete elevation of the filters a 213.75 feet, which is close to the elevation of the top of the westerly ditch.

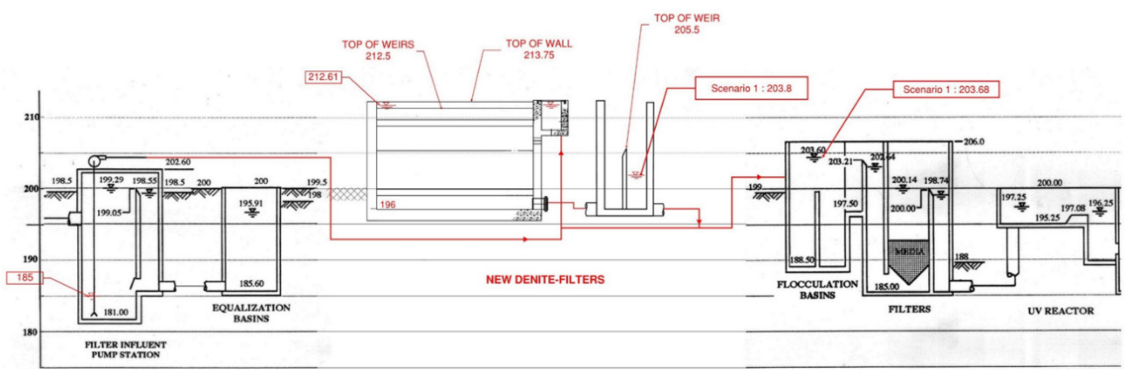

Figure 5.12 Preliminary Hydraulic Profile Showing Denitrification Filters for Alternative 1A

Pump Evaluation

A preliminary evaluation of the pumping capacity of the existing tertiary effluent feed pumps was carried out. The design criteria for the existing pumps are shown in Table 5.8. OVSD indicated that the motors on the pumps were reduced in size during the last plant expansion. The motor size for the original pumps is not known.

Table 5.8 Design Criteria for Existing Tertiary Filter Feed Pumps

Item	Description
Tertiary Filter Feed Pumps	
Type of Pump	Vertical Turbine
Drive Type	VFD
Number of Pumps	2 + 1
Design Duty Point	1,500 gpm at 25 feet
Motor Size, hp	15

The analysis determined that the existing pumps, with their 15-horsepower (hp) motors would be able to deliver a maximum of 2.3 mgd to the tertiary filters/denitrification filters. This would allow 2 mgd to pass through the denitrification filters, with 0.3 mgd bypassing, and the denitrification filter effluent being re-filtered in the tertiary filters. This arrangement is shown on Figure 5.13.

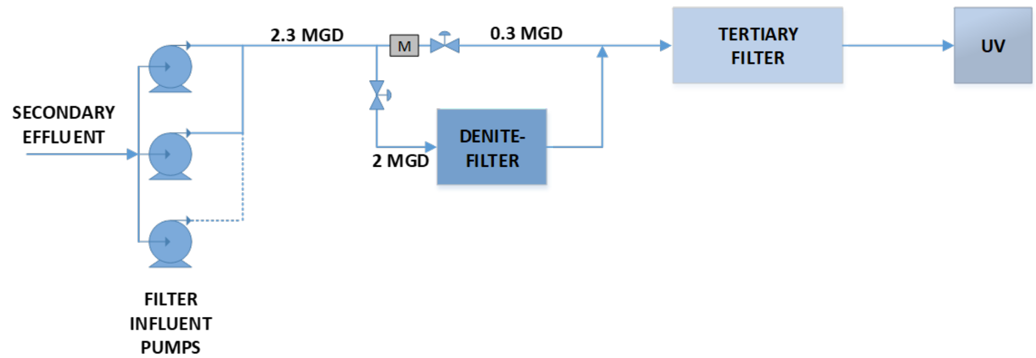

Figure 5.13 Maximum Flow Conditions for Series Operation of Denitrification and Tertiary Filters With Existing Pumps and 15-hp Motors

Further analysis showed that with all three pumps in operation, a maximum of 2.73 mgd could be delivered, with 2 mgd of that flow going to the denitrification filters and the rest bypassing directly to the tertiary filters. If the valve to the denitrification filters is shut, the system would return to the current configuration and be capable of delivering 7.2 to 7.4 mgd.

If the existing pumps were replaced with 20 hp pumps, then larger flows could be delivered to the tertiary filters while retaining the 2 mgd flow to the denitrification filters, which would provide additional flexibility for plant operation.

For the purposes of this planning document it was assumed that larger pumps would be provided. The final choice regarding the pumps can be made during the final design.

Preliminary Piping Arrangement

Figure 5.14 shows a schematic of the preliminary piping arrangement to connect the denitrification filters upstream of the existing tertiary filters. As shown, the existing feed line to the tertiary filters (yellow) will be intercepted downstream of the coagulant dosing point and the tertiary filter bypass line. A new flowmeter would be added at this location and then a tee will direct flow through a second flow meter and control valve (red line) to the denitrification filters. Flow from the denitrification filters will return to either the inlet to the flocculation basins, or to the box downstream of the tertiary filter channel to be directed to UV. Manual valves will control whether or not the denitrification filter effluent is re-filtered in the tertiary filters, or bypasses directly to UV disinfection.

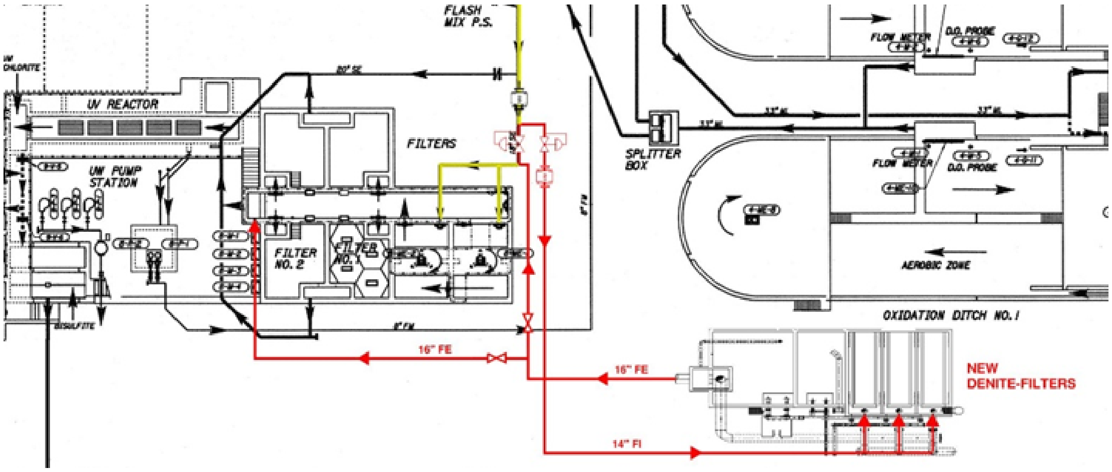

Figure 5.14 Schematic of Piping Arrangement for Alternative 1A

Figure 5.15 shows the proposed piping arrangement on the site plan.

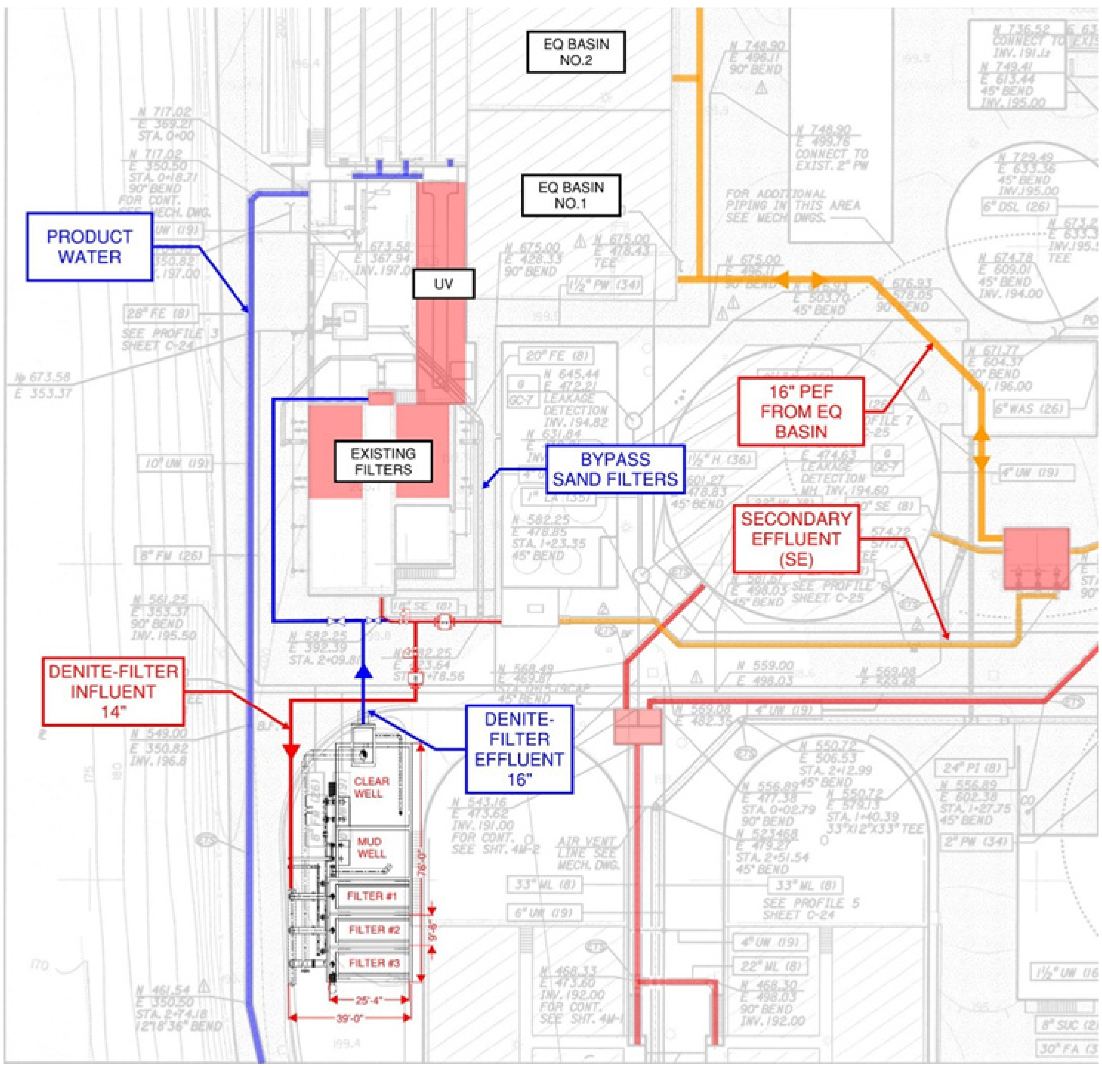

Figure 5.15 Preliminary Site Plan Arrangement for Alternative 1A

5.4.4.2 Cost Estimate for Alternative 1A

The estimated construction cost and project cost for Alternative 1A is presented in Table 5.9. Alternative 1A includes the modification required for Alternative 1, together with 2 mgd of denitrification filters. The cost also includes the modifications to the yard piping to include the new valve station and flow meters, larger tertiary filter feed pumps, as well as the new chemical storage and dosing facilities for Micro-C. As shown the construction cost is estimated to be about $16.1 million, with a total project cost estimate of $21.7 million. Detailed costs are included in Appendix 5A.

Table 5.9 Alternative 1A Cost Estimate

Description	Alternative 1A
5-Stage Bardenpho Ditch Modifications	$8,400,000
Denitrification Filters Modifications (including chemical storage)	7,707,500
Total Construction Cost	**$16,107,500**
Allowance for Engineering, Legal, Admin etc. (35%)	$5,638,000
Total Project Cost	**21,745,500**

5.4.4.3 Comparison of Costs

Table 5.10 presents a comparison of the planning level cost estimates for all five alternatives evaluated as part of TM 5. As shown, providing the process redundancy in Alternative 1A by adding 2 mgd of denitrification filters, increases the cost significantly compared with Alternative 1. However, the cost estimates for Alternative 1A are still lower than Alternatives 2 and 3. All alternatives that include the existing ditches (Alternatives 1, 1A, and 2) all include approximately $3.5 million for repairs to the ditches to address the ASR issues. The extent to which this allowance will be needed, will be determined during final design.

Table 5.10 Comparison of Planning Level Construction and Project Costs for all Five Alternatives

Description	Alternative 1 5-Stage Bardenpho in Combined Ditch Configuration	Alternative 1A Alt 1 + 2 mgd Denitrification Filters	Alternative 2 3-Stage Bardenpho Ditches + Denitrification Filters	Alternative 3 5-Stage Bardenpho in Aeration Basin Configuration	Alternative 4 New 5-Stage Bardenpho Process
Preliminary Construction Cost Estimate	$8,400,000	16,107,500	$17,190,000	$17,700,000	13,000,000
Allowance for Engineering, Legal, Admin etc. (35%)	$2,940,000	$5,638,000	$6,017,000	$6,195,000	$4,550,000
Total Project Cost Estimate	$11,340,000	$21,745,500	$23,207,000	$23,895,000	$17,550,000

Since it is preferable to make use of the existing facilities rather than build a new dedicated aeration basin and abandon the existing ditches (Alternative 4), the preferred alternative is Alternative 1A.

5.5 Long-Term Objectives

This section looks beyond 2025 to potential regulatory issues that may develop during the planning window of the Facilities Plan (that is to 2039). For each of the above alternatives, the objective was to evaluate the potential discharge limits that might be set, where applicable, and modifications that would be needed at the treatment plant to accommodate such limits. This high-level evaluation was aimed to establish site constraints, budget planning level costs, and potential plant configuration changes that would be needed.

This section summarizes the three main long-term regulations and considerations:

- Future State-wide discharge limits for nitrogen and phosphorus and/or reasonably anticipated future limits arising from a TMDL reopener driven by the Los Angeles Regional Board.
- Effluent salinity reduction needs for total dissolved solids (TDS) or a specific parameter, such as chloride.
- Implementation of a recycled water program.

5.5.1 Future State-Wide Discharge Limits for Nitrogen and Phosphorus

As discussed in Section 5.4, meeting the TMDL for TN and TP based on the future anticipated flow and loading to the WWTP, will require treating to an effluent TN and TP of 3 mg/L and 1 mg/L, respectively. Therefore, since these limits match the anticipated State-wide discharge limits and the plant will already be configured to achieve such limits in 2035, no further consideration was given to the State-wide discharge limits.

5.5.2 Effluent Salinity Reduction Needs

TM 4 discussed the regulatory environment and included a section that discussed the WWTP effluent salinity. The WWTP currently comfortably meets its numeric concentration-based effluent limits for salts (TDS, sulfate, chloride, and boron), which are set equal to the Basin Plan surface water objectives for Reach 2 of the Ventura River. Table 5.11 shows the numerical monthly limits in terms of concentration and mass (lbs/day), as well as the current average values measured in the WWTP effluent. As can be seen, the plant effluent is currently well below the discharge requirements for all parameters.

Table 5.11 Wastewater Effluent Salinity and Limits

Item	Unit	Discharge Limits (Average Monthly Values)	Current Average in WWTP Effluent(1)
TDS	mg/L	1,500	820
	lbs/day	38,000	12,515(2)
Sulfate	mg/L	500	228
	lbs/day	13,000	427(2)
Chloride	mg/L	300	153
	lbs/day	7,500	2,335(2)
Boron	mg/L	1.5	0.51
	lbs/day	38.0	7.8(2)

Notes:
(1) Average concentration calculated from four samples collected in 2018.
(2) Daily load (lbs/day) calculated using future average plant flow rate of 1.83 mgd.

It seems unlikely that a TDS limit will apply to the WWTP effluent based on the River discharge limits. However, chloride may be different. The surface water objective for chloride in the reach that the WWTP discharges to (300 mg/L) is much higher than those assigned to the reaches upstream (50 to 60 mg/L). For comparison, the highest surface water objective for chloride in neighboring watersheds is 150 mg/L, and this level may be considered too high by agricultural stakeholders for chloride sensitive crops. Also, the surface water objective for the reach that the WWTP discharges to (Reach 2), is an order of magnitude higher than the groundwater objective for the Lower Ventura River groundwater basin (30 mg/L chloride). Thus, it seems possible that during the planning horizon of the Facilities Plan, OVSD may be required to reduce the chloride concentration in the WWTP effluent.

For the purposes of this document, it was assumed that a new limit of 50 mg/L chloride would be implemented by the end of the planning period (2039). There are a limited number of ways in which chloride can be removed from water. Chloride salts are typically highly soluble, so precipitation of insoluble chloride compounds does not offer many possibilities. The most efficient way to remove chloride is via a desalting process such as reverse osmosis (RO), electrodialysis reversal (EDR), or ion exchange (IX).

5.5.2.1 Alternative A – Ion Exchange

IX is a unit process that uses ion selective resin to remove unwanted ions from water. Ions in the feed water are exchanged for ions on the resin. Resins are typically synthetic polymers with different functional groups depending on the application. For example, a porous crosslinked polyacrylic acid resin that has carboxylic acid functional groups, will be in the hydrogen ion (H^+) form initially. When feed water is passed through the resin bed, the H^+ ions will be displaced by other cationic (+ve) ions that have a greater affinity for attachment to the resin, such as calcium (Ca^{2+}). In this case, calcium would be removed from the water, and two hydrogen ions would be displaced for each Ca^{2+} ion removed. Other cations, such as magnesium (Mg^{2+}) and sodium (Na^+), would also be removed, depending on the relative affinity. This is an example of a weak acid cation (WAC) resin. A schematic of the IX process using WAC is shown on Figure 5.16.

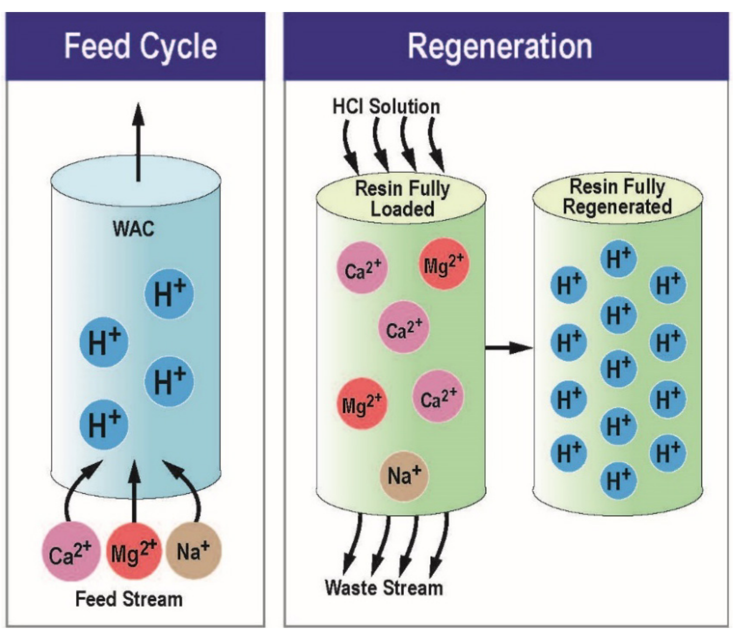

Figure 5.16 Schematic of Ion Exchange Process for WAC Resin

Most IX resins can be regenerated, although there are some intended for single use. Resin regeneration occurs when the resin is flushed with a solution that has a high concentration of the original ionic form of the resin. In the case of the WAC described above, the resin would be regenerated with a hydrochloric acid (HCl) solution, to provide a very high concentration of H^+ ions to displace the other cations that were attached to the resin during the feed cycle. The displaced ions would become part of the regen solution and would be captured and then discharged as a waste stream.

IX Process Description

In this case, with chloride as the target ion, a two-stage process is required. The first stage would be a WAC resin in the H^+ form, as described above, which would remove calcium from the feed stream and therefore soften the water. This is required so that scaling does not occur in the downstream second-stage vessel during regeneration. The second stage would use a strong base anion (SBA) resin in the hydroxide (OH^-) form. This resin would exchange chloride (Cl^-) for the OH^- ions, but would also remove other competing anions such as sulfate (SO_4^{2-}), nitrate (NO_3^-) and phosphate (PO_4^{3-}) to varying degrees. The first vessel would be regenerated with HCl. The second stage vessel would be regenerated with sodium hydroxide (NaOH).

To achieve the desired effluent blend of <50 mg/L chloride, about 75 percent of the effluent stream would need to be treated. Figure 5.17 illustrates this side stream treatment configuration schematically, and Figure 5.18 shows a preliminary process flow diagram for the system. Two vessels of each type would be provided to achieve continuous operation when vessels go into a regeneration cycle.

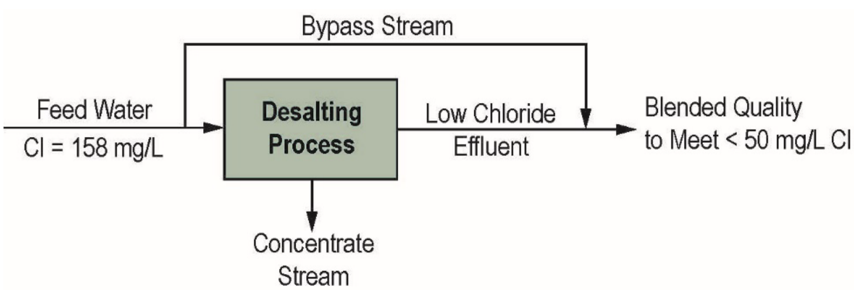

Figure 5.17 Schematic of Side-Stream Treatment for Chloride Reduction

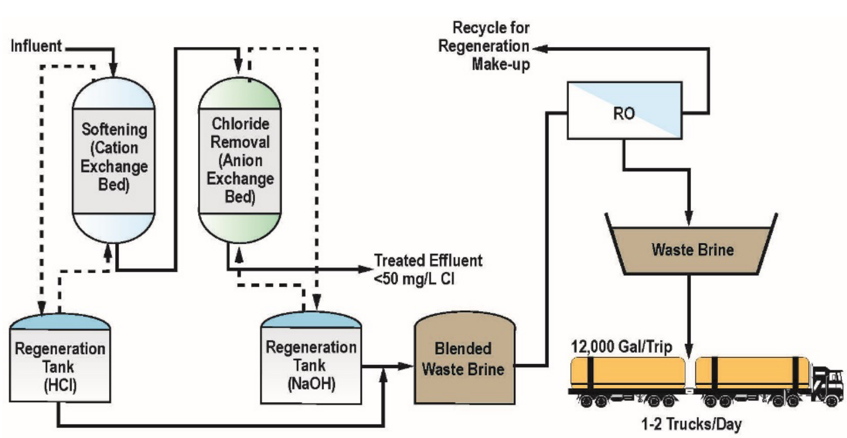

Figure 5.18 Preliminary Process Flow Diagram for IX System

Information from the resin supplier, Purolite, was used to develop preliminary design criteria for the IX system. These are summarized in Table 5.12. Based on the current secondary effluent quality, the vessels will require regeneration four to five times per day. The total volume of waste generated each day is expected to be about 100,000 gallons. In order to reduce the volume of the spent regen solution to a manageable volume for trucking, a small RO system has been provided. The RO unit would recover most of the solution as a high-quality permeate that would be recycled to make up more regen solution. The remainder (10 to 15 percent) would be discharged to one of the unused aerobic digester tanks for storage and pickup by tanker truck for disposal. Based on the values shown in Table 5.12, only one or two truckloads per day would be needed.

Table 5.12 IX System Preliminary Design Criteria

Item	Stage 1	Stage 2
Number of Vessels	2(1)	2(1)
Vessel Diameter, ft	12	12
Resin Type	WAC	SBA
Resin Depth, in	37	69
Resin Volume, ft(2)	350	650
Run Cycle, hr	5.1	5.1
Regen Solution	4% HCl	4% NaOH
Volume of Spent Regen, gal/d	50,000(2)	50,000(2)
Batch RO System Flow, gpm	150	
Batch RO System Recovery, %	85 - 90	
Recycled Volume, gal/d	85,000	
Waste for Disposal, gal/d	15,000	

Notes:
(1) One vessel in operation each cycle and the other in standby mode until a regeneration is required.
(2) Actual spent regen solution volumes would need to be verified during preliminary design

IX System Layout

A preliminary footprint for the IX system is shown on Figure 5.19. A total area of approximately 5,200 square feet would be needed to accommodate the four IX resin vessels, the chemical storage area, and the regen solution and waste tanks. The chemical storage area was based on an assumption of about seven days of storage of both HCl (35 percent) and NaOH (50 percent). Duty and standby RO units are included in the footprint.

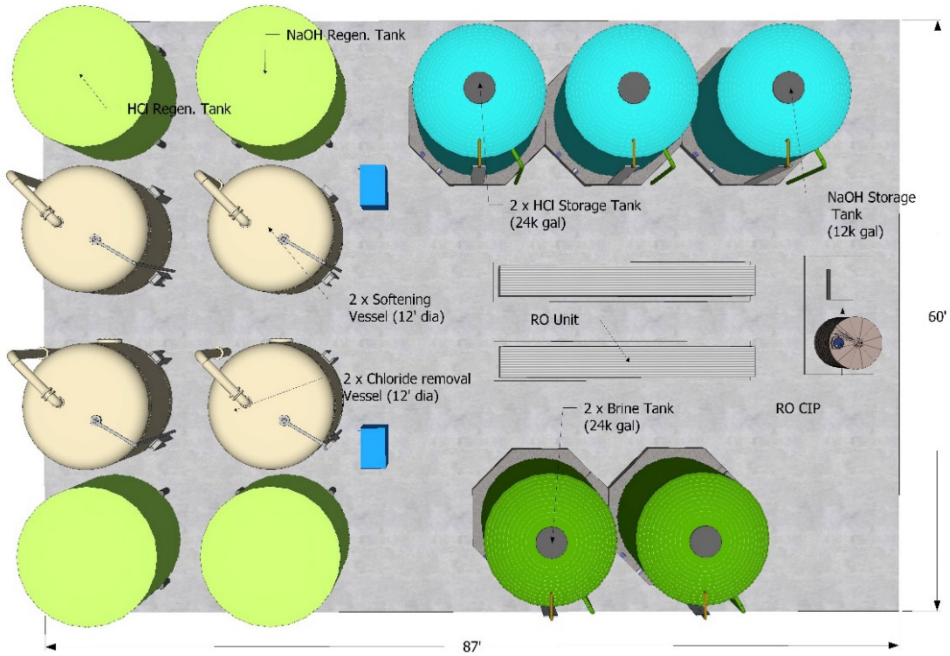

Figure 5.19 Approximate Footprint Requirements for IX System

IX Preliminary Cost Estimate

The IX preliminary cost estimate will be provided as part of the final submittal of this TM.

5.5.2.2 Alternative B - Electrodialysis Reversal

EDR is an electrochemical separation process that uses a direct current (DC) voltage and IX membranes to desalinate water. In an EDR process, alternating cationic and anionic membrane pairs create product and concentrate compartments within a membrane stack. A schematic illustrating the principles of the EDR process is provided on Figure 5.20. In the schematic, the membranes are shown immersed in a sodium chloride (NaCl) solution. When a DC voltage is applied, the ions begin to migrate. One compartment becomes depleted of ions as both positive and negative ions can pass through the cation and anion selective membranes, respectively, yielding desalinated product water. The adjacent compartment becomes more concentrated due to the influx of ions from both sides; the ion-selective membranes trap the ions in this compartment, generating the concentrate. In an EDR stack, the cell pairs are repeated many times to achieve the desired salt removal and product-water recovery. Most of the combined concentrate flow is recycled to continuously collect more ions. In order to control the TDS levels in the concentrate loop and avoid scale formation within the membrane stack, a portion of the flow is wasted as concentrate blowdown, and this volume is replaced with EDR feed water. EDR is not specific to chloride ions and will result in depletion of most of the ions in the feed stream. Thus, an EDR system could be set up to reduce the chloride concentration in a portion of the total effluent (75 to 80 percent), which could then be blended with the bypass stream to produce the desired chloride concentration in the combined effluent, as shown earlier on Figure 5.17.

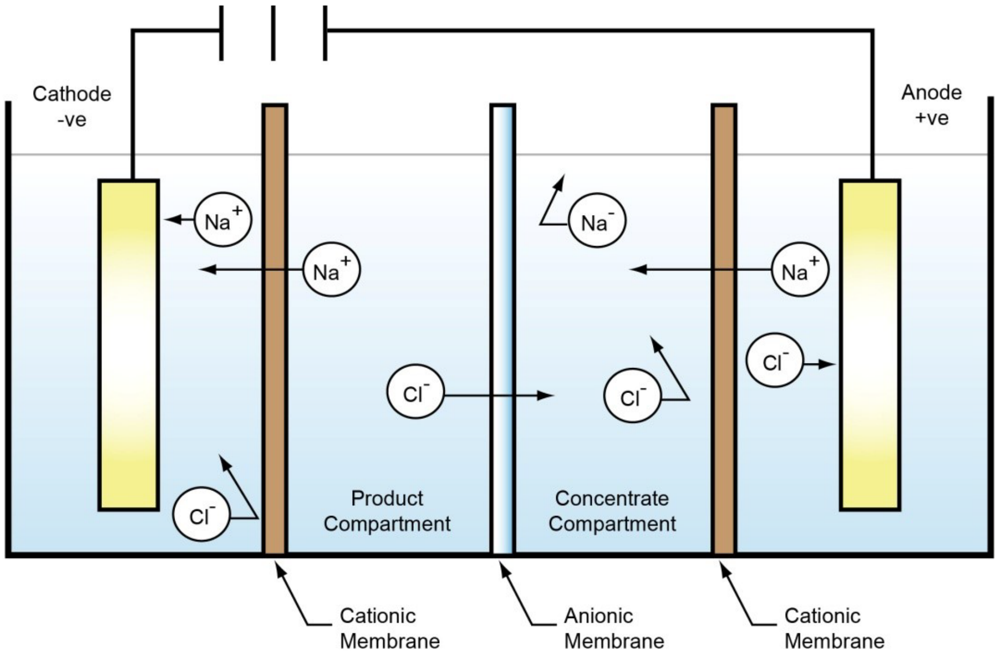

Figure 5.20 EDR Process Schematic

Generally speaking, EDR is less efficient at removal of chloride than RO (discussed in next section), and the costs are typically higher. For these reasons, EDR will not be considered further for this application.

5.5.2.3 Alternative 3 - Reverse Osmosis

RO is a pressure-driven membrane process in which water is forced across a semi-permeable membrane leaving the majority of dissolved ions on the feed side of the membrane. A schematic of the RO process is shown on Figure 5.21. Similar to the EDR process, RO is not selective for chloride and will remove the majority of ions from solution. Thus, side stream treatment, as illustrated on Figure 5.10, would also be appropriate in this case.

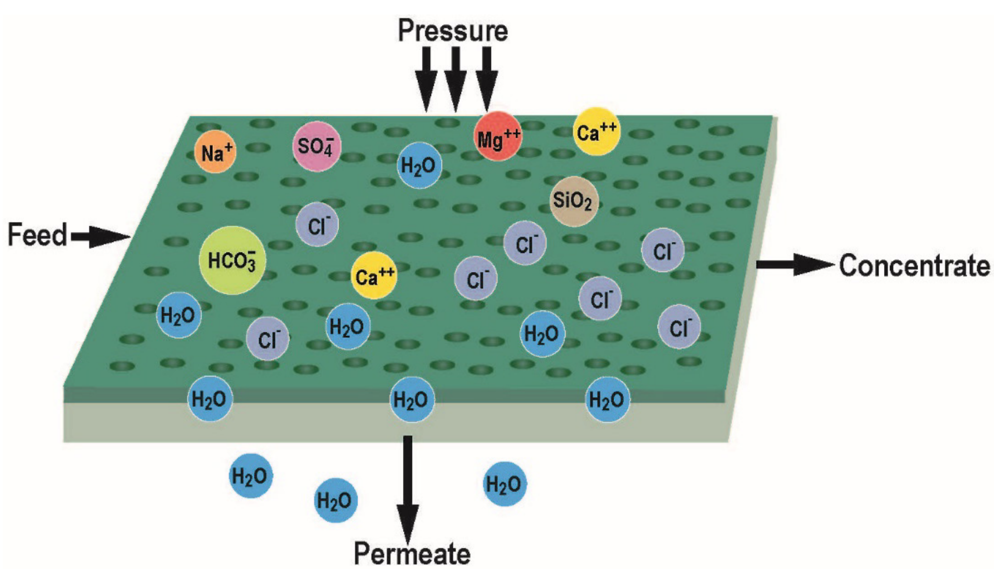

Figure 5.21 Schematic Representation of RO Process at Membrane Interface

RO Process Description

To achieve a blended effluent chloride concentration of <50 mg/L, approximately 1.5 mgd or 77 percent of the effluent flow would need to be treated through a microfiltration (MF) or ultrafiltration (UF) and RO combination. MF or UF would be needed upstream of the RO to provide necessary pretreatment of tertiary effluent. Preliminary design criteria for a side-stream MF and RO system are shown in Table 5.13. To provide redundancy so that the RO system can be continuously in operation, three RO trains would be provided. During normal operation two trains would operate and one would be in standby. When one train requires cleaning, the standby unit would be brought on line. For the purposes of this Facilities Plan, it was assumed that the RO process would be able to achieve a recovery of 85 percent. The concentration of silica in the plant effluent would need to be checked in order to confirm the estimated recovery.

Table 5.13 Side-Stream RO System Preliminary Design Criteria

Description	Preliminary Design Criteria
Number of MF Trains	3 (2 duty, 1 standby)
Feed Flow	1.5 mgd
Recovery	92%
Backwash Water	Recycle to Headworks
Number of RO Trains	3 (2 duty, 1 standby)
RO Pressure Vessels per Train	21
Feed Pressure	100 psi
Membrane Type	ESPA II
RO System Recovery	85%
Total Feed Flow	972 gpm
Concentrate Flow	146 gpm
Blended Chloride Concentration	<50 mg/L

As shown in Table 5.13, the volume of concentrate requiring disposal would be around 150 gpm, or 216,000 gal/day. This is more than could be trucked from the site. A 2012 study by MWH (now Stantec) investigating the use of RO to achieve very low nutrient concentrations, proposed installing a brine pump station and an 8-inch high-density polyethylene (HDPE) pipeline to convey 0.3 to 0.45 mgd of RO concentrate to a new 16-inch diameter ocean outfall in Ventura. The brine pipeline would be about 5.3-miles long and, together with the pump station, was estimated to cost $5.8 million. The 1 mile of ocean outfall was estimated to cost an additional $12.5 million.

In order to avoid the costs and environmental issues associated with construction of a brine line to the ocean, this evaluation has considered concentration of the RO brine followed by trucking.

RO Brine Concentration With EDR Followed by Trucking

There are various technologies that could be considered for concentrating the RO brine. However, most of them result in production of a solid byproduct that must also be disposed of, which adds complexity to the operation. EDR was discussed earlier and is considered here for concentrating RO brine as it can achieve concentration without producing a solid byproduct. The reversible nature of the process allows it to concentrate hardness without scaling the EDR membranes, and silica (which is usually the limiting constituent with RO recovery) is unaffected by EDR because it is not charged. Treating RO brine with EDR would allow the brine flow rate to be reduced, but TDS concentration would increase. Suez, one EDR technology provider, has estimated that, based on the assumed constituents in RO brine, the EDR system could achieve a recovery of 73 percent. This increases system recovery from 85 percent (with RO alone) to about 96 percent (with RO and EDR) reducing the brine flow rate from 216,000 gal/day to about 56,000 gal/day. Figure 5.22 shows the process flow diagram for this alternative. Product from the EDR system would be expected to have a chloride concentration greater than the 50 mg/L limit. If this stream was blended with RO permeate and the bypass stream, the resulting blended stream would have a chloride level above the 50 mg/L target. To rectify this, flow to the RO would need to be increased slightly from 1.4 to 1.5 mgd. This would increase all other flows slightly, but the resulting volume of concentrate for disposal would be about 55,000 gal/day. Assuming that trucks with two liquid tanks (6,000 gallons each) would be used to dispose of the brine, the number of truck loads would be about five per day, which is manageable.

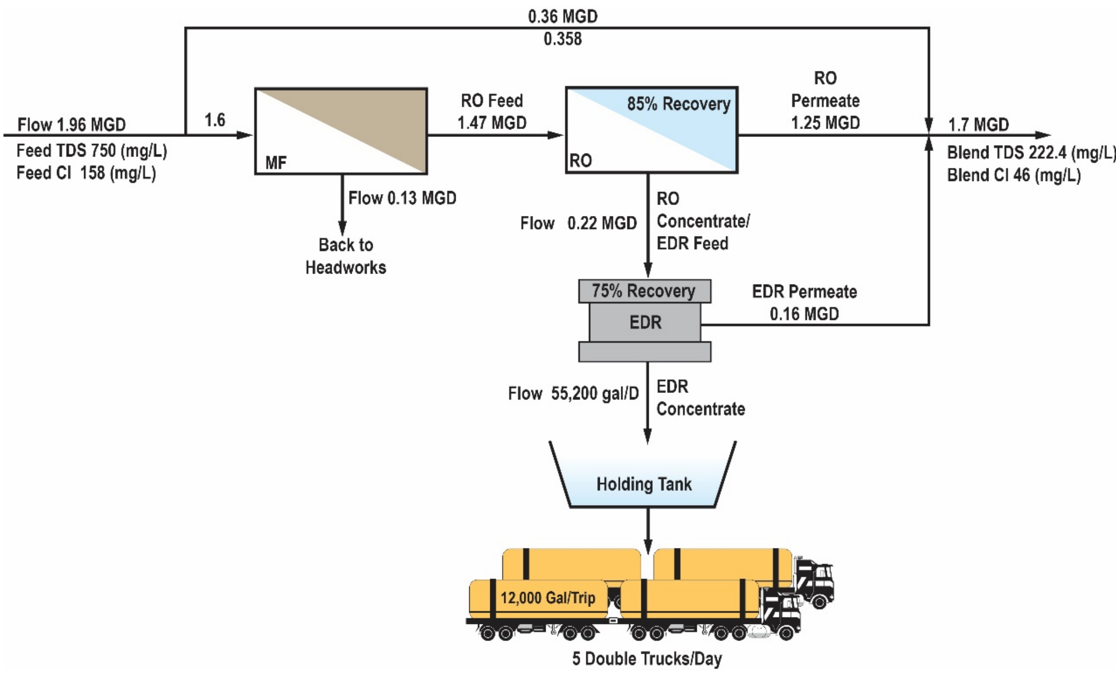

Figure 5.22 RO Brine Reduction and Disposal Using EDR and Trucking

RO/EDR System Layout

A preliminary footprint for the MF/RO/EDR system was developed. A total area of approximately 60 foot by 110 foot (6,600 square feet) would be needed to accommodate the four MF trains, the three RO trains, the clean-in-place (CIP) equipment, and the EDR system. A 20,000-gallon break tank would be provided between the MF and RO processes.

RO/EDR Preliminary Cost Estimate

The RO/EDR preliminary cost estimate will be provided as part of the final submittal of this TM.

5.5.3 Implementation of a Recycled Water Program

For the recycled water program, it has been assumed that FAT is to be implemented at the treatment plant around 2035. Based on the current recycled water regulations, FAT would provide flexibility to allow OVSD to use the water for groundwater augmentation via spreading or injection, or surface water augmentation, i.e., addition of the water to a reservoir used as a source of potable water, such as Lake Casitas.

Summary of Recycled Water Regulations

The California Division of Drinking Water (DDW) developed extensive regulations related to recycled water, the most recent of which came into effect on October 1, 2018. These regulations address IPR through either surface spreading and subsurface application (injection), or reservoir augmentation. In this case subsurface (injection) and reservoir augmentation are being considered, since both require the same level of treatment, although water that meets the requirements for injection could be used for surface spreading if such spreading sites are available. General requirements for pathogen control and TN are listed in Table 5.14. In this case, the TN concentration in the plant effluent would already meet the criteria for TN.

Table 5.14 Summary of Pathogen and Nitrogen Requirements for Injection and Reservoir Augmentation

Description	Subsurface Application (Injection)	Surface Water Augmentation (up to 1% by volume)	Surface Water Augmentation (up to 10% by volume)
Pathogenic Microorganisms			
Enteric Virus	12-log Reduction	8-log Reduction	9-log Reduction
Giardia Cyst	10-log Reduction	7-log Reduction	8-log Reduction
Cryptosporidium oocysts	10-log Reduction	8-log Reduction	9-log Reduction
Total Nitrogen	<10 mg/L	<10 mg/L	
1,4 Dioxane	>0.5-log Reduction	>0.5-log Reduction	
Total Organic Carbon	<0.25 mg/L	<0.25 mg/L	

Control of other parameters, such as regulated contaminants, is presented in the regulations and is similar for both applications.

Subsurface replenishment (injection) and reservoir augmentation requires the use of FAT. FAT includes RO and an advanced oxidation process (AOP). If these technologies are applied, then it is possible that no diluent water will be required for surface spreading. Diluent water is a source other than of wastewater origin that is required in the case of less highly treated effluent to reduce the ratio of total organic carbon (TOC) of wastewater origin. Thus, FAT treated water can be either surface spread or injected directly into the ground, or supplied to a surface reservoir. Additionally, a minimum of two months of subsurface travel time is required before extraction for potable use. These two months provide Response Retention Time (RRT), which is time to monitor water quality and respond to water quality concerns.

The best-known example of a FAT system is the Orange County Water District's Groundwater Replenishment System (GWRS) project. The GWRS project is operated with both surface spreading basins as well as subsurface injection wells.

Other aspects that are required as part of such a potable reuse recycled water program include a Wastewater Source Control Program to monitor and administer industrial pretreatment and source control.

For this evaluation, both a RO-based and non-RO-based treatment configuration have been considered. A non-RO solution is being considered since it would not produce an RO brine that would need to be disposed of, which would be a major cost saving.

Achieving FAT with a non-RO treatment configuration does not meet the current regulations, however, the regulations do leave it open to the project sponsor to use an alternative approach to demonstrate equivalency of water quality.

Non-RO-Based Treatment Configuration

Considering a non-RO based treatment configuration only makes sense if the TDS in the wastewater effluent is low enough to meet groundwater or surface water replenishment limits. In the case of OVSD, the current average TDS is 820 mg/L, and this could be expected to be higher by the time a recycled water project is undertaken around 2035. The groundwater TDS objectives in the vicinity of the plant vary from 800 mg/L in the Upper Ventura Basin, to 1,000 mg/L in the San Antonio Creek Area, up to 1,500 mg/L in the Lower Ventura Basin – see Table 5.15. Therefore, based on current TDS values, it seems possible that non-RO-based treated recycled water could be recharged into the San Antonio Creek Area and Lower Ventura Basins. However, in addition to TDS, these basins also carry sulfate, chloride, and boron objectives. The sulfate concentration in the effluent is currently lower than the objectives in all three basins, but chloride is higher in all three, and boron is already a little above the objective value for the Upper Ventura Basin. This indicates that at least partial desalting would be needed to achieve the chloride objectives.

Table 5.15 Groundwater Basin Salinity Objectives in Vicinity of OVSD Treatment Plant

Basin	TDS	Sulfate	Chloride	Boron
Upper Ventura	800	300	100	0.5
San Antonio Creek Area	1,000	300	100	1.0
Lower Ventura	1,500	300	30	1.5
Current Average WWTP Effluent	820	228	153	0.51
Lake Casitas Treated Water Quality	390	166	24	0.2

The other thing to consider is the salinity in a potential surface water reservoir. As mentioned, the closest reservoir is Lake Casitas. Table 5.15 also shows the treated water quality from Lake Casitas taken from a recent Water Quality Report. As shown, all parameters are well below the current average treated effluent values. This indicates that a salinity removal step would be required as part of the treatment train.

Calculations show that, to achieve a quality that would match that of Lake Casitas and to avoid degradation of the lake quality, RO treatment of the entire effluent stream would be needed. Based on this finding, no further evaluation of a non-RO-based treatment configuration was performed.

RO-Based Treatment Configuration

FAT is defined in the regulations as treatment of an oxidized wastewater using RO and an oxidation treatment process that provides no less than 0.5 log (69 percent) reduction of 1,4-dioxane. The RO permeate must have no more than 5 percent of weekly samples with a TOC concentration greater than 0.25 mg/L.

Treatment configuration for FAT is shown on Figure 5.16. This is very similar to the side-stream treatment configuration discussed earlier, except now all flow is passing through the MF and RO combination, and an ultraviolet (UV)/AOP has been added to the RO permeate for oxidation of Chemicals of Emerging Concern (CECs), including 1,4-dioxane. To achieve UV/AOP conditions and high concentrations of hydroxyl radicals, an external chemical such as hydrogen peroxide or sodium hypochlorite is added just upstream of UV.

Because RO is required, there will also be RO concentrate that has to be treated. In this case, the volume will be greater than for side-stream treatment approach for chloride reduction. Also, it is likely that, because in the EDR process the product water does not pass through a membrane, the ions migrate, and the EDR product water will not be suitable for blending with the RO permeate. Thus, as shown on Figure 5.23, the EDR product water will be discharged directly to the River outfall. The EDR product is expected to have a TDS concentration of around 670 mg/L, and the chloride concentration would be less than 100 mg/L. These values are lower than the current effluent concentrations and would be well below the TMDL limits for these salts.

Table 5.16 Full Advanced Treatment Preliminary Design Criteria

Description	Preliminary Design Criteria
MF/UF	
Number of MF Trains	4 (3 duty, 1 standby)
Feed Flow	20 mgd
Recovery	92%
Backwash Water	Recycle to Headworks
RO	
Number of RO Trains	3 (2 duty, 1 standby)
Feed Pressure	100 psi
Membrane Type	ESPA II
RO System Recovery	85%
Total Feed Flow	1.83 mgd
Concentrate Flow	0.27 mgd
UV/AOP System	
AOP Chemical	Sodium Hypochlorite
UV Dose, mJ/cm^2	800 - 1000
Flow Rate	1.56 mgd
RO Brine Concentration (EDR)	
Number of EDR Trains	2
Flow per Train	190 gpm
EDR Recovery	73%
River Discharge Flow	0.21 mgd
Final Concentrate Flow	74,000 gal/d
Number of Trucks (at 12,000 gal/truck)	6 to 7 per day

Note:
Abbreviation: mJ/cm^2=millijoule per square centimeter

Preliminary design criteria for the FAT system are shown in Table 5.16. As shown, the capacity of the FAT system would only be about 0.4 mgd greater than the side-stream treatment system discussed earlier for chloride removal.

A schematic of the FAT system together with RO brine concentration using EDR is shown on Figure 5.23. The footprint of the MF, RO, and EDR systems would be expected to be similar to those discussed earlier. The UV/AOP system would add an additional 2,400 square feet, which would include some chemical storage area. The combined area for the FAT facilities is expected to be around 60 feet by 150 feet (9,000 square feet).

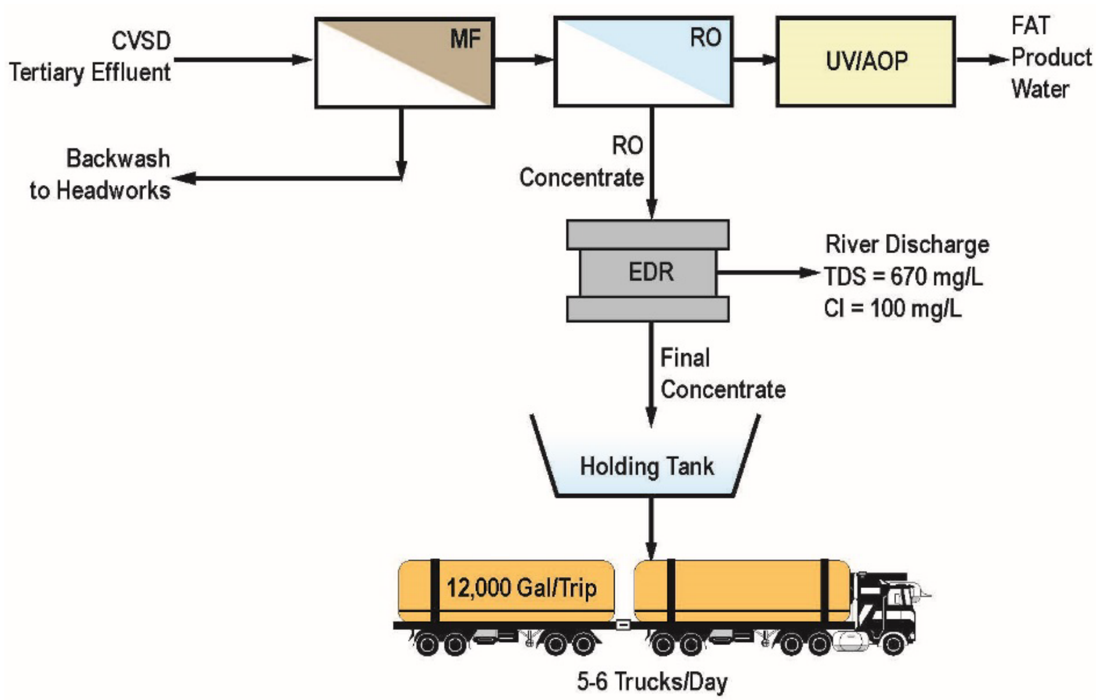

Figure 5.23 Schematic of the Full Advanced Treatment Configuration Plus RO Brine Concentration

5.6 Consideration of Future Site Layouts

The alternative selected to meet the 2025 TMDL limit will have an impact on the availability of space and potential location of future facilities that might be needed for the long term. Considering the long-term possibilities, it was assumed for the purposes of this discussion that the site layout would need to accommodate facilities to either reduce the effluent chloride concentration to below 50 mg/L, or provide full advanced-treated effluent for recycled water use. The footprint needs for reducing chloride with either IX or MF/RO/EDR are similar. It was also assumed that only one technology would be implemented, i.e., either chloride reduction or recycled water using FAT, not both. Since the FAT system plus RO brine concentration using EDR requires the largest footprint, it was decided that for planning purposes the footprint associated with this system would be used.

In the site layouts that follow, each TMDL alternative (Alternatives 1 through 4) is shown with its associated site modifications in 2025 in red, and future requirements to accommodate FAT and RO brine concentration with EDR are shown in blue.

Figure 5.24 shows the future potential layout using Alternative 1 as the approach to achieve the TMDL in 2025. As shown, both ditches would remain in operation for this alternative. Future FAT facilities would be placed where the northerly digester and mechanical facilities currently are. The UV/AOP system in this case would be separate due to space limitations, located where the existing buildings are to the west of the open digester. The open digester would be used to store concentrated RO brine prior to trucking.

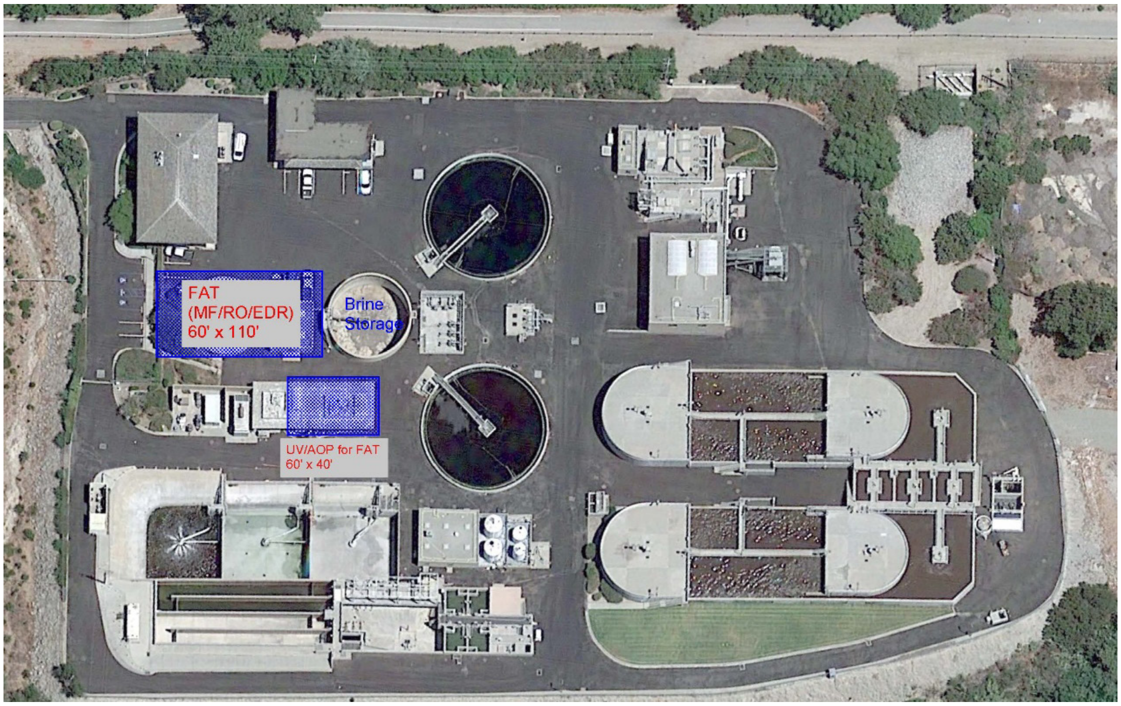

Figure 5.24 Future Site Configuration Using Alternative 1 as Approach to Achieve TMDL

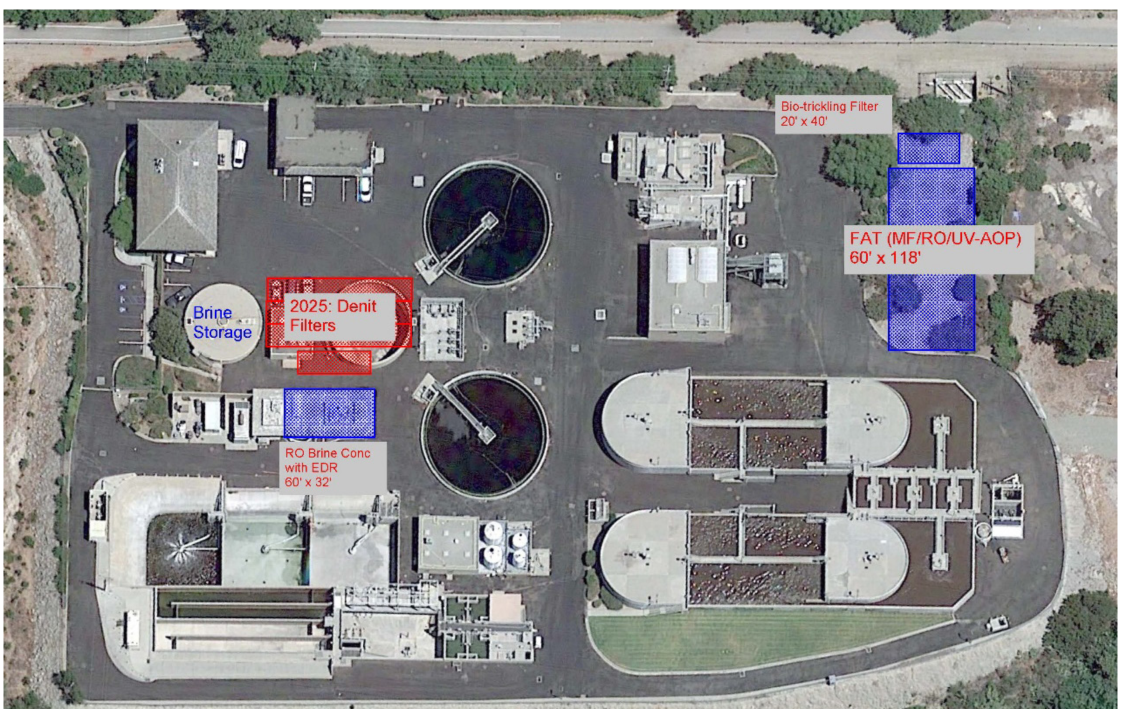

Figure 5.25 Future Site Configuration Using Alternative 2 as Approach to Achieve TMDL

Figure 5.25 shows the future potential layout using Alternative 2 as the approach to achieve the TMDL in 2025. In this case, both ditches would remain in service, and the new denitrification filters would occupy the area to the north of the secondary clarifiers. As shown, the future FAT facilities would be placed in the area where the existing odor control biofilter is located. A new higher-rate biotrickling filter would be constructed in its place. There is not sufficient space in this part of the plant to accommodate the EDR system for RO brine concentration. So, these facilities would be located where the buildings to the west of the digesters are. The final waste stream would be stored in the northerly digester prior to truck pickup.

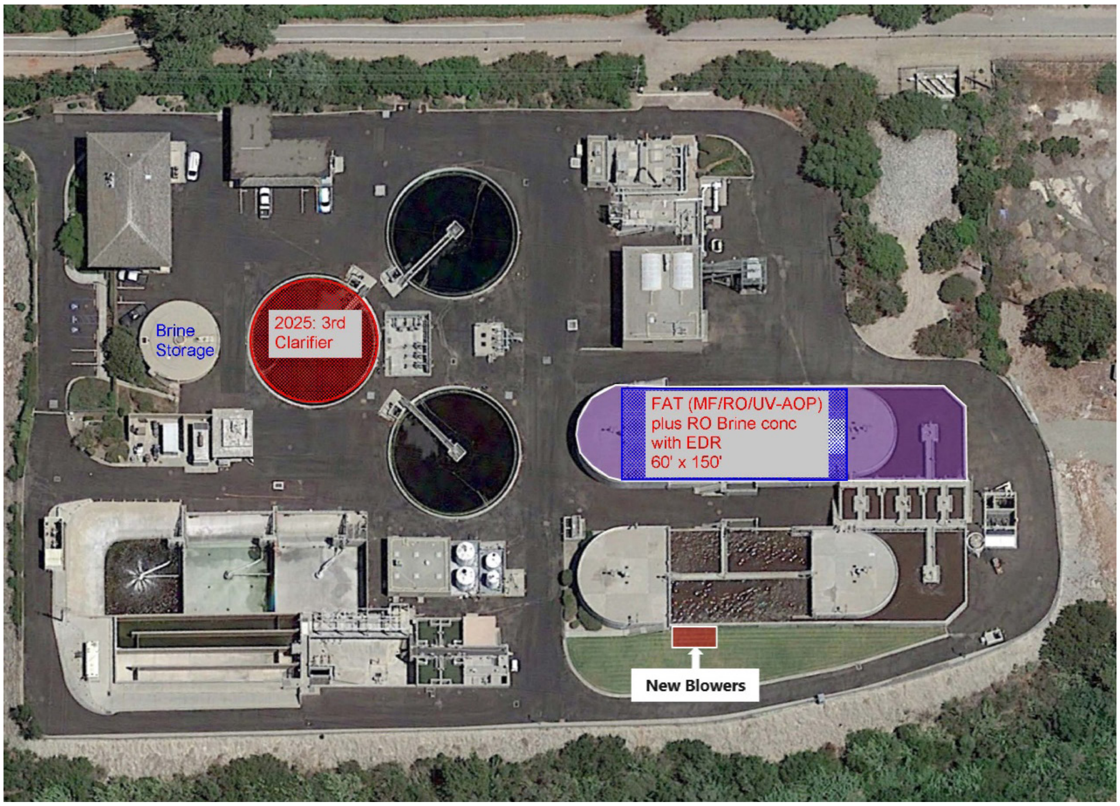

Figure 5.26 Future Site Configuration Using Alternative 3 as Approach to Achieve TMDL

Figure 5.26 shows the future potential layout using Alternative 3 as the approach to achieve the TMDL in 2025. In this case, only one ditch would remain in service, and a third secondary clarifier would be constructed, as shown. The future FAT facilities would be placed in the area once occupied by OD 2, where there would be adequate space. The final concentrated waste stream would be stored in the northern digester prior to truck pickup.

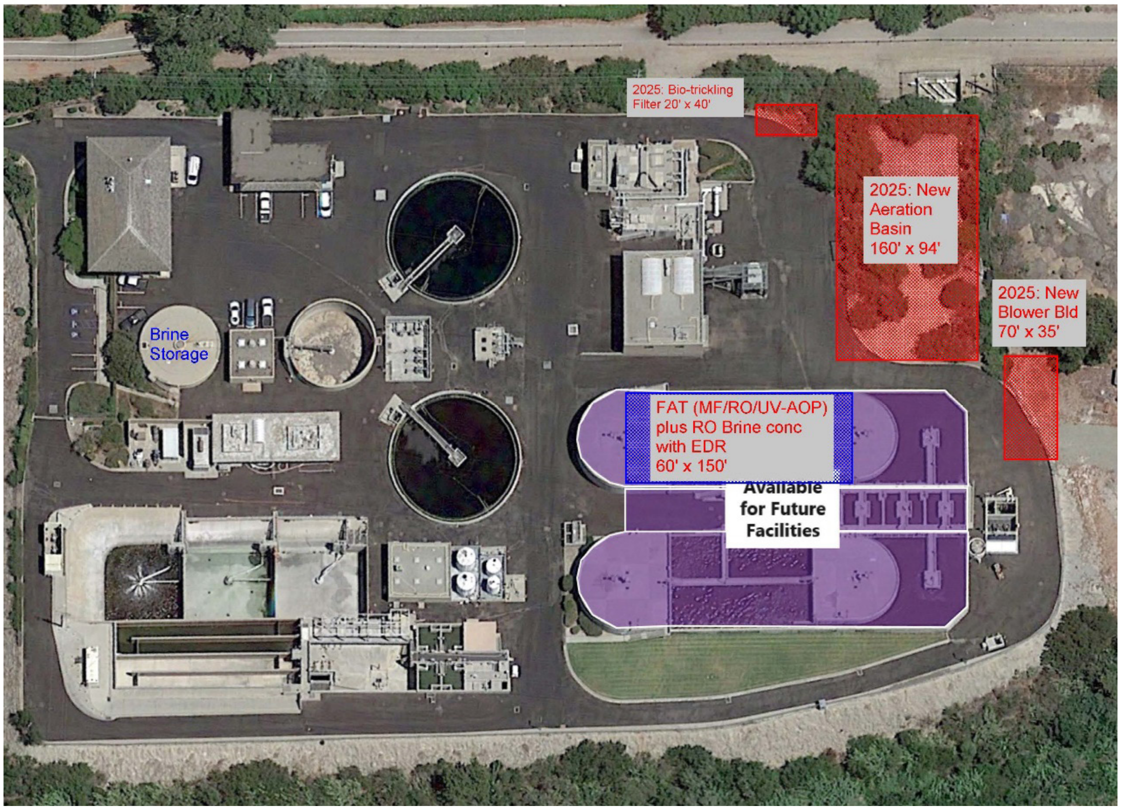

Figure 5.27 Future Site Configuration Using Alternative 4 as Approach to Achieve TMDL

Figure 5.27 shows the future potential layout using Alternative 4 as the approach to achieve the TMDL in 2025. In this case, neither ditch would remain in service. The new aeration basin would be constructed in the southern area currently occupied by the biofilter. A new higher rate biotrickling filter would be constructed as close to the headworks area as possible, and a new blower building would be constructed close to the new aeration basin. It is currently shown partially off the site in the flood plain area, which would need to be built up to accommodate the structure. There would be a significant amount of space on the site for the future FAT facilities once both ditches are demolished. The figure shows the facilities over OD 2, but these could be moved to suit a more open site configuration. Similar to the other alternatives, the final concentrated waste stream would be stored in the northern digester prior to truck pickup.

5.7 Comparison of Alternatives Considering Long-Term Objectives

Earlier the four alternatives were compared with respect to achieving the 2025 TMDL requirements, and Alternatives 1 and 4 faired best in terms of the 20-year life-cycle cost estimate, but Alternative 1 ranked lowest when considering some of the non-economic factors; whereas Alternative 4 was also highly ranked when considering non-economic factors.

Evaluating the impacts of implementing a recycled water system that incorporates FAT and concentration of RO brine using EDR, shows that the site could accommodate the new facilities, irrespective of which alternative was selected. However, in some cases, the site would be more cramped and congested than in others, and the flow would be more complex. Alternative 4 provides the best site arrangement because it frees up considerable site area. Alternative 2 has the least desirable layout in terms of flow and congestion.

5.8 Summary

The evaluation of alternatives to enhance the existing treatment plant performance to meet the more stringent effluent discharge requirements for TN and TP has been completed. Five possible alternatives that can meet the treatment objectives were identified, developed, and evaluated in terms of both economic and non-economic factors. Four of these alternatives include use of one or both of the existing oxidation ditches. Due to concerns with the condition of the concrete in the oxidation ditches and whether the structures will last another 20 years, an additional alternative was developed which would construct a new separate aeration basin designed to meet current and future operating conditions and the effluent limits required in the TMDL.

In terms of cost, Alternative 1 (combining both existing ditches into a single 5-stage Bardenpho system) had the lowest estimated 20-year life-cycle costs of $14.4 million, and an estimated construction cost of $8.4 million. However, this alternative lacked redundancy, making it impractical as a long-term solution. Accordingly, a hybrid of Alternatives 1 and 2, called Alternative 1A, was developed, to include the benefits of Alternative 1 and the redundancy features of Alternative 2.

Alternative 1A includes using both ditches in series during the winter months in a 5-stage Bardenpho configuration, with the flexibility to take one ditch out of service in the summer months. When one ditch is out of service, the required TMDL limits would be achieved via polishing in denitrification filters with a capacity of 2 mgd. This would provide a high level of flexibility to plant operations.

Because of its ability to makes use of all existing facilities and incorporate good process redundancy and operational flexibility, Alternative 1A, with an initial preliminary estimated construction cost of $16.1 million, was selected as the preferred alternative.

Looking to the long term, around 2035, each of the four alternatives was used as a starting point to evaluate how the site might look if a recycled water project is undertaken. The recycled water project was considered because that project would require facilities that would occupy most space on the site. The evaluation showed that all five alternatives provide enough space on the site to accommodate facilities that would be needed to implement a recycled water project that would include FAT of the effluent plus concentration of the RO brine to allow it to be trucked from the site for disposal. However, some alternatives result in a slightly more operator-friendly layout than others. Alternative 2 was particularly constrained in terms of available space because a lot of it would be taken up by the new denitrification filters as part of the 2025 TMDL project. On the other hand, Alternative 4 results in the most open site configuration, creating more space on the site because both ditches would be decommissioned and could, therefore, be demolished, creating additional area.

Overall, considering cost, the condition of the concrete in the existing ditches, other non-economic factors, and the future site facilities, Alternative 1A is considered to be the preferred alternative.

Appendix 5A

COST ESTIMATES

(See facilities plan for detailed cost estimates)

Preliminary Cost Estimate for Short-Term TMDL Project for Ojai Valley Sanitary District

Cost Breakdown	Alternative 1 5-Stage Bardenpho in Combined Ditch Configuration	Alternative 1A 5-Stage Bardenpho + ADWF Denitification Filters	Alternative 2 3-Stage Bardenpho Ditches + Denitrification Filters	Alternative 3 5-Stage Bardenpho in Aeration Basin Configuration	Alternative 4 New 5-Stage Bardenpho TanksProcess
Modifications to achieve TMDL Limits	$500,000	$4,410,000	$5,000,000	$6,000,000	$7,500,000
Mechancial Equipment Replacement	$1,500,000	$1,500,000	$1,500,000		
Structural Upgrades to address ASR Issues (Concrete Rehab and Replacement)	$1,000,000	$1,000,000	$1,000,000	$1,000,000	-
Structural Upgrades to address ASR Issues (Concrete Lining)	$3,400,000	$3,400,000	$3,400,000	$1,800,000	-
Third Secondary Clarifier	-	-	-	$3,000,000	-
Additional De-Nit Filters	-	-	$100,000	-	-
Relocation of Odor Control Biofilter	-	-	-	-	$500,000
Sub-total 1	**$6,400,000**	**$10,310,000**	**$11,000,000**	**$11,800,000**	**$8,000,000**
Site Work Allowance (5%)	$320,000	$515,500	$550,000	$590,000	$400,000
Electrical Allowance	-	$2,062,000	$2,200,000	$1,770,000	$2,000,000
Sub-total 2	**$6,720,000**	**$12,887,500**	**$13,750,000**	**$14,160,000**	**$10,400,000**
Contingency (25%)	$1,680,000	$3,220,000	$3,440,000	$3,540,000	$2,600,000
Preliminary Construction Cost Estimate	**$8,400,000**	**$16,107,500**	**$17,190,000**	**$17,700,000**	**$13,000,000**
Allowance for Engineering, Legal, Admin etc (35%)	$2,940,000	$5,638,000	$6,017,000	$6,195,000	$4,550,000

Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 6
CONDITION ASSESSMENT OF EXISTING FACILITIES

REVISED FINAL | August 2020

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Ojai Valley Sanitary District
Facilities Master Plan

Technical Memorandum 6
CONDITION ASSESSMENT OF EXISTING FACILITIES

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Carollo Project No. 11321A00

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CONDITION ASSESSMENT OF EXISTING FACILITIES

6.1 Introduction

The purpose of this technical memorandum (TM) is to document the operational and physical condition of the various processes at Ojai Valley Sanitary District's (OVSD's or District's) wastewater treatment plant (WWTP). This information will then be used to identify modifications required as part of the project to meet the total maximum daily load (TMDL) requirement in 2025. The preliminary condition assessment of existing facilities is summarized in the following Sections, and was based on the findings and discussion with operations and maintenance (O&M) staff during the Workshop Meeting on March 26, 2019.

6.2 Background

OVSD currently provides service to a population of about 24,000 people, and operates and maintains about 120 miles of sewer mainlines ranging from 6 inch to 21 inch, five sewer lift stations, and one WWTP. The WWTP is a tertiary plant with an average dry-weather design capacity of 3 million gallons per day (mgd) and an instantaneous peak flow capacity of 9 mgd. The current annual average flow is around 1.7 mgd. Untreated wastewater is collected from the City of Ojai; the unincorporated communities of Meiners Oaks, Mira Monte, Oak View, Casitas Springs, and Foster Park; and the North Ventura Avenue area.

6.3 Methodology

The existing as-built drawings were reviewed, and a preliminary list of the various processes and facilities was developed. A process-by-process review of the condition assessment was performed with O&M staff in the Meeting No. 3 Alternatives Evaluation Workshop on March 26, 2019. The findings from the condition assessment are summarized in the following sections for each process area. To assist with the assessment of remaining useful life for concrete structures, estimates were made for typical structure life in various process areas. Some process areas have harsher conditions, and concrete life is lower in those areas. A summary of typical values used in the analysis is shown in Table 6.1.

Table 6.1 Typical Structure Useful Life

Process Area	Estimated Structure Lifetime (Years)	Comments
Headworks	50-70	Harshest environment for concrete
Primary Clarifiers	Approximately 80	Not applicable for OVSD
Aeration Basins	85-90	OVSD has special circumstances due to ASR(1)
Aerobic Digester	Approximately 75	Not in service

Table 6.1 Typical Structure Useful Life (continued)

Process Area	Estimated Structure Lifetime (Years)	Comments
Dewatering	85-90	
Tertiary Filters	85-90	ASR Considerations(1)
Chemical Building	85-90	

Notes:
(1) Certain structures at the WWTP have observed Alkali Silica Reactivity (ASR) which impacts the expected lifetime of the structure.

6.4 Area 1: Headworks, Dewatering, and Odor Control

6.4.1 Headworks

The raw wastewater flow enters the headworks through a 30-inch diameter trunk sewer. The headworks facility includes two in-channel grinders, which grind large solids entering the WWTP. Downstream of the grinders, plant influent is directed to four submersible pumps (2 pumps per wet well) that lift the flow to a vortex grit removal system followed by a rotary drum fine screen. The screened influent is then routed to secondary treatment. Grit and bar screenings are hauled off-site for disposal in a landfill.

6.4.2 Dewatering

Sludge from secondary clarifiers is pumped either to the oxidation ditches (return activated sludge (RAS)), or directly to the belt press for dewatering (waste activated sludge (WAS)). The belt press dewaters WAS typically to about 14 percent solids, which is then composted in the sludge drying beds. The District uses on-site windrow composting during dry weather and hauls sludge to an off-site composting facility during wet weather.

6.4.3 Odor Control

Foul air, captured from the existing headworks facilities, is routed to an existing wood chip biofilter for treatment.

Figure 6.1 shows the area of Headworks, Dewatering, and Odor Control, and Table 6.2 shows the summarized information about installation and estimated life for structural and mechanical equipment. Mechanical equipment is generally expected to have a useful life of about 15 to 20 years, but this can be lower in some harsh process environments.

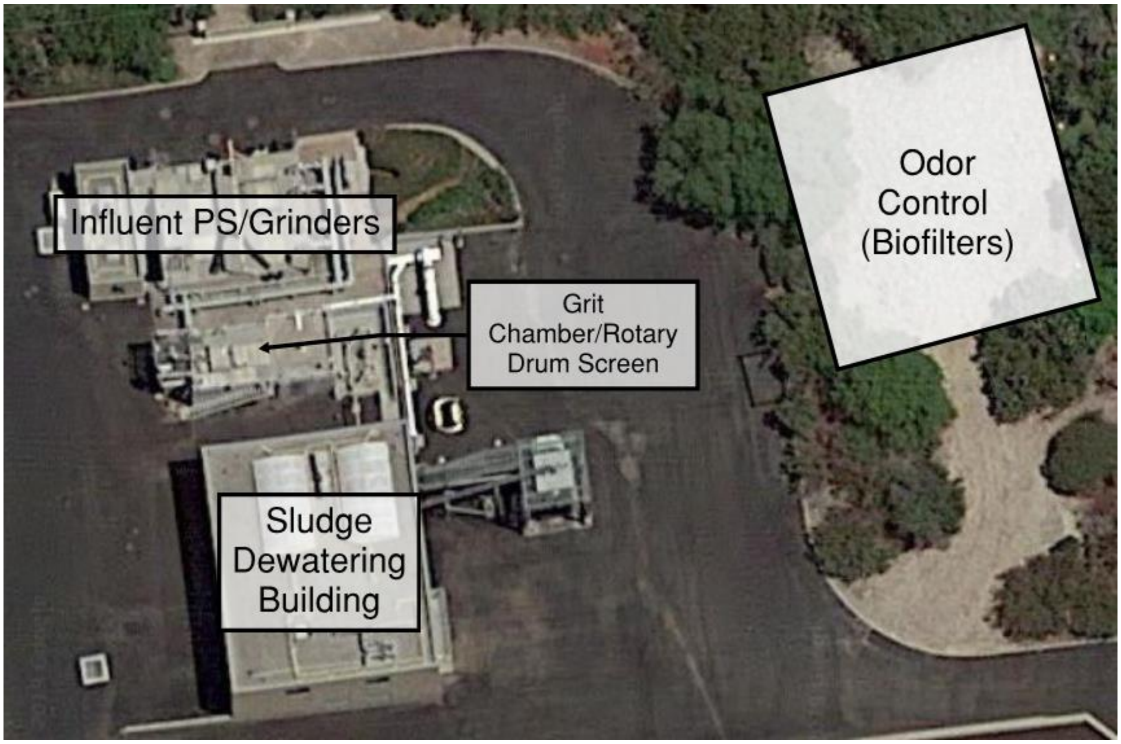

Figure 6.1 Headwork, Grit Chamber, Sludge Dewatering, and Odor Control Facility

Table 6.2 Mechanical and Structural Remaining Lifetime

Process/Facility	Installation	Approx. Age	Structural Remaining Life	Mechanical Remaining Life
Influent Pump Station/Grinders	1997	22	30-48	0-3
Grit Chamber/Rotary Drum Screen	1997	22	30-48	0-3
Sludge Dewatering Building	1997	22	63	0-3
Odor Control	1997	22	48	0-3

In general, the condition of the headworks and grit chamber equipment is poor. The main mechanical equipment is 25 years old, and most equipment is close to the end of its useful life. The majority of the mechanical equipment and instrumentation will require replacement.

The condition of the structural and mechanical assets at the headwork, grit chamber, sludge dewatering, and odor control facility was noted by OVSD staff during the Workshop Meeting and is summarized subsequently:

- Headworks:
  - The rock trap is not operable and gets ragged up. It needs to be removed.
  - The influent channels have very low velocities and result in grit accumulation, Narrow channels will be evaluated as part of replacement with bar screens.
  - The channel grinders do not work well, and require frequent repairs. The grinders should be replaced with bar screens in 3 to 5 years.

- All gates need to be replaced.
- The four submersible pumps are new and work well. Replacement or refurbishment is not needed.

- Grit Chamber:
  - Influent channels have very low velocities and result in grit accumulation. Narrow channels will be evaluated as part of final design.
  - Grit pumps are in good shape.
  - The grit mixer was replaced recently, so the motor and gearbox are new.
  - The concrete slab on the grit basin has a lot of cracks. However, upon observation these seem to be just surface cracks and not structural. Also, the grit basin appears to be lined.
- Sludge Dewatering:
  - The two sludge transfer pumps (rotary lobe pumps) have issues with vibration.
  - The lobes for the pumps are replaced regularly but have limited life remaining.
  - The belt filter press (BFP) is old (22 years), and there is no redundancy. There is room for a second BFP in the building.
- Odor Control:
  - The condition of odor control facilities (biofilter) is acceptable.

6.5 Area 2: Oxidation Ditches, Secondary Clarifiers, and RAS/WAS Pumping

At secondary treatment, the influent flows through three anaerobic tanks in series. After the flow leaves the anaerobic tanks, it enters two identical parallel oxidation ditches that are sectioned into anoxic and aerobic zones. Flow from both oxidation ditches is combined in the mixed liquor splitter box and flows by gravity to two 85-foot diameter clarifiers. A portion of the clarifier underflow is sent to dewatering, and the remainder is routed back to the first anaerobic tank as the RAS flow. Secondary effluent flows to the filter influent pump station, where a portion can be sent via gravity to the equalization basin, and the rest pumped to the tertiary facilities. Figure 6.2 shows the oxidation ditches, secondary clarifiers, and RAS/WAS pumping station, and Table 6.3 shows the summarized information of installation date and estimated life for structural and mechanical equipment.

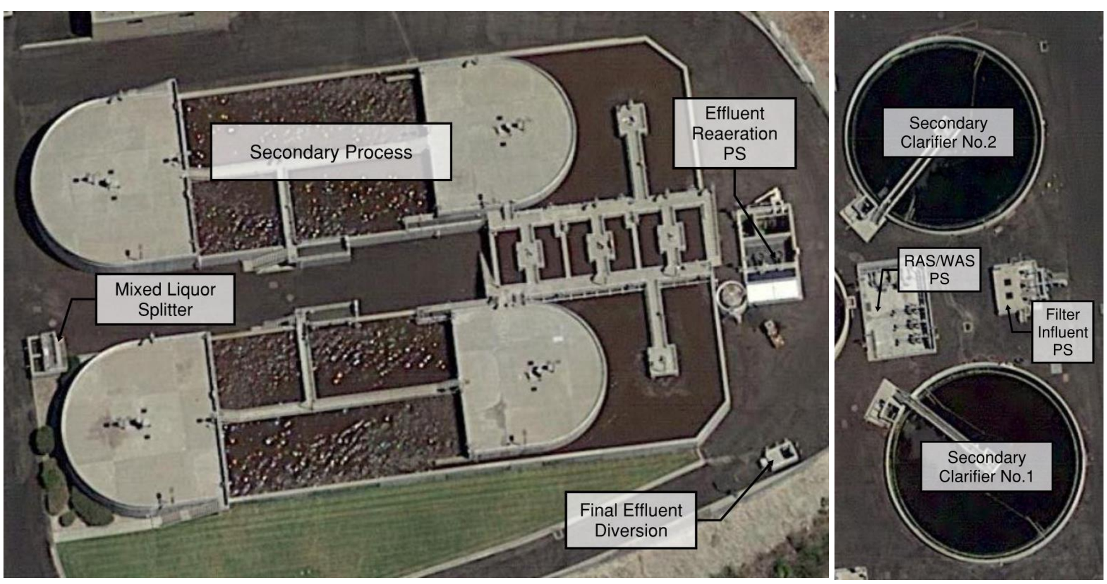

Figure 6.2 Secondary Treatment and Secondary Clarifiers

Table 6.3 Mechanical and Structural Remaining Lifetime

Process/Facility	Installation	Approx. Age	Structural Remaining Life	Mechanical Remaining Life
Secondary Process	1997	22	63-68(1)	0-3(2)
Effluent Reaeration Structure(3)	1997	22	63-68	10-15
Final Effluent Diversion Structure(4)	1997	22	63-68	10-15
Mixed Liquor Splitter	1997	22	63-68	15-20(5)
RAS/WAS Pump Station	1997	22	63-68	0-3
Filter Influent Pump Station	1997	22	63-68	5-10
Secondary Clarifier Nos. 1 and 2	1997	22	63-68	5-10

Notes:
(1) This remaining useful life is impacted by the presence of alkali silica reactivity (ASR) in some structures. See further discussion in the succeeding paragraphs.
(2) This condition refers to the turbine mixers, aerators and motorized weirs. Gates have been replaced over time.
(3) This structure only includes gates.
(4) Gates in this structure were replaced a few years ago and are in good condition.
(5) Original gates were replaced with stainless steel gates.

The condition of each process is summarized in the following sections based on input provided by OVSD Staff during the Workshop Meeting:

- Secondary Process:
  - The condition of secondary process mechanical equipment is poor.
  - All the major mechanical equipment in the anaerobic stage (radial blade turbine mixers), the anoxic stages (radial blade turbine mixers), and the aeration zones (aerators and motorized weirs) are original and are now 22 years old, and in need of replacement.
  - All but two of the oxidation ditch gates have been replaced over time. Once these are replaced the District feels that the condition of the gates will be good.
  - The two gates in the mixed liquor splitter box were replaced with stainless steel gates.
  - The anaerobic radial turbine mixer was rebuilt a few times.
  - The condition of the effluent reaeration structure, final diversion structure, and mixed liquor splitter box is good. Gates in the effluent diversion structure were replaced a few years ago.
- Secondary Clarifiers:
  - In general, the structural components of the secondary clarifiers appeared to be in good condition.
  - No issues with the concrete were observed.
  - The mechanical components are in good condition.

- RAS/WAS Pump Station:
  - The Plant RAS/WAS pump station consists of three non-clog horizontal centrifugal RAS pumps, two rotary lobe WAS pumps, and two submersible sump drainage pumps.
  - The RAS pumps work well. The WAS pumps need to be replaced because they have some vibration issues.
- Filter Influent Pump Station:
  - The filter influent pump station consists of three vertical turbine pumps.
  - The mechanical and structural condition is good.

6.5.1 Oxidation Ditches Structural Condition

The basins were originally constructed as part of the 1997 project, and the concrete now shows visible signs of deterioration. Oxidation Ditch No. 1 (west side ditch) was constructed first. Oxidation Ditch No. 2 was constructed about 18 months later. Ditch No. 1 has the worst condition. The outside of the basin wall for Ditch No. 1 can be seen on Figure 6.3.

Based on a report prepared for Oakridge Geoscience Inc. by the CTL Group, the following observations were made about the structural components of the aeration basins:

- Various concrete in and around the basins is cracking or damaged and requires repair.

Figure 6.3 Oxidation Ditch No. 1 - West Side of the Structure

- A core sample from Oxidation Ditch No. 1 was taken by the CTL Group from the inside wall (Figure 6.4). The presence of alkali silica reactivity (ASR) was detected in this core sample.

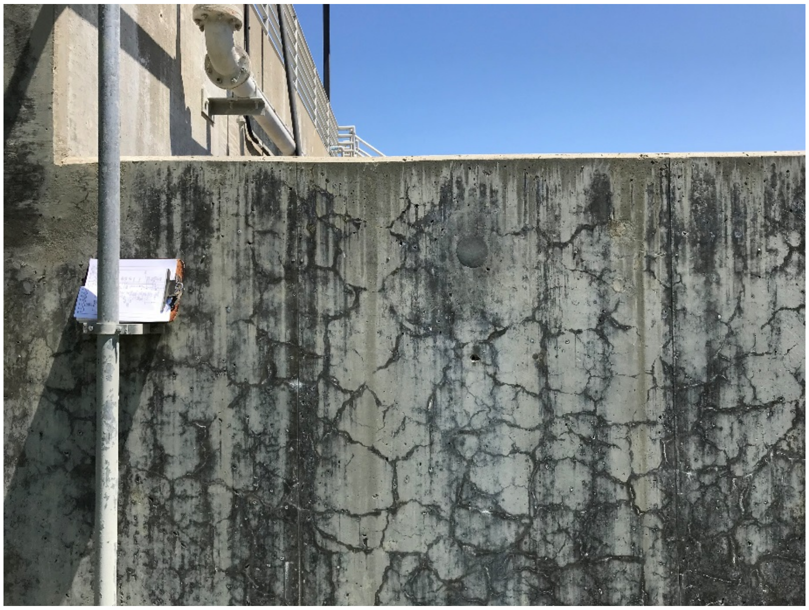

Figure 6.4 OXD-3 Sample Core Location – Ditch No. 1

- The results from tests of the core sample taken by CTL Group are summarized in the following:
  - ASR was observed as darkened reaction rims around some aggregate particles, clear to milky ASR gel deposits in some voids, and locally in some micro-cracks. Micro-cracks passing through or extending out of the reactive aggregate particles were observed.
  - Continuous fluid contact on concrete surfaces can accelerate ASR.
  - The water/cement ratio is estimated at 0.5 to 0.6 versus the design ratio of 0.4. On that basis, it appears additional water was added to the cement.
  - No fly ash or other non-cementitious material was added to the mix to reduce cement content. Current mix design typically includes fly ash to reduce cement content.
  - The chemical test used to evaluate ASR potential in 1994 was based on a positive/negative test procedure. This test method can be inaccurate and is not recommended for current concrete evaluations.
- Various visible cracks on top of the oxidation ditch walls and elsewhere were observed as shown on Figure 6.5.

Figure 6.5 Cracks in the Oxidation Ditch Structure - Ditch No. 1

Based on the core samples and visual observations of the structural condition of the oxidation ditches, the following initial recommendations can be made:

- Perform a comprehensive visual assessment of the affected structure(s). This would include mapping the cracks and other damage in the field. The description of cracking should include crack length, spacing, alignment, and any exudation observed. Additionally, evidence of structure and expansion, such as movement of joints, should be documented. The purpose for doing this survey would be to:
  - Serve as a field basis for diagnosing ASR and grading it as low, medium, or high.
  - Identify other mechanisms that may be causing the deterioration observed.
  - Allow for a general condition assessment of the structure.
  - Identify potential need for additional testing.

- Request the following information from the CTL Group (laboratory that did the petrographic testing):
  - Characterize the laboratory evidence of ASR as being low, medium, or high.
  - Provide a prognosis for the potential future ASR reactivity. This may require further testing of the samples in their possession or may require additional cores.

Based on results from the recommendations as previously stated, the following options can be further evaluated for structural rehabilitation of the oxidation ditches:

1. Do nothing and continue to monitor, including mapping and measurement of cracks, and make repairs or adjust the approach over time. The wall cracks are fairly significant after 22 years and are expected to get worse with time because the ditches are full of water, and therefore, the ASR reactivity will continue. It is not clear whether the structures will last another 20 years. The concern is that, as cracks worsen, the rebar will be exposed to water and result in corrosion.
2. Repair existing cracks, and coat with a moisture-resistant coating, such as an epoxy or a polyurethane liner. This is a fairly significant effort due to the large surface area of concrete that is exposed to water. In addition, there is no guarantee that cracks may not occur in the liner in future, which would let in water and allow the ASR to worsen in those specific areas. Flexible liner/coatings can be considered to account for continued movement.
3. Complete replacement of the affected structures. The District feels that a major structural rehabilitation is needed because the life expectancy is probably not more than 20 years. This might include removal and rebuilding of some or all of the decks that support the aerators. Structural evaluation of the current structure including seismic assessment will have to be completed for partial structural rehabilitation. Another option would be to construct a new aeration basin that supports the process, to meet the TMDL requirements in 2025.

For the purposes of the Facilities Master Plan it will be assumed that if the oxidation ditches are to be reused, then structural rehabilitation will be required. For Ditch No. 1 this would include removal and reconstruction of the two aerator decks, as well as lining of the entire structure with an epoxy or polyurethane liner. For Ditch No. 2, only lining of the ditch will be assumed.

6.6 Area 3: Tertiary Filters and Disinfection

At the tertiary facilities, secondary effluent flows through two flocculation basins in series before exiting through a channel. The channel feeds four deep-bed, continuous backwash sand filters before being routed to an ultraviolet (UV) system for disinfection. The UV system consists of one channel with five banks of UV lamps. As a backup, the flow from the tertiary filters can be routed through a chlorine contact tank downstream of the UV system.

After disinfection, flow is routed through a 28-inch diameter pipe, to a reaeration structure, and then into a 36-inch diameter pipe to the outfall. Figure 6.6 shows the tertiary filters and disinfection area, and Table 6.4 shows the summarized information of installation and estimated life for structural and mechanical equipment.

Due to their physical proximity, the three secondary effluent equalization basins were included in this area. These basins are interconnected with the tertiary effluent feed pump station, downstream of the secondary clarifiers, and can be operated as either one, two or three basins in parallel. Each includes a submersible mixer and blowers.

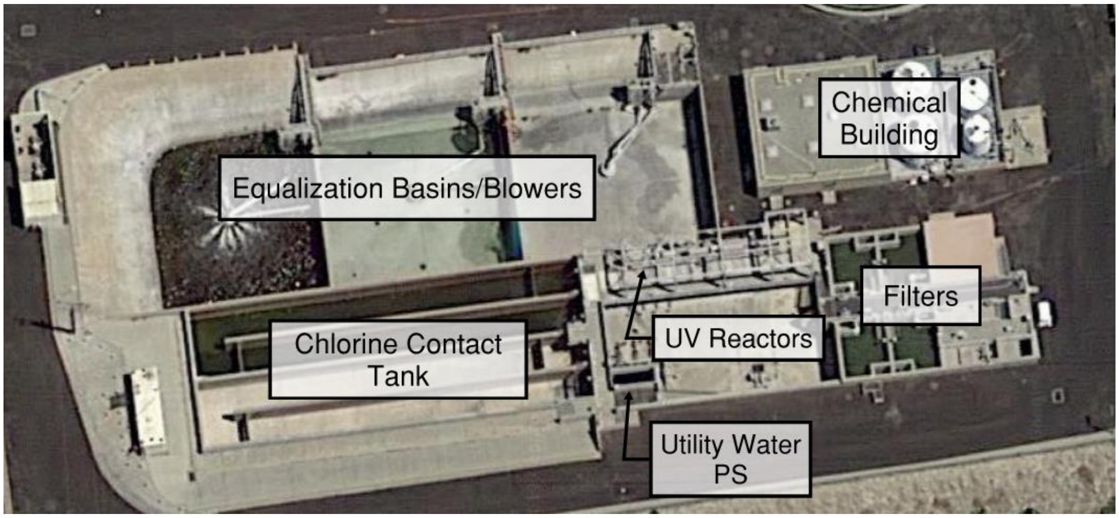

Figure 6.6 Tertiary Filters and Disinfection Treatment Process

Table 6.4 Mechanical and Structural Remaining Lifetime

Process/Facility	Installation	Approx. Age	Structural Remaining Life	Mechanical Remaining Life
Equalization Basins/Blowers	1979	40	45	10 - 15 (1)
Filters	1997	22	68	10 - 15
UV Reactors	1997	22	68	~10 (2)
Chlorine Contact Tank	1997	22	68	10 - 15
Utility Water Pump Station	1997	22	68	N/A(3)
Chemical Building	1997	22	48	N/A(4)

Notes:
(1) Note that the blowers do not need replacement as they are no longer in use. The existing submersible mixers are in good condition.
(2) The UV system (transformers, ballasts and UV racks) were replaced in 2013.
(3) The UW pump station is regularly maintained by the District. This equipment does not need to be included in the CIP.
(4) The chemical dosing pumps are scheduled for replacement soon and do not need to be included in the short term CIP.

The condition of each of the process facilities is summarized in the following sections based on input from OVSD staff provided at the Workshop Meeting:

- Equalization Basins:
  - The Equalization Basins are in good structural condition.
  - The aerated mixing system and blowers are not in operation. The basins are mixed using submersible mixers and are in good condition. The blowers do not need to be replaced as they are not in use.

- Tertiary Filtration:
  - Filters consist of two vertical shaft flocculators and four deep bed sand filtration units and are in good condition.
  - The channels upstream of the filters have very low velocities during low flows and may cause accumulation of solids, but are in good condition. During final design, the channel shall be modified to improve velocities or replaced with new tertiary influent pipe.
- UV Disinfection:
  - The UV racks were installed in 1997 and replaced in 2013. The mechanical and structural condition is good.
  - There are items that need to be addressed that were identified in an August 2018 study conducted by Carollo Engineers, Inc. (Carollo). See Appendix B.3 of the Facilities Plan.
- Chlorine Contact Tank:
  - The basin was constructed in 1997 with rotating skimmer mechanism for scum removal.
  - Sodium hypochlorite is used as a backup disinfectant to the UV system during storm events or normal process interruptions.
  - Prior to discharge, sodium bisulfite is added to the treated effluent to remove residual chlorine.
  - The overall mechanical and structural condition is good.
- Utility Water Pump Station and Chemical Building:
  - The water utility pump station consists of three horizontal end suction pumps, two static aerators, and one strainer. This equipment receives regular maintenance by the plant staff and is in good condition. The strainer was replaced in May 2019.
  - The chemical feed pumps in the chemical building are scheduled for replacement soon and do not need to be included in the short term CIP. Structurally, the condition of the chemical building is good.

6.6.1 Tertiary Filters Structural Condition

For the filters structural condition, core sampling analysis by the CTL Group was completed for two locations. Figures 6.7 through 6.9 show the locations of the core samples.

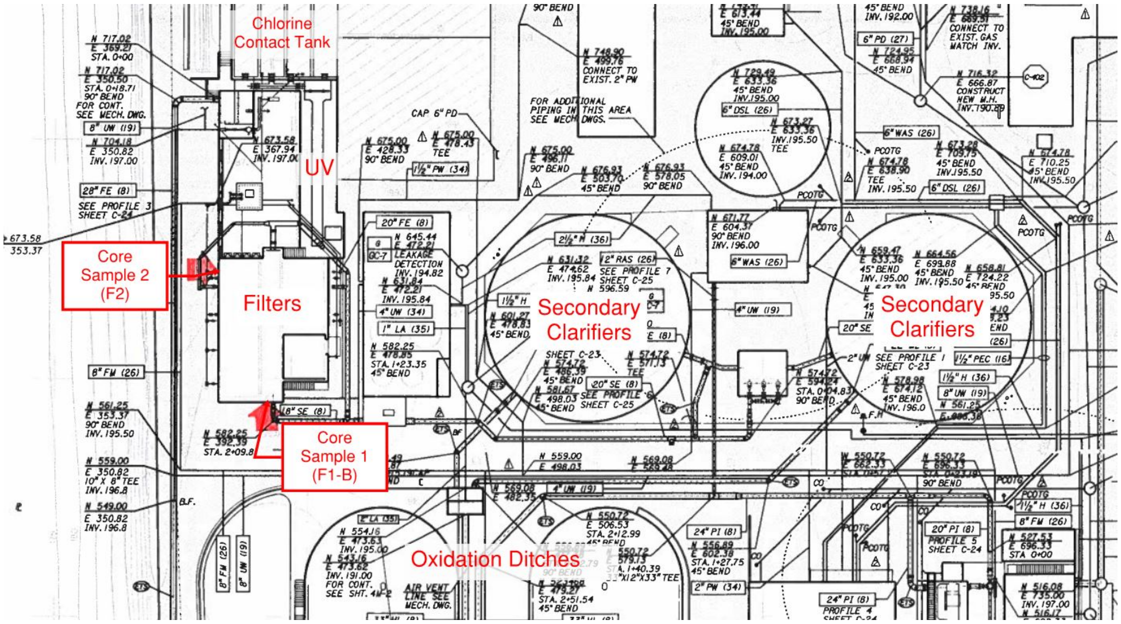

Figure 6.7 Filter Building Core Sample Locations

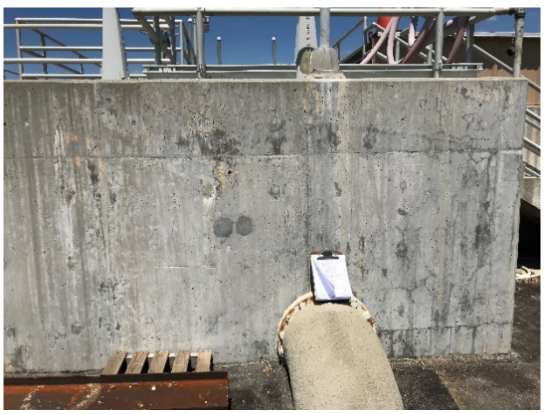

Figure 6.8 F1-B Core Sample Location

Figure 6.9 F2 Core Sample Location

The findings from the analysis of core samples F1-B are summarized as follows, and are the same as those mentioned earlier for the oxidation ditch:

- ASR is present in the core samples tested – Filter Building F1-B (south end of structure).
- Continuous fluid contact on concrete surfaces can accelerate ASR. Cores F1-B were taken from areas with continuous water contact.

The following are the summarized findings for the core sample F2:

- ASR was not observed in Core F2 on the west side of the filter structure.
- Core F2 sample was taken from an "open" section of the structure not exposed to continuous water contact.

Core F2 was taken from an "open" section of the structure, which means the wall from which it was taken is not water-bearing now or in the past. For clarification, Core F2 was reported by the CTL Group as having no significant ASR distress (not a negative finding for ASR). This is not to say that ASR is not present, as the Oakridge Geoscience memo suggests. If the structure from which Core F2 was extracted were to be subjected to high levels of moisture, the deleterious ASR that was observed in the other two cores would likely develop. ASR requires a source of moisture to develop to damaging levels. Hence, the potential for damaging ASR is likely present in the filter structure, but the potential for ASR was not estimated by the CTL Group.

For the purposes of the Facilities Plan it will be assumed that the filters structure will be lined with an epoxy or a polyurethane liner where possible to prevent further concrete deterioration due to ASR.

6.7 Area 4: Aerobic Digester

The old aerobic digester building with two rotary lobe blowers and two digester tanks are all off line and out of service. It is not anticipated that these units will be brought back into service. As such, this area will not be included in the CIP project list.

6.8 Summary

The following presents a summary of the modifications that need to be included in either a short term CIP, perhaps as part of the TMDL project, or a longer term CIP, based on the condition assessment feedback obtained from District staff:

1. Headworks and Influent Pump Station:
   a. Remove the existing Rock Trap.
   b. Replace the existing channel grinders with new bar-screens and a new screenings washer/compactor.
   c. Make provisions to address grit settling during low-flow conditions.
   d. Replace all existing gates.
2. Grit Chamber:
   a. Make provisions to address grit settling in the channels during low-flow conditions.
   b. Replace the existing grit mixer.
   c. Evaluate and rehabilitate structure as needed to address corrosion and cracks.
3. Sludge Dewatering:
   a. Replace existing sludge transfer pumps with new pumps. Consider alternative technology to minimize vibration issues.

   b. Replace the existing BFP with a new dewatering unit. Consider installing a redundant dewatering unit.
4. Oxidation Ditches:
   a. Replace all original mechanical equipment (anaerobic mixers, anoxic mixers, and aerators)
   b. Demolish and replace the aerator decks in Ditch No. 1.
   c. Line both ditches with an epoxy or a polyurethane liner to minimize further concrete deterioration due to ASR. The possibility of lining only one ditch can be determined during the preliminary design phase.
5. RAS/WAS Pump Station:
   a. Replace the existing WAS pumps.
6. Tertiary Filtration:
   a. Make provisions to address solids accumulation during low-flow conditions.
   b. As far as possible, line the filter structure with an epoxy or a polyurethane liner.
7. UV Disinfection:
   a. Incorporate the recommendations from Carollo's August 2018 report and an on-site meeting on August 30, 2019.

A cost estimate for the 2025 TMDL project elements is included in the Facilities Plan.

Appendix B

DETAILED COST ESTIMATES FOR CIP PROJECTS

Engineers...Working Wonders With Water®

OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 01 - Addressing Short Term TMDL - Alt 1A

Original Estimate

PREPARED BY : MFS

LOCATION : Ojai Valley WWTP

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Demolition**					
	Remove Vertical Aerator Reactor	4	LS	$5,000	$20,000	
	Remove Anaerobic Vertical Mixer	3	LS	$5,000	$15,000	
	Demo Exterior Wall of Ditch No.1	1	LS	$195,500	$195,500	
	Dirt Work	1	LS	$27,200	$27,200	
	New Denit-Filter Excavation	1	CY	$7,145	$7,145	
	New Denit-Filter Pipeline Excavation	1	LS	$148,911	$148,911	
	Total					$413,756
2	**Concrete**					
	Base Slab, 12"	34	CY	$250	$10,000.00	
	Base Slab, 24"	29	CY	$237	$8,100.00	
	Walls, 18"	149	CY	$593	$103,900.00	
	Walls, 12"	75	CY	$719	$63,300.00	
	60" Concrete Pipe Encasement	74	LF	$584	$50,800.00	
	28" Concrete Pipe Encasement	18	LF	$299	$6,300.00	
	Ditch NO.1 Concrete Rehab	1	LS	$865,600	$865,600	
	Ditch NO.1 Concrete Lining	1	LS	$1,700,000	$1,700,000	
	Ditch NO.2 Concrete Lining	1	LS	$1,700,000	$1,700,000	
	12" Edge Forms, Flat Mat On Grade, Add	103	LF	$12	$1,200	
	12" Structural Flat Mat On Grade	76	CY	$555	$42,200	
	12" Straight Wall >8' High	169.63	CY	$912	$154,700	
	10" Straight Wall >8' High	49.8	CY	$1,036	$51,600	
	12" Elevated Slab To 20'	8.232	CY	$430	$3,500	
	12" Edge Forms, Slab On Grade, Add	60	LF	$11	$700	
	12" Flat Non-Formed S.O.G.	33.33	CY	$518	$17,300	
	Total					$4,779,200
3	**Metals, Wood, and Plastics**					
	Aluminum Grating with Rebate	1510	SF	$19	$34,000	
	Aluminum Handrail	690	LF	$39	$31,000	
	Aluminum Stairs	27	RSR	$550	$17,000	
	Structural Aluminum	6145	LB	$8	$60,000	
	Al Bracket P680	6	EA	$83	$1,000	
	Redwood Baffles	760	SF	$7	$6,000	
	FRP Weir/ Launder	30	LF	$19	$1,000	
	Aluminium OSHA Handrail	212	LF	$75	$19,000	
	Aluminium Stairs, Including Railing and Supports	60	LF	$561	$40,000	
	Total					$209,000
4	**Modification and New Equipment**					
	1 Ton Base Mounted Davit Crane	1	EA	$15,369	$18,100	
	Anoxic Mixer	3	EA	$32,500	$114,600	
	Surface Aerator	4	EA	$127,700	$600,400	

	SST Slide Gate	2	EA	$10,500	$24,700	
	Denitrificatiion Filters	1	LS	$1,120,000	$1,120,000	
	New Denit-Filter Installation	1	LS	$744,950	$745,000	
	New Denit-Filter Piping Connection	1	LS	$74,495	$74,500	
	3000 gal Storage Tank	2	EA	$10,000	$20,000	
	Dosing Pumps	3	EA	$7,135	$21,400	
	18" Dimj Awwa Butterfly Valve, No Op	2	EA	$10,000	$20,000	
	16" Dimj Awwa Butterfly Valve, No Op	2	EA	$10,000	$20,000	
	Add For Ea 6" Indicator Post Over 6'	2	EA	$138	$300	
	Up To 6' Trench/Bury Indicator Post	2	EA	$2,304	$4,600	
	Magnetic Flow meters	2	EA	$16,000	$32,000	
	Vertical Turbine 20 HP Tertiary Pump	3	EA	$34,269	$102,800	
	Total					$2,918,400
5	**Pipeline**					
	Pipe Penetration Sleeve	3	EA	$280	$1,000	
	18" Cl 52 Cldi Mj Pipe In Open Trench	10	LF	$93	$1,100	
	6" 45° 125# Cldi Fxf Ell	2	EA	$937	$2,200	
	6" Flg Cldi Pipe In Open Trench	20	LF	$52	$1,200	
	18" Flg Cldi Pipe In Open Trench	60	LF	$223	$15,700	
	8" 90° 125# Cldi Fxf Ell	2	EA	$1,046	$2,500	
	8" Flg Cldi Pipe In Open Trench	60	LF	$67	$4,700	
	12" Flg Cldi Pipe In Open Trench	60	LF	$113	$8,000	
	14" New Filter Influent and 16" Filter Effluent and Fitting	1	LS	$96,351	$113,300	
	Total					$149,700
	ITEM NOS. 1-5 SUBTOTAL					**$8,470,056**
6	**Allowances**					
	Electrical and Instrumentation	7	%		$592,904	
	Mechanical	10	%		$847,006	
	Site and Yard Work Allowance	5	%		$423,503	
	Total					$1,863,000
	SUBTOTAL					**$10,333,056**
	Estimating Contingency	30	%			$3,100,000
	SUBTOTAL					**$13,433,056**
	General Conditions	10	%			$1,343,306
	SUBTOTAL					**$14,776,362**
	General Contractor Overhead & Profit	15	%			$2,216,454
	SUBTOTAL					**$15,649,510**
	Escalation	0	%			$0
	SUBTOTAL					**$15,649,510**
	Sales Tax on 50% of Subtotal Above	7.75	%			$521,000
	CONSTRUCTION COST SUBTOTAL					**$16,171,000**
	Engineering, Management, and Legal	35	%			$5,660,000
	PROJECT COST (April 2019 Dollars)					**$21,831,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Jul-20

JOB # : Proj No. 01 - Addressing Short Term TMDL - Alt 1A

Refined Estimate

PREPARED BY : MFS

LOCATION : Ojai Valley WWTP

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Demolition**					
	Remove Vertical Aerator Reactor	4	LS	$5,000	$20,000	
	Remove Anaerobic Vertical Mixer	3	LS	$5,000	$15,000	
	Demo Exterior Wall of Ditch No.1	1	LS	$195,000	$195,000	
	Dirt Work	1	LS	$27,200	$27,200	
	New Denit-Filter Excavation	1	CY	$7,145	$7,145	
	New Denit-Filter Pipeline Excavation	1	LS	$148,911	$148,911	
	Total					$413,256
2	**Concrete**					
	Base Slab, 12"	34	CY	$250	$10,000.00	
	Base Slab, 24"	29	CY	$237	$8,100.00	
	Walls, 18"	149	CY	$593	$103,900.00	
	Walls, 12"	75	CY	$719	$63,300.00	
	60" Concrete Pipe	74	LF	$584	$50,800.00	
	28" Concrete Pipe	18	LF	$299	$6,300.00	
	Ditch NO.1 Concrete Rehab	1	LS	$865,600	$865,600	
	Ditch NO.1 Concrete Lining	1	LS	$0	$0	
	Ditch NO.2 Concrete Lining	1	LS	$0	$0	
	12" Edge Forms, Flat Mat On Grade, Add	103	LF	$12	$1,200	
	12" Structural Flat Mat On Grade	76	CY	$555	$42,200	
	12" Straight Wall >8' High	169.63	CY	$912	$154,700	
	10" Straight Wall >8' High	49.8	CY	$1,036	$51,600	
	12" Elevated Slab To 20'	8.232	CY	$430	$3,500	
	12" Edge Forms, Slab On Grade, Add	60	LF	$11	$700	
	12" Flat Non-Formed S.O.G.	33.33	CY	$518	$17,300	
	Total					$1,379,200
3	**Metals, Wood, and Plastics**					
	Aluminum Grating with Rebate	1510	SF	$19	$34,000	
	Aluminum Handrail	690	LF	$39	$31,000	
	Aluminum Stairs	27	RSR	$550	$17,000	
	Structural Aluminum	6145	LB	$8	$60,000	
	Al Bracket P680	6	EA	$83	$1,000	
	Redwood Baffles	760	SF	$7	$6,000	
	FRP Weir/ Launder	30	LF	$19	$1,000	
	Aluminium OSHA Handrail	212	LF	$75	$19,000	
	Aluminium Stairs, Including Railing and Supports	60	LF	$561	$40,000	
	Total					$209,000
4	**Modification and New Equipment**					
	1 Ton Base Mounted Davit Crane	1	EA	$15,369	$18,100	
	Anoxic Mixer	3	EA	$32,500	$114,600	
	Surface Aerator	4	EA	$127,700	$600,400	
	SST Slide Gate	2	EA	$10,500	$24,700	
	Denitrificatiion Filters	1	LS	$1,120,000	$1,120,000	
	New Denit-Filter Installation	1	LS	$744,950	$745,000	
	New Denit-Filter Piping Connection	1	LS	$74,495	$74,500	
	3000 gal Storage Tank	2	EA	$10,000	$20,000	

	Dosing Pumps	3	EA	$7,135	$21,400	
	18" Dimj Awwa Butterfly Valve, No Op	2	EA	$10,000	$20,000	
	16" Dimj Awwa Butterfly Valve, No Op	2	EA	$10,000	$20,000	
	Add For Ea 6" Indicator Post Over 6'	2	EA	$138	$300	
	Up To 6' Trench/Bury Indicator Post	2	EA	$2,304	$4,600	
	Magnetic Flow meters	2	EA	$16,000	$32,000	
	Vertical Turbine 20 HP Tertiary Pump	3	EA	$34,269	$102,800	
	Total					$2,918,400
5	**Pipeline**					
	Pipe Penetration Sleeve	3	EA	$280	$1,000	
	18" Cl 52 Cldi Mj Pipe In Open Trench	10	LF	$93	$1,100	
	6" 45° 125# Cldi Fxf Ell	2	EA	$937	$2,200	
	6" Flg Cldi Pipe In Open Trench	20	LF	$52	$1,200	
	18" Flg Cldi Pipe In Open Trench	60	LF	$223	$15,700	
	8" 90° 125# Cldi Fxf Ell	2	EA	$1,046	$2,500	
	8" Flg Cldi Pipe In Open Trench	60	LF	$67	$4,700	
	12" Flg Cldi Pipe In Open Trench	60	LF	$113	$8,000	
	14" New Filter Influent and 16" Filter Effluent and Fitting	1	LS	$96,351	$113,300	
	Total					$149,700
	ITEM NOS. 1-5 SUBTOTAL					**$5,069,556**
6	**Electrical, I&C and Miscellaneous**					
	Electrical Materials and MCC Modifications	1	LS	$155,000	$155,000	
	Conduits and Ductbanks	1	LS	$85,000	$85,000	
	Lighting	1	LS	$45,000	$45,000	
	Instrumentation	1	LS	$175,000	$175,000	
	Programming	1	LS	$85,000	$85,000	
	Miscellaneous Mechanical					
	Utility water to Denite Filters and Chemical Facility	1	LS	$45,000	$45,000	
	Potable water to Chemical Facility	1	LS	$35,000	$35,000	
	Emergency Eye Wash and Showers	2	LS	$7,500	$15,000	
	Pipe Supports for Denite Filters Piping	1	LS	$45,000	$45,000	
	Small Diameter Chemical Piping and Supports	1	LS	$58,000	$58,000	
	Hose Bibs and Racks	5	LS	$5,200	$26,000	
	Misc. Pipe Couplings and Fittings	1	LS	$53,000	$53,000	
	Misc. Valves and Gates	1	LS	$92,000	$92,000	
	Site and Yard Work / Paving	1	LS	$295,000	$295,000	
	Total					$1,209,000
	SUBTOTAL					**$6,278,556**
	Estimating Contingency	30	%			$1,884,000
	SUBTOTAL					**$8,162,556**
	General Conditions	10	%			$816,256
	SUBTOTAL					**$8,978,812**
	General Contractor Overhead & Profit	15	%			$1,346,822
	SUBTOTAL					**$10,325,633**
	Escalation	0	%			$0
	SUBTOTAL					**$10,325,633**
	Sales Tax on 50% of Subtotal Above	7.25	%			$296,000
	CONSTRUCTION COST SUBTOTAL					**$10,622,000**
	Engineering, Management, and Legal	25	%			$2,656,000
	PROJECT COST (April 2019 Dollars)					**$13,278,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

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OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 02 - Headworks and Influent Pump Station

LOCATION : Ojai Valley WWTP

PREPARED BY : MFS

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Demolition**					
	Remove of Rock Trap	1	LS	$10,000	$10,000	
	Remove Channel Grinder	2	LS	$10,000	$20,000	
	Remove Aluminium Slide Gate	2	LS	$5,000	$10,000	
	Remove Aluminium Stop Gate	2	LS	$5,000	$10,000	
	Total					$50,000
2	**New Equipment**					
	Slide Gate	4	LS	$30,600	$122,000	
	Bar Screen	2	EA	$335,331	$671,000	
	Bar Screen Installation	2	EA	$67,066	$134,000	
	Screenings Conveyor	80	LF	$1,635	$131,000	
	Screenings Washer/Compactor	1	EA	$64,523	$65,000	
	Screenings Washer/Compactor Installation	1	EA	$16,130.77	$16,000	
	Total					$1,139,000
	ITEM NOS. 1-2 SUBTOTAL					**$1,189,000**
3	**Allowances**					
	Electrical and Instrumentations	25	%		$297,250	
	Mechanical	15	%		$178,350	
	Structural	1	LS	$100,000	$100,000	
	Total					$576,000
	SUBTOTAL					**$1,765,000**
	Estimating Contingency	30	%			$530,000
	SUBTOTAL					**$2,295,000**
	General Conditions	10	%			$229,500
	SUBTOTAL					**$2,524,500**
	General Contractor Overhead & Profit	15	%			$378,675
	SUBTOTAL					**$2,903,175**
	Escalation	0	%			$0
	SUBTOTAL					**$2,903,175**
	Sales Tax on 50% of Subtotal Above	7.25	%			$84,000
	CONSTRUCTION COST SUBTOTAL					**$2,987,000**
	Engineering, Management, and Legal	25	%			$747,000
	PROJECT COST (April 2019 Dollars)					**$3,734,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 03 - Grit Chamber

LOCATION : Ojai Valley WWTP

PREPARED BY : MFS

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Demolition**					
	Remove Grit Mixer	1	LS	$5,000	$5,000	
	Total					$5,000
2	**Modification and New Equipment**					
	Pista Grit Mixer	1	LS	$114,450	$114,000	
	Pista Grit Mixer Installation	1	EA	$22,890	$23,000	
	Channel Modification	1	LS	$109,000	$109,000	
	Total					$246,000
	ITEM NOS. 1-2 SUBTOTAL					**$251,000**
3	**Allowances**					
	Electrical and Instrumentations	15	%		$37,650	
	Mechanical	15	%		$37,650	
	Structural	1	LS	$50,000	$50,000	
	Total					$125,000
	SUBTOTAL					**$376,000**
	Estimating Contingency	30	%			$113,000
	SUBTOTAL					**$489,000**
	General Conditions	10	%			$48,900
	SUBTOTAL					**$537,900**
	General Contractor Overhead & Profit	15	%			$80,685
	SUBTOTAL					**$618,585**
	Escalation	0	%			$0
	SUBTOTAL					**$618,585**
	Sales Tax on 50% of Subtotal Above	7.25	%			$18,000
	CONSTRUCTION COST SUBTOTAL					**$637,000**
	Engineering, Management, and Legal	25	%			$160,000
	PROJECT COST (April 2019 Dollars)					**$797,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

Engineers...Working Wonders With Water®

OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 04 - Sludge Dewatering

LOCATION : Ojai Valley WWTP

PREPARED BY : MFS

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Demolition**					
	Remove WAS Pumps	2	LS	$5,000	$10,000	
	Remove Belt Press	1	LS	$25,000	$25,000	
	Remove Conveying System	1	LS	$5,000	$5,000	
	Removing Dewatering Polymer System	1	LS	$5,000	$5,000	
	Total					$45,000
2	**New Equipment**					
	Dewatering Screw Press & Control Panel	2	EA	$544,570	$1,089,000	
	Dewatering Polymer System	2	LS	$51,920	$104,000	
	Secondary Polymer System	2	LS	$44,250	$89,000	
	Screw Press Air Compressor	1	EA	$20,060	$20,000	
	Cake Conveyor	80	LF	$2,478	$198,000	
	Cake Conveyor Install	1	LS	$16,520	$17,000	
	Monorail System	0	LS	$224,200.00	$0	
	Total					$1,517,000
	ITEM NOS. 1-2 SUBTOTAL					**$1,562,000**
3	**Allowances**					
	Electrical and Instrumentations	20	%		$312,400	
	Mechanical	15	%		$234,300	
	Structural	1	LS	$100,000	$100,000	
	Total					$647,000
	SUBTOTAL					**$2,209,000**
	Estimating Contingency	30	%			$663,000
	SUBTOTAL					**$2,872,000**
	General Conditions	10	%			$287,200
	SUBTOTAL					**$3,159,200**
	General Contractor Overhead & Profit	15	%			$473,880
	SUBTOTAL					**$3,633,080**
	Escalation	0	%			$0
	SUBTOTAL					**$3,633,080**
	Sales Tax on 50% of Subtotal Above	7.25	%			$105,000
	CONSTRUCTION COST SUBTOTAL					**$3,738,000**
	Engineering, Management, and Legal	25	%			$935,000
	PROJECT COST (April 2019 Dollars)					**$4,673,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

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OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 05 - Tertiary Filtration

LOCATION : Ojai Valley WWTP

PREPARED BY : MFS

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Channel Modification**					
	Channel Modification	1	LS	$50,000	$50,000	
	Total					$50,000
	ITEM NOS. 1 SUBTOTAL					**$50,000**
	SUBTOTAL					**$50,000**
	Estimating Contingency	30	%			$15,000
	SUBTOTAL					**$65,000**
	General Conditions	10	%			$6,500
	SUBTOTAL					**$71,500**
	General Contractor Overhead & Profit	15	%			$10,725
	SUBTOTAL					**$82,225**
	Escalation	0	%			$0
	SUBTOTAL					**$82,225**
	Sales Tax on 50% of Subtotal Above	7.25	%			$3,000
	CONSTRUCTION COST SUBTOTAL					**$85,000**
	Engineering, Management, and Legal	25	%			$22,000
	PROJECT COST (April 2019 Dollars)					**$107,000**

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OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 06 - UV Disinfection

LOCATION : Ojai Valley WWTP

PREPARED BY : MFS

REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	UV Equipment					
	Replace UV Equipment (Trojan UV3000 Plus)	1	LS	$880,000	$880,000	
	Installation	50	%		$440,000	
	Total					$1,320,000
	ITEM NOS. 1 SUBTOTAL					**$1,320,000**
2	Allowances					
	Electrical and Instrumentation	15	%		$198,000	
	Bypass Pumping				$50,000	
	Mechanical	15	%		$198,000	
	Total					$446,000
	SUBTOTAL					**$1,766,000**
	Estimating Contingency	30	%			$530,000
	SUBTOTAL					**$2,296,000**
	General Conditions	10	%			$229,600
	SUBTOTAL					**$2,525,600**
	General Contractor Overhead & Profit	15	%			$378,840
	SUBTOTAL					**$2,904,440**
	Escalation	0	%			$0
	SUBTOTAL					**$2,904,440**
	Sales Tax on 50% of Subtotal Above	7.25	%			$84,000
	CONSTRUCTION COST SUBTOTAL					**$2,988,000**
	Engineering, Management, and Legal	25	%			$747,000
	PROJECT COST (August 2019 Dollars)					**$3,735,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.

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OJAI VALLEY SANITARY DISTRICT

2019 Facilities Plan

TASK : Ojai Valley CIP Cost Estimate

ESTIMATE PREPARATION DATE : Nov-19

JOB # : Proj No. 07 - RW Program or Higher Quality Effluent
LOCATION : Ojai Valley WWTP

PREPARED BY : MFS
REVIEWED BY : RD

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT COST	SUBTOTAL	TOTAL
1	**Microfiltration**					
	MF Equipment (3 + 1 trains) incl CIP, pumps	2,000,000	$/gal	$0.35	$700,000	
	Installation	30	%		$210,000	
	Total					$910,000
2	**Reverse Osmosis**					
	RO Equipment (2 + 1 trains) incl CIP, pumps, CF	1,750,000	$/gal	$0.50	$875,000	
	Installation	30	%		$262,500	
	Total					$1,137,500
3	**UV/AOP**					
	UV/AOP Equipment - Incl chemical dosing	1	LS	$575,000	$575,000	
	Installation	30	%		$172,500	
	Total					$747,500
4	**EDR**					
	EDR Equipment - 2 x 2 line, 4 train ; incl CIP	1	LS	$2,200,000	$2,200,000	
	Installation	30	%		$660,000	
	Total					$2,860,000
5	**Concentrate Storage Tanks**					
	30000 Gallon Tank	2	EA	$100,000	$200,000	
	Concrete Pad / Truck Loading	1	LS	$55,000	$55,000	
	Total					$255,000
6	**Common Process Building**					
	Facility Building 60' x 150'	9000	$/SF	$250	$2,250,000	
	Total					$2,250,000
	ITEM NOS. 1-6 SUBTOTAL					**$8,160,000**
7	**Allowances**					
	Electrical	10	%		$816,000	
	I&C	5	%		$408,000	
	Yard and Site Work	8	%		$652,800	
	Total					$1,877,000

	SUBTOTAL					**$10,037,000**
	Estimating Contingency	30	%			$3,012,000
	SUBTOTAL					**$13,049,000**
	General Conditions	10	%			$1,304,900
	SUBTOTAL					**$14,353,900**
	General Contractor Overhead & Profit	15	%			$2,153,085
	SUBTOTAL					**$16,506,985**
	Escalation	0	%			$0
	SUBTOTAL					**$16,506,985**
	Sales Tax on 50% of Subtotal Above	7.25	%			$474,000
	CONSTRUCTION COST SUBTOTAL					**$16,981,000**
	Engineering, Management, and Legal	25	%			$4,246,000
	PROJECT COST (August 2019 Dollars)					**$21,227,000**

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Appendix C

CAROLLO 2018 UV SYSTEM EVALUATION REPORT

Ojai Valley Sanitary District
Ojai Valley WWTP UV System Evaluation

UV System Evaluation Results Report

DRAFT | August 2018

Andrew Salveson 08/31/18
CA PE No. C56902
Nicola Fontaine 08/31/18
CA PE No. 76863

Contents

Appendices

Tables

Figures

Abbreviations

BOD	Biological Oxygen Demand
Carollo	Carollo Engineers, Inc.
CB	Collimated Beam
DDW	Division of Drinking Water
EOLL	End of Lamp Life
FF	Fouling Factor
gpm	gallons per minute
gpm/ft^2	gallons per minute per square foot
HMI	Human Machine Interface
µm	micron
MG	million gallons
mg/L	milligrams per liter
mgd	million gallons per day
mJ/cm^2	millijoules per square centimeter
mL	milliliter
MPN	Most Probable Number
nm	nanometers
NPDES	National Pollutant Discharge Elimination System
NWRI	National Water Research Institute
O&M	Operation and Maintenance
OVSD	Ojai Valley Sanitary District
PSD	Particle Size Distribution
RED	Reduction Equivalent Dose
SFF	Sleeve Fouling Factor
TDS	Total Dissolved Solids
TSS	Total Suspended Solids
UV	Ultraviolet
UVI	Ultraviolet Intensity
UVT	Ultraviolet Transmittance
WWTP	Wastewater Treatment Plant

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Summary

This work was completed in response to total coliform violations in the final effluent from the Ojai Valley Sanitary District (OVSD) Wastewater Treatment Plant (WWTP). The cause for these violations is often found to be from more than one reason at other sites.

This work examines the concepts above through a three step process: (1) Hypothesis of Impact, (2) Data Collection, and (3) Analysis and Conclusions, all of which is detailed in the main body of this report. In total, the following conclusions can be made based upon the work and effort documented herein:

- Compliance
  - Over a period of June 2017 to August 2018, the total coliform levels were at 2.2 most probable number per 100 milliliters (MPN/100 mL) or below 80 percent of the time. A higher level of compliance reliability is desired (e.g., 90 to 95 percent or greater).
  - The Colilert method used for OVSD investigations typically results in higher measured concentrations than Standard Method SM9222B (multiple tube fermentation method). SM9222B is currently required for all compliance monitoring.
  - Continue to sample for compliance immediately downstream of UV Bank E as required by the permit. Sampling further downstream will result in compliance challenges due to regrowth.
- Filter Performance
  - High filter effluent total coliform concentrations likely result in high ultraviolet (UV) system effluent total coliform concentrations. Said another way, the UV system does not appear to have sufficient dose capacity, as currently controlled, to reliably disinfect high influent total coliform concentrations.
  - Current information regarding shock chlorination of the filters does not indicate a significant improvement to post filter total coliform concentrations.
  - For the days analyzed, the filters were operating within design conditions and provided adequate conditioning of the water ahead of UV for disinfection.
  - For total coliform, filtration measurably improves the UV dose-response. For filtered effluent, a UV dose of at least 30 mJ/cm^2 is needed to meet the 2.2 MPN/100 mL geometric mean target. Note that this dose is a VALIDATED UV dose and is not the UV dose shown on your existing UV system Human Machine Interface (HMI).
  - For the examined secondary effluent and for this day of sampling, a UV system with a dose of 30 to 50 mJ/cm^2 could meet the 2.2 MPN/100 mL compliance target without filtration. This implies a high quality unfiltered secondary effluent (on the day of sampling). Note that this dose is a VALIDATED UV dose and is not the UV dose shown on your existing UV system HMI.
  - Particle count analysis (particle size distribution) supports the filter performance and collimated beam results; particle loading (number and size) in the filter effluent are not anticipated to impact UV disinfection. OVSD has installed an online particle counter after filtration to continuously monitoring performance.

- Ultraviolet Disinfection
  - The online UV transmittance (UVT) analyzer is inaccurate and not conservative, by more than 10 percent. The subsequent impact on calculated dose is substantial, grossly overestimating applied UV dose.
  - Power feed to the UV system matches expectations, thus there does not appear to be any impact due to power irregularities.
  - 20 to 30 percent of the UV intensity was lost due to sleeve fouling, but chemical cleaning (in this case by hand) was able to return the UV quartz sleeve relative transmittance back to 100 percent.
  - UV intensity values vary by about 10 percent between the different UV intensity sensors. These values are not used for system control, but must be better trended to understand the reduction in intensity due to lamp aging.
  - UV reactor challenge testing, with different bank combinations in operation over a range of flow indicated that water level is impacting disinfection performance. The water level exceeded the downstream design water level of 24-inches and this increased level is amplified over each bank the further upstream from the level control gate.
  - Bank D and Bank E are the most effective banks due to lower water level.
  - The recommended ***near term*** control system is **flow pacing** control. The data presented indicates that two UV banks, fully operational with lamps near their end of lamp life and with cleaned quartz sleeves, are sufficient for compliance ~75 percent of the time under the flow test conditions. As quartz sleeves will foul and because there will be occasions where lamps will be out in a bank (see bullet on this below), the UV system must operate with two or more banks in service. Based upon the data in hand, the following bank operational strategy is recommended:
    - For any combination of 2 or more operating banks, Bank D or Bank E must be operating. This will ensure that one of the two most effective banks is in use due to the lower water level.
    - At UVTs greater than 65 percent and less than 70 percent
      - Minimum two banks in operation at flows less than or equal to 1.8 mgd
      - Minimum three banks in operation for flows of 1.8 to 2.8 mgd
      - Minimum four banks in operation for flows of 2.8 to 3.8 mgd
      - Minimum five banks in operation for flows greater than 3.8 mgd
    - At UVTs greater than 70 percent
      - Minimum two banks in operation at flows less than or equal to 2.1 mgd
      - Minimum 3 banks in operation for flows 2.1 to 3.3 mgd
      - Minimum 4 banks in operation for flows greater than 3.3 mgd
  - Based upon **flow pacing** control, a number of key parameters must be closely monitored and trended, as discussed in detail in this report. These are:
    - Quartz sleeve cleaning
    - UV lamp replacement
    - UV intensity
    - UV transmittance
    - Filter effluent turbidity
    - Filter effluent particle counts

- The recommended ***long term*** control system is **dose pacing** control. This type of control requires an extensive detailed bioassay testing and reprogramming of the system with a high degree of accuracy. The system would then modulate banks in operation based upon real time parameters:
  - UV intensity
  - UV transmittance
  - Flow
- Based upon **dose pacing** control, a number of key parameters must be closely monitored and trended, as discussed in detail in this report. These are:
  - Quartz sleeve cleaning, starting at a 3-week interval, but using UV intensity readings to refine the cleaning interval.
  - UV lamp replacement based upon UV intensity values and a maximum lamp age determined by the Supplier.
  - UV intensity
  - UV transmittance
  - Filter effluent turbidity
  - Filter effluent particle counts

A summary table of the investigations is included in Appendix A. This work also highlighted several future recommended efforts summarized in a table in Appendix B.

Introduction and Background

The OVSD WWTP provides wastewater treatment for the City of Ojai and the surrounding unincorporated Ojai Valley, producing water quality suitable for discharge to the Ventura River (under the terms of National Pollutant Discharge Elimination System [NPDES] permit CA0053961 Order R4-2013-0173).

Recently the OVSD has had problems with excursions in bacterial compliance. The purpose of this project was to evaluate the WWTP filtration and UV disinfection processes to determine the cause of the excursions and to recommend methods to increase disinfection reliability and operational efficiency.

Building on the OVSD efforts to date, Carollo Engineers, Inc. (Carollo) prepared a test plan to evaluate the filters and UV system (Ojai Valley WWTP UV System Evaluation Test Plan, 4/12/18). This report summarizes the results and recommendations from the evaluation.

Wastewater Treatment Plant

The WWTP is a tertiary plant with a dry weather design capacity of three (3) million gallons per day (mgd) and an instantaneous peak flow capacity of 9 mgd. Untreated wastewater is collected from the City of Ojai, the unincorporated communities of Meiners Oaks, Mira Monte, Oak View, Casitas Springs, Foster Park, and North Ventura Avenue area through approximately 120 miles of sanitary sewer lines.

The WWTP provides a high level of treatment with nutrient removal, filtration, and disinfection. The WWTP consists of influent grinding, grit removal and screening, primary sedimentation, activated sludge treatment with using an oxidation ditch with an anaerobic-anoxic and aerobic zones for biological oxygen demand (BOD), nitrogen, and phosphorus removal, secondary sedimentation, tertiary filtration, UV disinfection, and reaeration through static aerators prior to discharge. As a backup, the WWTP can use chlorination to disinfect the effluent. Equalization basins allow for evening out diurnal flows to the tertiary filters. The tertiary facilities were designed for an average flow of 3 mgd and a peak flow of 4.3 mgd. Treated effluent is discharged at Discharge Point 001 to the Ventura River.

Tertiary Filtration

The WWTP has four DynaSand filters that were installed in 1996. The deep bed, sand, continuous backwash filters were designed for a peak hydraulic loading rate of 5 gallons per minute per square foot (gpm/ft^2). With one filter out of service the capacity of the tertiary filters is 4.32 mgd.

There has been no equipment replacements other than filter air lift end assemblies and air lift tubes due to sand abrasion and wear since installation. The filter lifts are pulled up and cleared of algae and plastics on a weekly basis.

UV Disinfection

The Fischer & Porter 70UV6000 UV system was put in service in 1996 and has performed well for approximately 20 years. The UV system consists of one channel with five banks. The system was designed for a UV dose of 100 millijoules per square centimeter (mJ/cm^2) at a UVT of 55 percent. Note that this dose is not required in the NPDES permit. The disinfection component of the NPDES permit focuses solely upon total coliform disinfection.

The design was based on manufacturer validation data and a simplified approach that was in accordance with industry standards at the time of installation. Since installation, the National Water Research Institute (NWRI) has issued detailed UV Guidelines that specify much more accurate methods for sizing and operating UV systems (with a 2003 version, a 2012 version, and now a 2018/2019 version under development). Accordingly, without extensive "Validation" testing of the installed reactor, it is difficult to precisely estimate the true capacity of the installed system. Said another way, when your current UV reactor says that the UV dose is "X", the actual delivered dose is in fact much lower than this predicted value.

The UV control system currently uses the Point Source Summation method to calculate dose. It accounts for flow and UVT, but not UV intensity (UVI).

Compliance Requirements

The OVSD is regulated under the NPDES effluent requirements for water discharged to the Ventura River. NPDES effluent limits are contained in Table 1.

Table 1 NPDES Effluent Discharge November 2013 Permit

Permit Item	Compliance
BOD_5	10 mg/L monthly average 15 mg/L daily maximum
Total Suspended Solids (TSS)	10 mg/L monthly average 15 mg/L daily maximum
pH	6.5 instantaneous minimum 8.5 instantaneous maximum
Total Dissolved Solids (TDS)	1500 mg/L monthly average
Temperature	Not to exceed 86°F or higher than receiving water
Turbidity	Average 2 NTU within a 24-hour period 5 NTU more than 5 % of the time (72 minutes) within a 24-hour period 10 NTU at anytime
Total Coliform	7-day median - 2.2 MPN/100 mL Not exceed in more than 1 sample in any 30 day period - 23 MPN/100 mL Never exceed 240 MPN/100mL in any 30 day period

As the WWTP does not currently produce recycled water, all requirements for recycled water are listed as "not applicable" in the NPDES permit. The requirements for turbidity and total coliform do reflect the requirements for recycled water but the use of these in the NPDES permit is likely intended to protect the beneficial use of the Ventura River.

Disinfection Performance History

The OVSD provided historical data on the UV system operation and disinfection performance from July 24, 2017 through August 3, 2018. Figure 1 presents the data from this period with the

7-day median total coliform effluent limit of 2.2 MPN/100 mL shown in red. Figure 1 is missing two samples from October 26 and 27, 2017 that had a total coliform concentrations of 140 and greater than 1600 MPN/100 mL, respectively. These were removed for clarity. After those samples in October the filters were chlorinated and the total coliform effluent concentration returned to less than 2.2 MPN/100 mL.

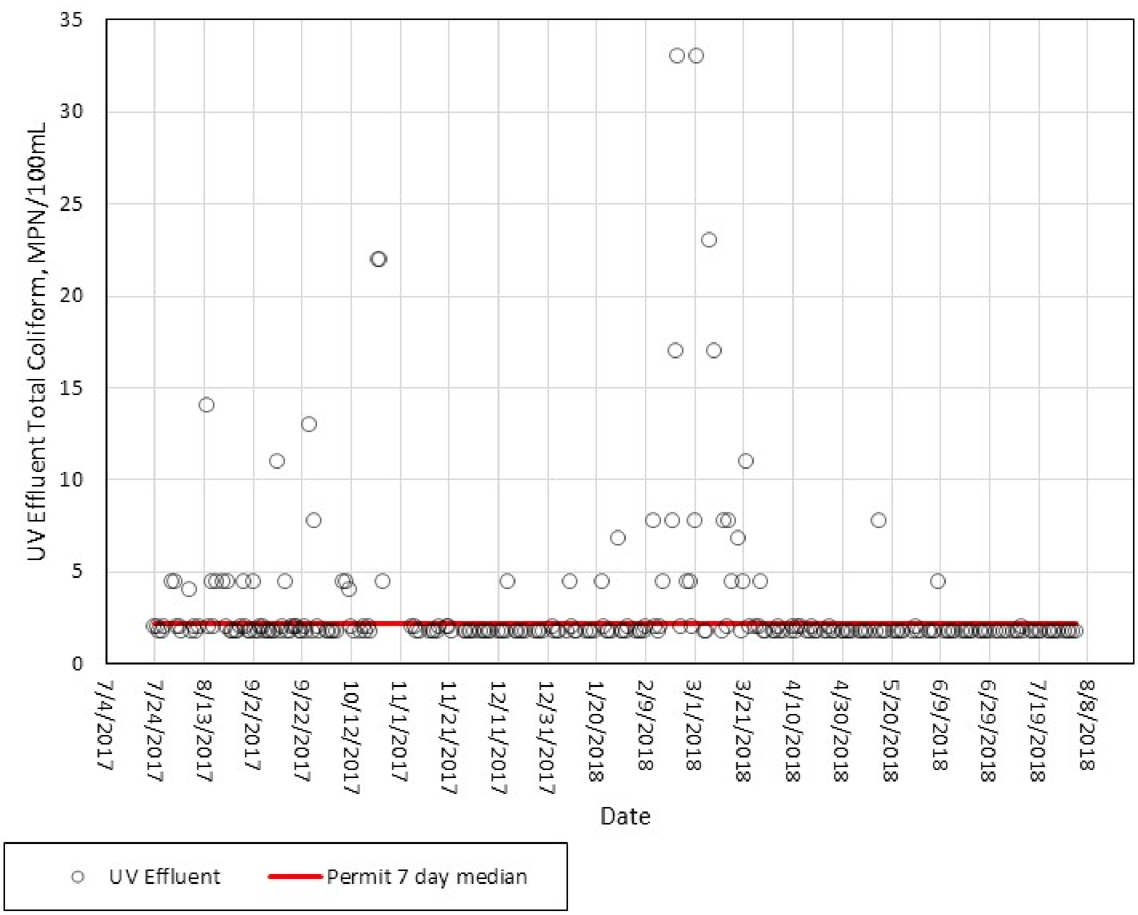


Figure 1 UV Effluent Total Coliform Concentration - July 24, 2017 to August 3, 2018

Figure 2 presents the percentile graph of the total coliform data from this time period. This figure includes the two October data points. During this time period the OVSD was below the total coliform permit limit of 2.2 MPN/100 mL 80 percent of the time. **A reasonable goal would be to reduce bacterial numbers to below the targets for 90 to 95 percent of the time, a large improvement in overall reliability.**

The data presented in Figure 1 and 2 includes data during system cleaning (filter influent channel, UV channels, etc.) and shock chlorination of the filters; performed by the OVSD in an effort to remedy performance issues. The results of this shock chlorination are presented in Table 2.

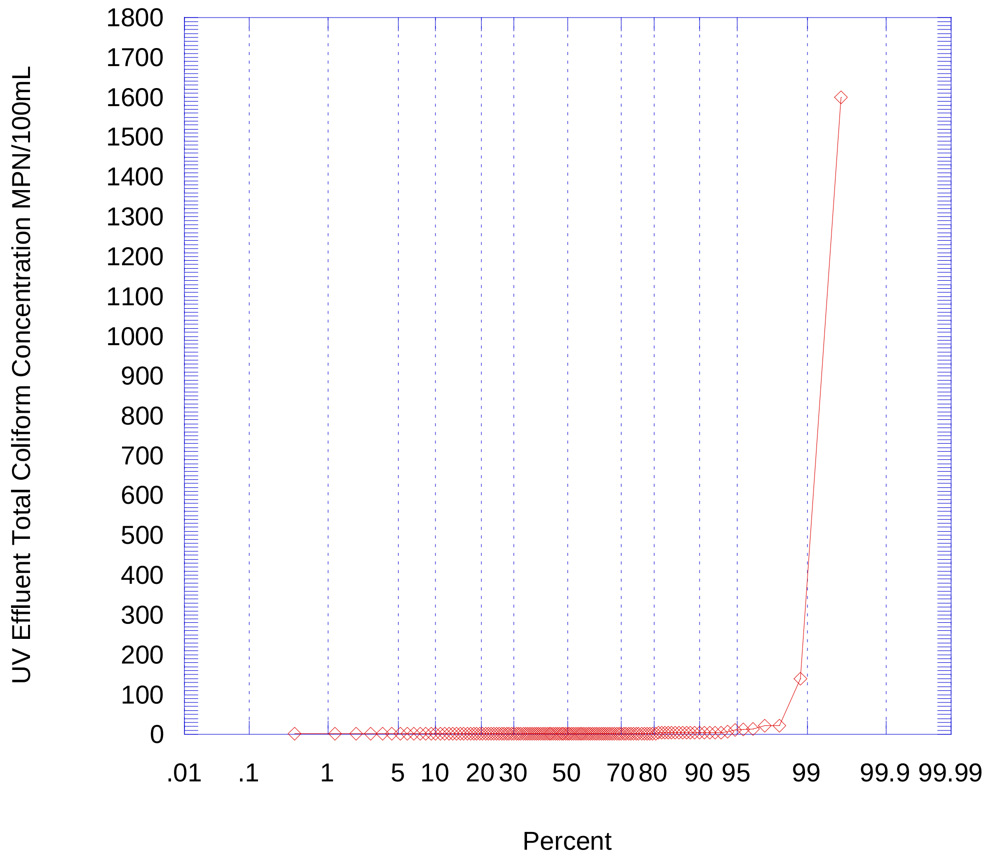


Figure 2 UV Effluent Total Coliform Concentration Percentile Graph - July 24, 2017 to August 3, 2018

Analysis and Conclusions

Review of the historical data and the dates of the cleaning and shock chlorination events do not appear to show an improvement in water quality (total coliform levels) after such events. Repeat testing of shock chlorination as well as impact of cleaning the upstream channel area ahead of filtration to reduce total coliform is recommended.

Table 2 Water Quality Before and After Cleaning and Shock Chlorination Events - March 2018

Date	Event	No. Filters Online	Filter Loading Rate, gpm/ft^2	Filter Influent Total Coliforms, MPN/100 mL [(1)]	Filter Effluent Total Coliforms, MPN/100 mL [(1)]	Filter Effluent Turbidity, NTU [(2)]	UV Effluent Total Coliforms, MPN/100 mL [(1,4)]	No. Banks On-Line
2-26-18	Normal Operation	3	2.2	48,840	7,540	0.81	4.5	5
2-28-18	Normal Operation	3	2.3	26,130	2,620	0.61	2.0	3
3-1-18	Cleaned Filter Influent Channel							
3-2-18	Normal Operation	3	2.6	61,310	10,500	0.83	33	3
3-5-18	Chlorinated Filters[(3)]							
3-6-18	Chlorinated Filters[(3)]							
3-7-18	Normal Operation	3	2.3	155,310	34,480	0.91	23	3
3-9-18	Normal Operation	3	2.3	241,960	81,640	0.90	17	3
3-12-18	Normal Operation	3	2.5	173,290	21,870	0.67	<1.8	3
3-14-18	Normal Operation	3	2.4	104,620	54,750	0.73	2.0	3

Notes:
(1) Samples analyzed by Colilert.
(2) On-line turbidimeter.
(3) 3.5 hours at 16 mg/L sodium hypochlorite.
(4) Chlorine contact basin was used for disinfection during this entire period. Total coliform values in effluent was <1.8 MPN/100 mL. Extra samples were collected from UV channel for the purposes of evaluating performance.

Evaluation of Filtration Operation and Filter Effluent Water Quality

Hypothesis

UV performance can be directly impacted by the passage of large and numerous particles that shield UV light from disinfection. Filters, either through operating outside of acceptable ranges (e.g., flux) or due to media loss or mudballs or other reasons, can provide preferential flow pathways through the filter that pass (instead of remove) particles. The testing below was intended to examine if filter performance may be a reason for UV compliance issues.

Data Collection

Conventional Filter Data

Single day sampling was conducted May 7, 2018 to further evaluate filter performance and to determine the approximate UV dose to meet disinfection performance targets. Table 3 contains the operational data of the WWTP and sample characteristics on May 7, 2018.

Table 3 WWTP Operational Conditions and Sample Characteristics May 7, 2018 - Single Day Sampling

Parameter	Units	Value
Flow to Filters	mgd	2.0
Number of Filters On-line	no.	2
Filter Loading Rate	gpm/ft^2	3.5
Filter Influent Turbidity(1)	NTU	1.43
Filter Effluent Turbidity(1)	NTU	0.57
Filter Effluent Turbidity(2)	NTU	0.37
Filter Influent Total Coliform(3)	MPN/100 mL	64,880
Filter Effluent Total Coliform(3)	MPN/100 mL	22,470
Filter Influent Total Coliform(4)	CFU/100 mL	47,500
Filter Effluent Total Coliform(4)	CFU/100 mL	22,500
Filter Influent UVT(1)	%	65
Filter Effluent UVT(1)	%	67
UV Banks On-line	no.	3
UV Effluent Total Coliform	MPN/100 mL	<1.8

Notes:
(1) Measured in samples collected by laboratory.
(2) On-line turbidimeter.
(3) Samples analyzed by Colilert. Unit's most probable number of bacteria per 100 mL.
(4) Samples analyzed by Membrane Filtration Method SM9222B. Unit's colony forming units per 100 mL.

Collimated Beam Dose Response

A collimated beam (CB) test is a bench-scale test that is used to determine the UV dose response of a microorganism, in this case total coliforms. The CB test, for the date and time of sampling, can clearly demonstrate the impact of water quality on UV disinfection performance and the approximate UV dose needed to hit a particular compliance target. A CB test on a water with high solids, for example, will require a higher dose to meet a particular target bacteria concentration, or may even be unable to hit a low bacterial concentration target at all due to shielding of bacteria by wastewater solids. For the CB test, both time and UV light intensity are directly measured. The UV dose is calculated using the intensity of the incident UV light, UVT of the water, and exposure time. UVT is the amount of UV light with a wavelength of 254 nanometers (nm) that passes through 1 centimeter (cm) of water.

The OVSD collected samples prior to and after filtration for CB analysis on total coliforms on May 7, 2018. CB analysis was performed by GAP EnviroMicrobial Services (London, Ontario, Canada). Sampling pre- and post-filtration was conducted to document the importance (or lack thereof) of filtration to downstream UV disinfection performance. The results of the total coliform CB testing on the pre-and post-filtration samples are presented in Figure 3. The UVT and turbidity results for each sample are shown in the figure.

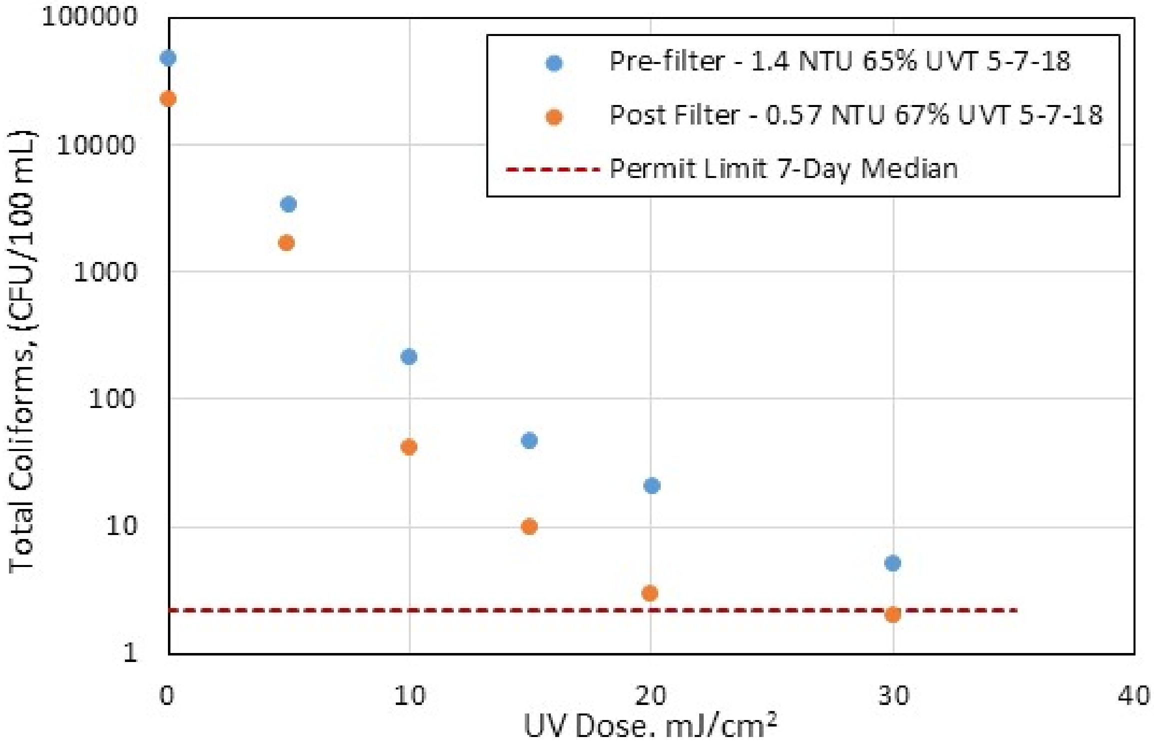


Figure 3 Filter Influent and Effluent Total Coliform Collimated Beam Analysis

Particle Size Distribution

To understand further the filter performance, samples were collected concurrent with the CB samples and were analyzed for UVT, turbidity, and particle size distribution (PSD). PSD is used to directly assess filter treatment performance, to understand more about the nature of particles that comprise total suspended solids (TSS), and to evaluate characteristics that cannot be explained with turbidity. Turbidity alone does not accurately measure for larger sized particles (i.e., larger than 7 micron [µm] in diameter) that can shield microorganisms.

The optical particle size analyzer used for this project was an AccuSizer 780 syringe injection sampler that is manufactured by Particle Sizing Systems of Santa Barbara, California. Samples that are analyzed for PSD are drawn through a small photozone (a narrow, slab-like region of uniform illumination produced by a laser diode) in the particle counter at a constant flow rate. The passage of particles through the photozone causes a pulse that is measured by a photodetector. The particles in suspension are sufficiently dilute so the particles pass, one at a time through the illuminated region of the photozone, avoiding those coincidence errors. The magnitude of the pulse varies depending on the mean diameter of the particle and the physical detection method. The illumination/detection system in the sensor is designed to provide monotonic increase in pulse height with increasing particle diameter. Results are generated by comparing the individual particle pulse heights with a standard calibration curve obtained from a set of uniform latex spherical particles with known diameters. The AccuSizer 780 has the ability to count and size particles in 128 size channels between 1 and 400 µm.

PSD testing is a useful tool that can be used to document the size and concentration of particles in the feed and filtrate of the disc filters, thus clearly showing the particle removal efficiency obtained by the filters. With a sufficient database of PSD results, conclusions can be readily made regarding the condition of filters due to water quality and operation and maintenance (O&M) procedures.

Figure 4 contains the PSD graph from the single day sampling of pre- and post-filtration. Figures 5, 6, and 7 present the filter effluent samples collected during the stress testing of the filter. Table 4 summarizes the filter influent and effluent total coliform concentrations with filter effluent turbidity and number of particles greater than 7 µm in diameter for the samples collected from May 14 to May 17, 2018.

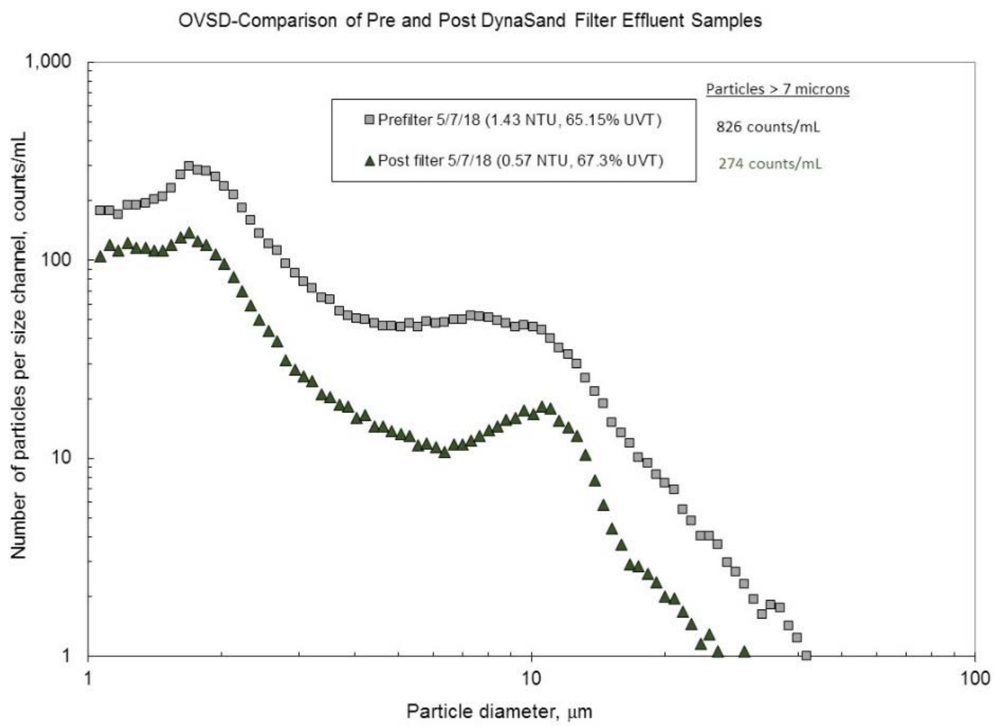


Figure 4 Filter Influent and Filter Effluent Particle Size Distribution - May 7, 2018

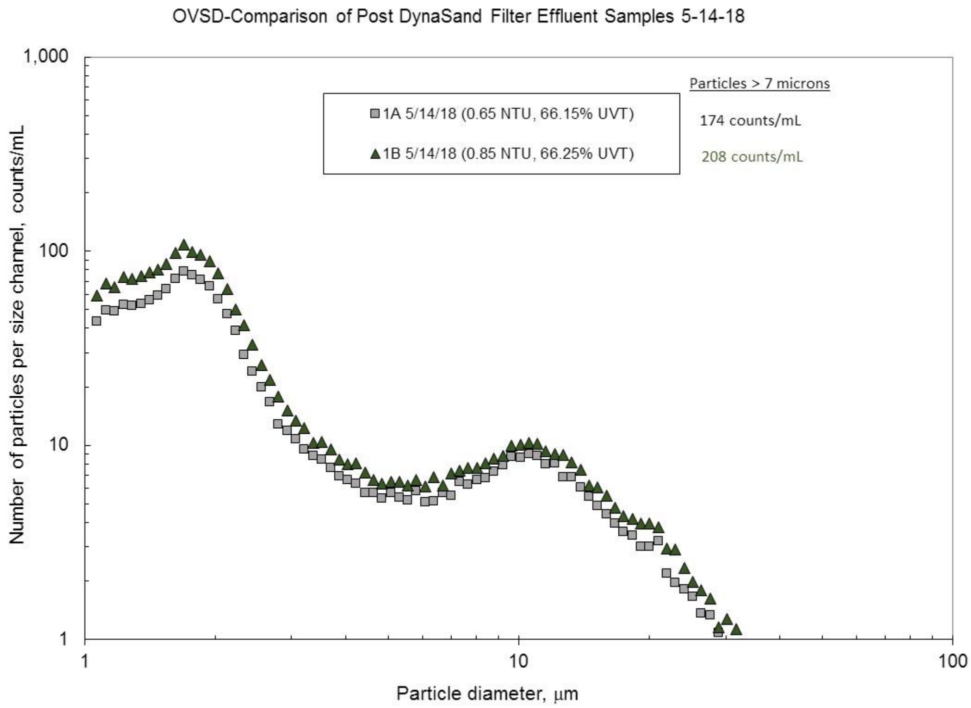


Figure 5 Filter Effluent Particle Size Distribution - May 14, 2018

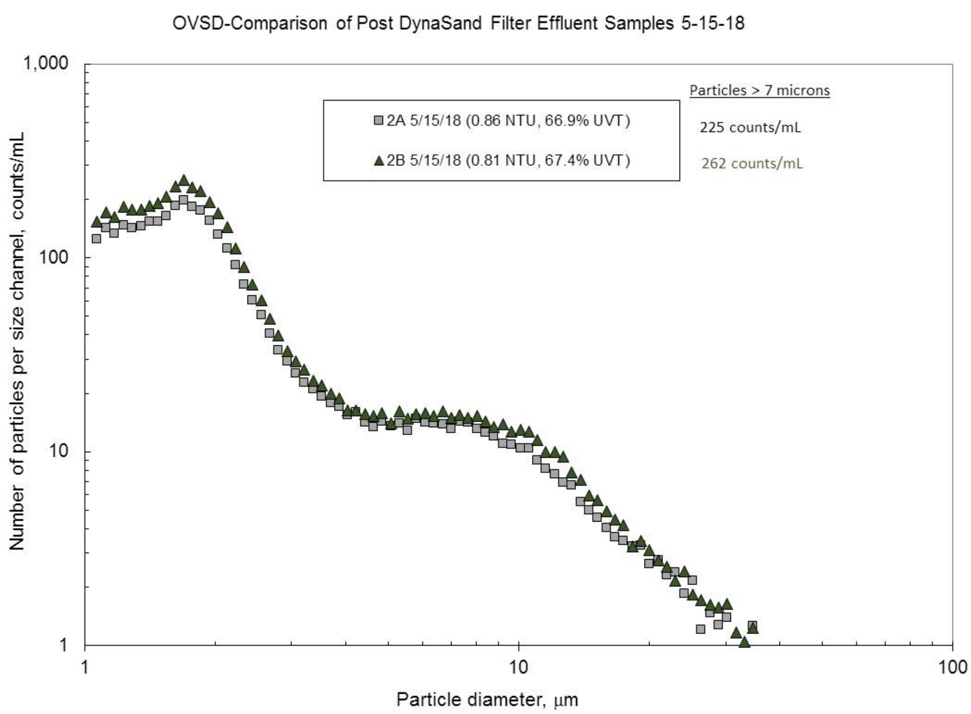


Figure 6 Filter Effluent Particle Size Distribution - May 15, 2018

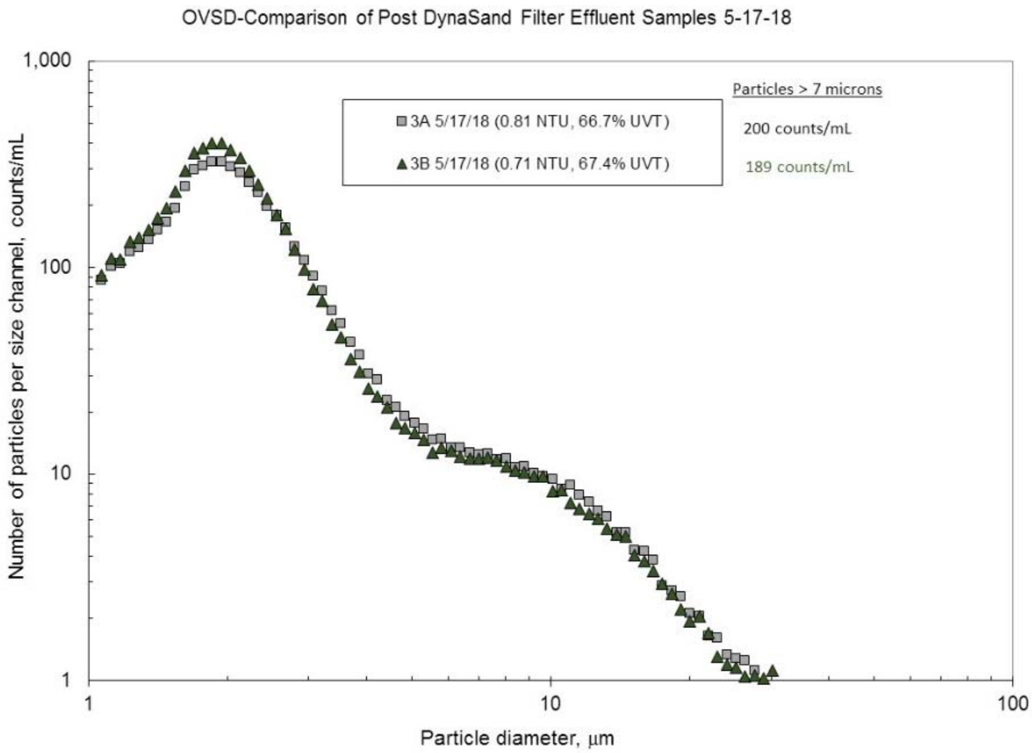


Figure 7 Filter Effluent Particle Size Distribution - May 17, 2018

Table 4 Summary of Filter Effluent Characteristics for Samples Collected May 14 to May 17, 2018

Date	Average Total Coliform Concentration, MPN/100mL	Average No. of Particles > 7 µm Diameter	Average Turbidity, NTU	Average UVT, %
5-14-18	1600	191	0.75	66.2
5-15-18	1700	244	0.84	67.2
5-17-18	2400	195	0.76	67.1

Notes:
(1) Average of two samples collected during PSD sample collection.
(2) Coliform enumeration by multiple tube fermentation SM9221B.
(3) Turbidity and UVT measured in the laboratory.

Analysis and Conclusions

Conventional Filter Data

The data presented in Table 3 indicates the filters were operating within design conditions and provided adequate conditioning of the water for disinfection. Filtration increased UVT by 2 percent and turbidity was reduced from 1.4 to 0.57 NTU, which is a reasonable level of reduction of solids.

A difference in bacterial enumeration methods was inconclusive since the filter effluent values between Colilert and SM9222B (membrane filtration) were similar. The difference between the filter influent values, where Colilert values were higher than SM9222B reflects what has been seen by Carollo at other sites in comparison to SM9221B (multiple tube fermentation). Further method comparison was run during the stress testing that supported the conclusion that the Colilert method typically results in higher concentrations than 9221B. Appendix C contains a method comparison graph. Not all samples run were enumerated by both methods. Note that Colilert is not approved for wastewater compliance monitoring at the time of this report.

Collimated Beam Dose Response

The collimated beam data in Figure 3 suggest the following:

1. For total coliform, filtration measurably improves the UV dose response. For filtered effluent, a UV dose of at least 30 mJ/cm2 is needed to meet the 2.2 MPN/100 mL geometric mean target.
2. For the examined secondary effluent and for this day of sampling, a UV system with a dose of 30 to 50 mJ/cm2 could meet the 2.2 MPN/100 mL compliance target without filtration. This implies a high quality unfiltered secondary effluent (on the day of sampling).

The dataset used to generate the conclusions for this study is clearly small. Samples collected and results obtained from other days and times may indicate different results, especially during filter upsets. However, for this data set in hand, the overall impact of filtration to assist UV performance to meet NPDES criteria was shown to be important. While filtration is important for the performance of the UV system, a properly sized and maintained UV system could meet permit compliance without filtration.

Particle Size Distribution

Per Figure 4, the overall measured particle removal by the filter, over the size range of 1 to 400 μm, was 43 percent. This decrease in particle removal correlates with the turbidity reduction of 40 percent. For particles greater than 7 μm, the size of particles that can potentially shield bacteria from UV disinfection resulting in reduced disinfection performance, the filter removed 33 percent of particles. The 33 percent removal of the particles by the filtration process on sufficiently conditions the water so it can be more efficiently disinfected by the UV disinfection system. The CB results support this conclusion that the filtration process sufficiently conditions the filter effluent so that the disinfection permit limits can be satisfied by the UV system. **Note:** these results and conclusions reflect a single day.

The particles per mL greater than 7 μm in the filter effluents collected on May 14, 15, and 17, 2018 were similar to the sample collected on May 7, 2018, ranging from 174 to 262 particles per mL.

The data in Table 4 confirms that the filtration process sufficiently conditions the water so it can be more efficiently disinfected by the UV disinfection system. Correlations on filter performance cannot be made without capturing samples during a filter upset.

Future Sampling Recommendations

It is recommended that OVSD plan to collect samples for PSD and CB testing during a filter upset in the future. This will help to better understand the impact of filter upsets on UV disinfection performance. Coolers have been provided to conduct this testing in the future.

Chemical Addition and Potential Coagulation and Sedimentation of Solids Ahead of Filtration

Hypothesis

After chemical addition, the large wet well from the point of chemical injection to the distribution of water to filtration appears to provide an opportunity for coagulation and sedimentation of solids which are anticipated to harbor large numbers of total coliform. These higher total coliform concentrations could be a component of the periodic high coliform concentrations in the filter effluent.

Data Collection

No data has been collected to date to investigate this hypothesis. The following steps are recommended to evaluate this further.

1. Clean the filter influent channel and stop polymer addition
    a. Stop polymer addition
    b. Sample the filter influent for total coliforms and turbidity daily
    c. Overtime determine if there is a rising concentration in total coliform, turbidity, and visual solids deposition
2. Depending on results of #1 - Clean the filter influent channel again
    a. Start polymer addition
    b. Sample the filter influent for total coliforms and turbidity daily
    c. Overtime determine if there is a rising concentration in total coliform, turbidity, and visual solids deposition
3. Depending on results of #2 - Clean the filter influent channel again
    a. Stop polymer addition
    b. Start sodium hypochlorite addition (continuous addition ~2 mg/L)
    c. Sample the filter influent for total coliforms and turbidity daily
    d. Overtime determine if there is a rising concentration in total coliform, turbidity, and visual solids deposition.

UV Disinfection Evaluation

The plant has a Fischer & Porter 70UV6000 UV system consisting of one channel with five banks. The system was designed for a UV dose of 100 mJ/cm^2 at a UVT of 55 percent. Table 5 contains the design information for the UV system. Note that a dose of 100 mJ/cm^2 is typically used for facilities that produce Title 22 recycled water. The WWTP does not currently produce recycled water. Further note that the UV system dose of 100 mJ/cm^2 is a predicted dose, and is likely not accurate or conservative as this UV system has not been "Validated" in accordance with the NWRI UV Guidelines. More likely than not, the UV dose shown on the HMI over predicts dose (and is not conservative or accurate).

Table 5 Original UV System Design Parameters

Design Parameter (1)	Value
Peak Flow Rate, mgd	4.3
Average Daily Flow Rate, mgd	3.0
Design UV Transmittance, %	55
Minimum UV Dose, mJ/cm^2	100
Number of Channels	1
Number of Banks per Channel	5
Number of Lamps per Bank	176
Total Number of Lamps	880
Power Consumption per Lamp, W	79

Notes:
(1) Parameters taken from the 1994 Design Criteria Drawing G-4.

The UV system has historically operated in Auto mode with additional banks placed on-line in Hand position if there is trouble meeting daily total coliform requirements. Generally five banks are on-line at flows above 3.5 mgd. The system dose setpoint is 170 mJ/cm^2. Additional banks are brought on-line in response to changes in UVT and flow. The low UVT setpoint has historically been set to 65 percent.

UV System Upgrades and Retrofit

Ironbrook UV (Ironbrook) performed system components and electronics upgrades from May 2012 to January 2013. Upgrades included:

- Lamp racks refurbishment/rewire
- Circuit boards
- Ballasts
- Internal cables
- Connectors
- Breakers
- Intensity meters replaced with new digital meters
- Intensity probes
- Fan thermostats
- Main incoming breakers
- PDC transformers
- Lamps and sleeves

About a year after the upgrades and retrofits the plant experienced problems with disinfection performance. During these times the WWTP could not meet performance with UV alone so the UV effluent was disinfected further with chlorine and then dechlorinated.

In August 2017, Ironbrook coordinated lamp output testing with the lamp manufacturer, Light Sources Inc. (LSI). Five lamps each from Bank numbers 2 and 3 were sent to LSI for output checks. The lamps from Bank 2 were 9,500 hours and Bank 3 was 10,800 hours. LSI found a depreciation in output between 3 to 13 percent. Carollo contacted Ironbrook and spoke with Jamie Collins in early February 2018. Ironbrook visited the site in October 2017 and checked the following items and confirmed everything was working properly.

- PDC transformers
- Ballast power
- Quartz sleeve fouling
- Channel cleaning
- Water level

Ironbrook measured the power consumption of all of the lamp racks when operating and all measurement were within normal operating range. In January 2018 a PDC transformer failed and was replaced.

UV Transmittance Meter

Figure 8 presents a comparison between the on-line UVT analyzer and the calibrated benchtop UVT meter used during the testing. The average difference between the online meter and the benchtop meter was 12.8 percent with a range of 11.8 to 13.9 percent, with the online analyzer overestimating UVT and thus resulting in underestimating UV dose. The on-line UVT analyzer should be calibrated and checked routinely, maintaining values within a few percent of a calibrated bench-top unit.

UV dose is calculated based on the system flow and online UVT value. If the UVT value is overestimated then the calculated dose is artificially high which causes the control system to turn off bank(s) of lamps to attain the setpoint dose of 170 mJ/cm^2. The actual dose will be lower than the setpoint dose thereby underestimating the UV dose and not providing adequate disinfection.

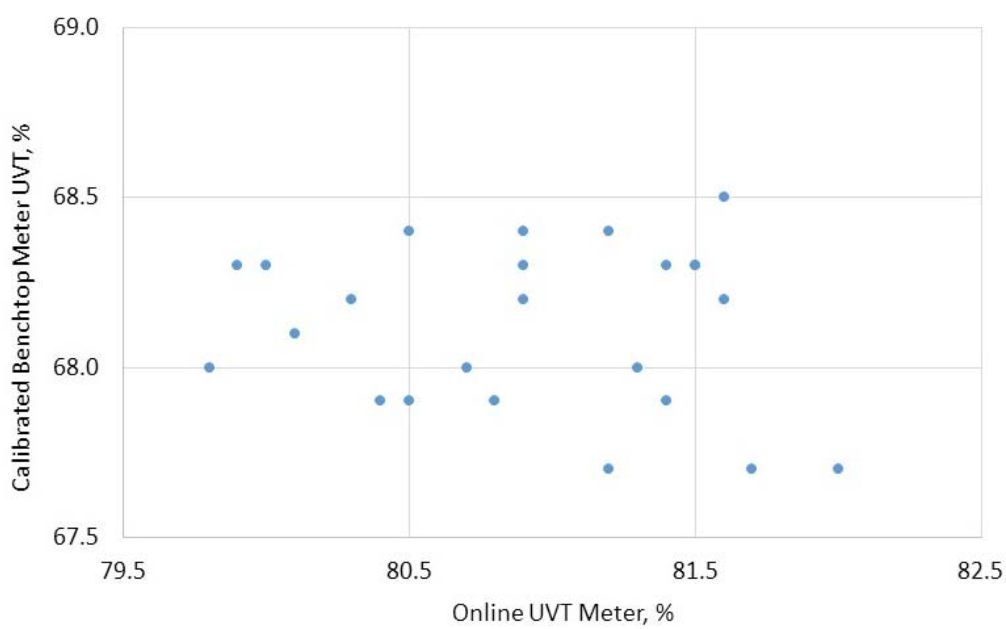


Figure 8 Comparison of Online UVT Meter to Benchtop UVT Meter

UV System Testing

Hypothesis

The data above indicates that for the dates measured, the filter effluent quality is sufficiently high to allow effective UV disinfection. Thus, the UV system should perform effectively if properly designed, operated, and sampled. The analysis below examines common shortfalls in UV performance.

It is noted here that OVSD staff already directly addresses several potential methods for UV compliance failures, these items along with how they are handled by OVSD, are:

- Compliance Sampling. OVSD currently performs all sampling after Bank E, upstream of the level control gate.
- UV Channel Cleaning. Under normal operations OVSD cleans one UV bank per week. Lamp sleeves are cleaned by hand. OVSD currently cleans the UV channel using a scouring wand once a week and periodically the channel is taken off line and power washed if needed due to algae or bryozoan growth.
- Hydraulic Design. The filter effluent enters the UV channel from the channel floor via a 24-inch pipe. The water flows through a baffle upstream of the first bank of lamps and the approach hydraulics appear fine. We did not measure the velocity profile upstream of the first bank to confirm the approach hydraulics. The normalized lamp velocity through the UV system is low at 17.0 gpm/lamp which represents a maximum velocity of 0.60 feet per second. The water level is controlled by a counter-balanced level control gate located at the end of the channel.

Data Collection

Full Scale UV Dose Response

Testing of the UV system to better estimate capacity and evaluate operation was performed on May 14, 15, and 17, 2018. During the testing the flow and the banks on was adjusted. Table 6 provides a summary of the testing conditions, noting that the "HMI UV Dose" is a calculated dose and should be viewed with skepticism, as discussed previously. Each row represents one test. Samples were collected from the UV influent and after the last bank (the current compliance sampling point) during each test. Samples were analyzed for total coliforms to determine log removal.

Table 6 contains the average UV intensity of the operational banks, whereas Table 7 contains the individual UV intensity readings. There was a 4 to 8 percent difference between the UV intensity sensors.

Table 6 UV Testing May 14, 15, and 17, 2018

Test ID	Flow, mgd	UVT Online Meter, %	UVT Bench-top Meter, %	Turbidity, NTU	TSS, mg/L	Temp, ^0C	Banks On	Average UV Intensity	HMI UV Dose
11	1.48	81.6	67.7	0.4	1.00	21.2	A, B	97.8	551
12	2.86	81.4	68.0	0.5	1.00	21.2	A, B	97.8	290
13	2.78	80.5	67.9	0.5	1.00	21.2	A, B	97.8	298
14	2.69	80.3	68.0	0.5	1.00	21.2	A, B	97.8	304
21	2.20	81.0	68.2	0.5	0.89	21.6	B, C	92.2	363
22	2.85	81.5	68.3	0.5	0.89	21.6	B, C	92.1	274
23	2.86	80.6	68.2	0.5	0.89	21.6	B, C	92.1	286
24	2.79	81.2	68.4	0.5	0.89	21.6	B, C	92.2	281
31	3.50	81.1	68.3	0.4	1.28	21.6	C, D	94.7	239
32	3.57	80.9	68.0	0.4	1.28	21.6	C, D	94.5	219
33	3.53	80.0	68.2	0.6	1.28	21.6	C, D	94.4	222
34	1.52	81.3	68.4	0.4	1.28	21.6	C, D	97.8	555

Table 7 UV Intensity Sensor Variation

Test ID	Banks On	Bank A UV Intensity	Bank B UV Intensity	Bank C UV Intensity	Bank D UV Intensity	Percent Difference Between Sensors	Average UV Intensity
11	A, B	101.7	93.9			8	97.8
12	A, B	101.7	93.9			8	97.8
13	A, B	101.7	93.9			8	97.8
14	A, B	101.7	93.9			8	97.8
21	B, C		93.8	90.5		4	92.2
22	B, C		93.8	90.5		4	92.1
23	B, C		93.8	90.4		4	92.1
24	B, C		93.9	90.4		4	92.2
31	C, D			91.9	97.4	6	94.7
32	C, D			91.9	97.2	5	94.5
33	C, D			91.9	96.9	5	94.4
34	C, D			91.8	97.3	6	97.8

Additional filter effluent samples were collected for CB analysis to determine the UV dose response of the indigenous total coliforms. Figure 9 contains the CB results for the filter effluent collected on May 14 and 15, 2018.

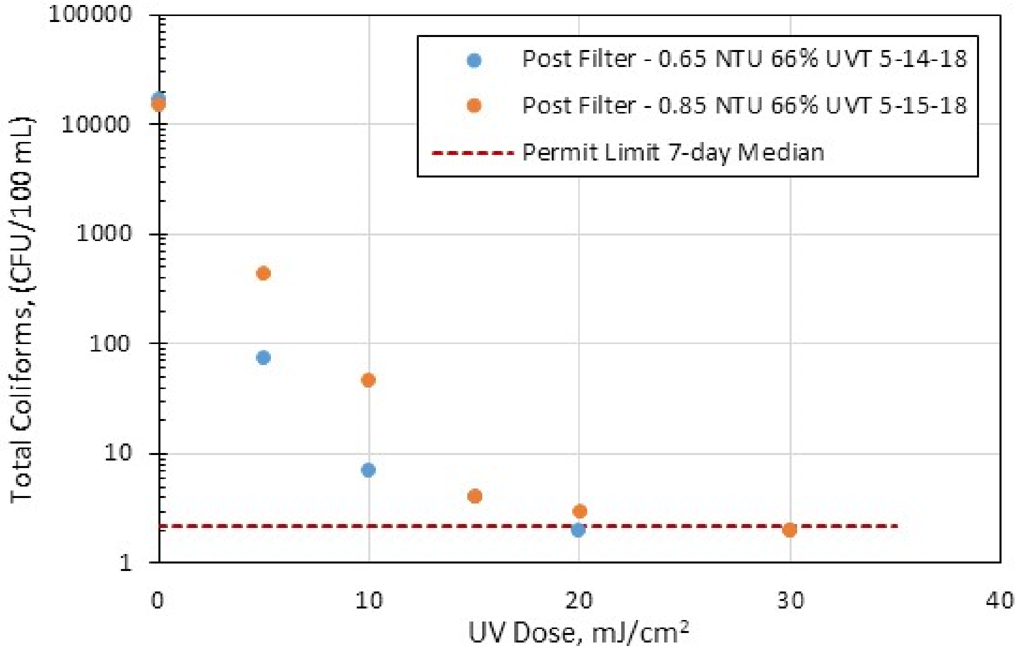


Figure 9 Filter Effluent Total Coliform Collimated Beam Analysis

Power

Measurements were taken during the UV stress testing to evaluate the power consumption and confirm it was not lower than the design power consumption after the retrofit. The Fischer & Porter UV system had a design power consumption of ~79 Watts (W) per lamp. The plant did not have a power meter available for the measurements so we estimated the UV system power factor to calculate the power consumption. The estimated power consumption of the lamps during the testing ranged from 69 to 77 W per lamp. This is an acceptable difference.

Quartz Sleeve Fouling

Fouling of the quartz sleeves housing the ultraviolet (UV) lamps will reduce the UV intensity and thereby dose delivered by the reactor. Fouling can be mitigated through cleaning practices. UV systems are designed to deliver the required dose under fouled conditions.

Sleeve fouling can vary significantly from site to site so it is important to understand the fouling characteristics of the WWTP's effluent so that an effective cleaning frequency can be determined. To evaluate quartz sleeve fouling a single UV Bank was not cleaned for 30 days of operation. Six fouled quartz sleeves and one new sleeve were shipped to the Carollo Validation Facility in Portland, OR for analysis.

The Carollo Optics Bench was used to determine fouling by measuring UV intensity at seven positions along the length of the quartz sleeve from the open end to the domed end. The Optics Bench measures the intensity of a fixed UV lamp through the sleeve at fixed positions. The positions are fixed so that they are the same on each sleeve when measured. The sleeve fouling factor (SFF) is calculated by normalizing the readings to the reference sleeve (new sleeve). It is reported as a percent. The reference sleeve (new sleeve) is set at 100 percent. The method measurement uncertainty is two percent.

Figure 10 contains the results of the six fouled sleeves and the new sleeve (clean). The seven positions measured are shown on the x-axis and the percent SFF is shown on the y-axis. It was noted that the fouling looked like calcium deposits (white powder) rather than iron fouling. All the sleeves had roughly 70 - 80 percent fouling.

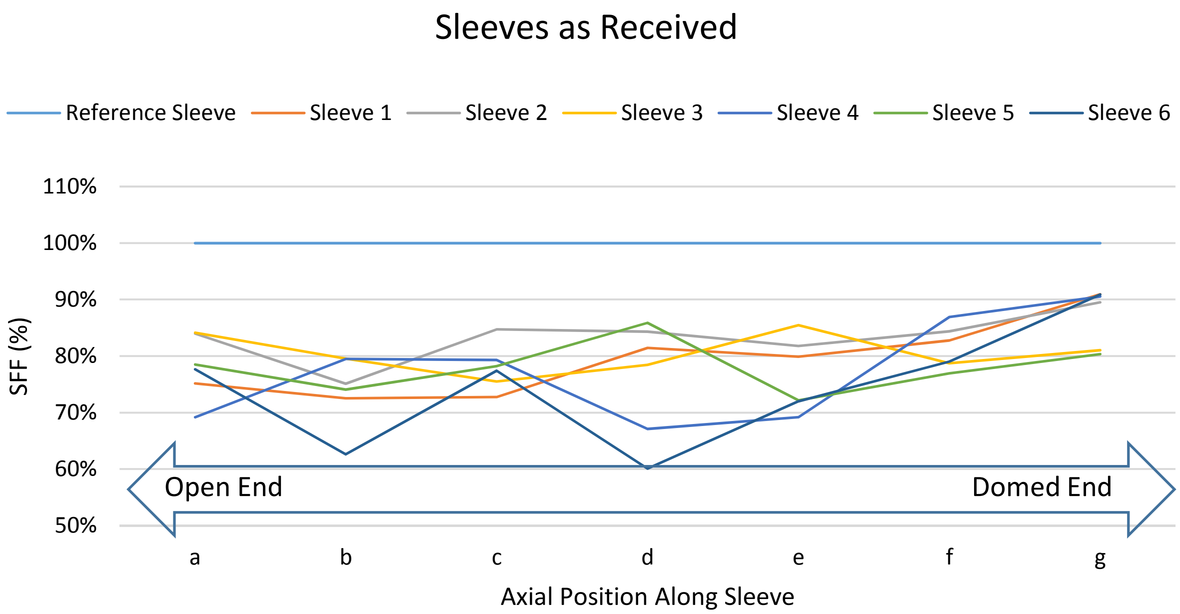


Figure 10 Fouling Measurements of Quartz Sleeves Operated for 30 days

The outside of the six fouled quartz sleeves were then chemically cleaned after the measurements in Figure 10 were taken. The sleeves were cleaned by hand with Lime Away and a Scotch Brite pad. The relative UV intensity of each cleaned sleeve (outside of sleeve cleaned) at the seven positions along the sleeve is presented in Figure 11.

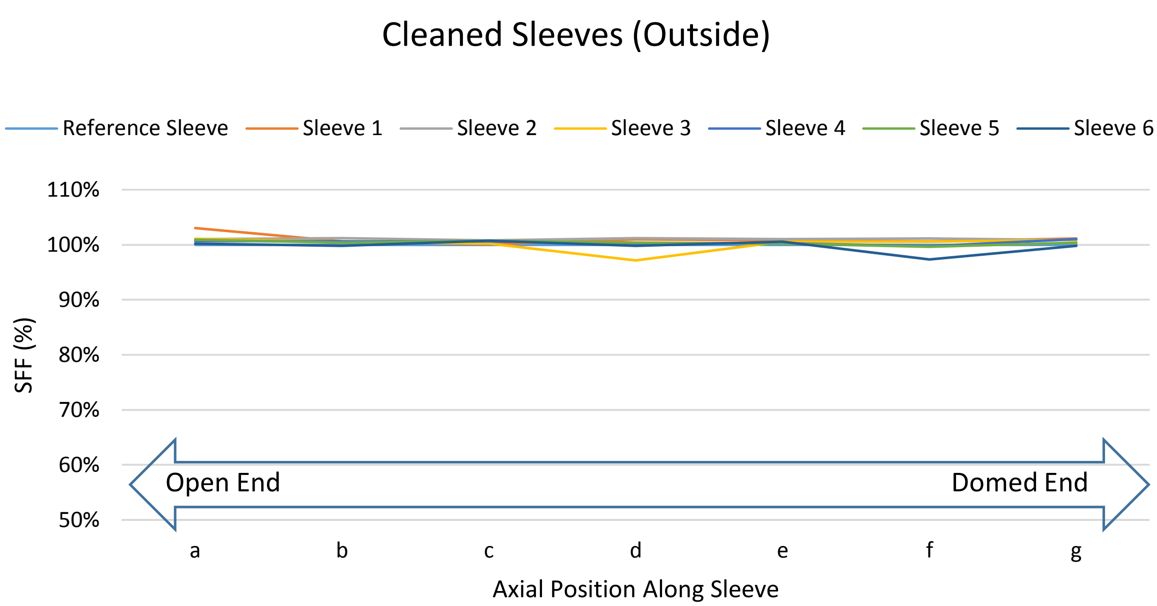


Figure 11 Fouling Measurements of Quartz Sleeves Operated for 30 days - Outside of Sleeve Cleaned

Analysis and Conclusions

Full Scale UV Dose Response

The analysis in this subsection applies directly to the results from testing the UV system under controlled conditions.

The CB results from Figure 9 are similar to the single day CB results that showed a UV dose of 30 mJ/cm^2 or greater is sufficient to meet the 2.2 MPN/100 mL geometric mean target. The collimated beam data was analyzed to develop a regression equation to estimate UV dose based on the log removal of total coliforms during each test. The data from Figure 9 is replotted in Figure 12, presenting the UV dose as a function of total coliform log removal. Table 8 contains the calculated total coliform UV dose for each test based on the CB results. Note that the calculated log removal and thereby dose is dependent on the influent concentration of total coliforms which limits the highest log removal and dose that can be achieved.

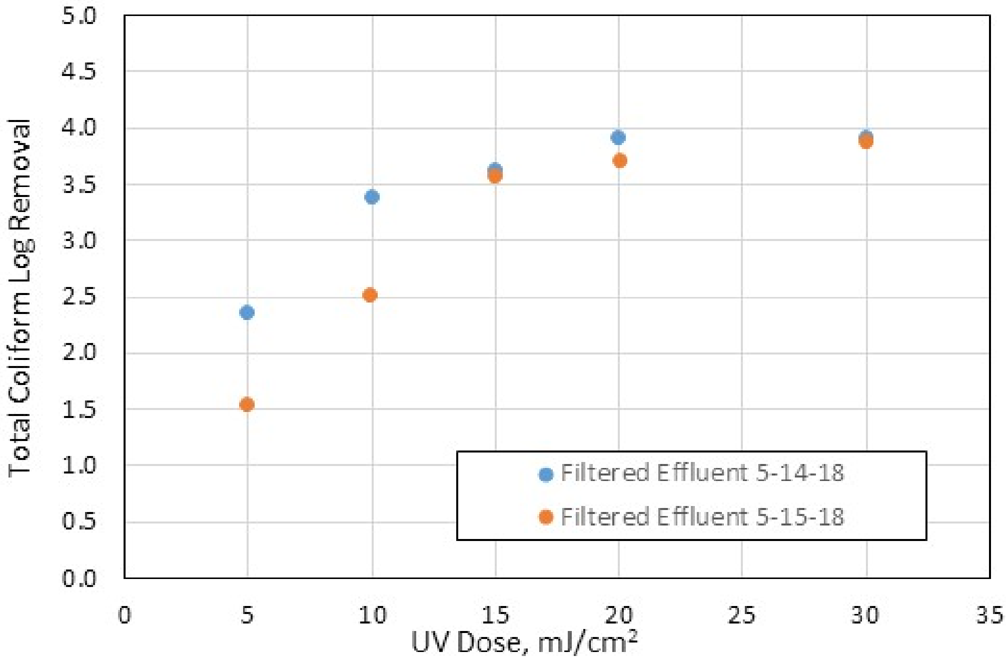


Figure 12 Filter Effluent Collimated Beam Analysis Total Coliform Log Removal vs Dose

Table 8 Estimated UV Dose - Testing May 14, 15, and 17, 2018

Test ID	UVT Bench-top Meter, %	Flow mgd	Banks On	Influent Total Coliforms, MPN/100 mL	Effluent Total Coliforms, MPN/100 mL	Log Removal of Total Coliforms	Calculated Total Coliform UV Dose, mJ/cm^2	HMI UV Dose
11	67.7	1.48	A, B	1600	13.0	2.09	4	551
12	68.0	2.86	A, B	1600	<1.8	2.95	>8	290
13	67.9	2.78	A, B	1600	2.0	2.90	8	298
14	68.0	2.69	A, B	1600	6.8	2.37	5	304
21	68.2	2.20	B, C	1700	2.0	2.93	8	363
22	68.3	2.85	B, C	2200	<1.8	3.09	>10	274
23	68.2	2.86	B, C	2200	2.0	3.04	9	286
24	68.4	2.79	B, C	5400	7.8	2.84	7	281
31	68.3	3.50	C, D	2400	<1.8	3.12	>10	239
32	68.0	3.57	C, D	3500	<1.8	3.29	>12	219
33	68.2	3.53	C, D	5400	<1.8	3.48	>14	222
34	68.4	1.52	C, D	16000	<1.8	3.95	>23	555

The data in Table 8 suggests:

- With two banks in operation at UVT values of 67 to 68 percent, total coliform compliance was met 75 percent of the time, similar to historical compliance results.
- There are contradictory results. For example, a test at 1.48 mgd resulted in 13 MPN/100 mL but a test at 2.86 mgd, under nearly identical operational conditions, resulted in <1.8 MPN/100 mL.

The next step is to understand what UV dose the existing UV system can deliver. The proper method to make this determination is to utilize independent third party "validations" of equipment; however, this does not exist for the Fischer & Porter system.

Using Fischer & Porter data and information for a 70UV6000 system the MS2 Reduction Equivalent Dose (RED) for each test was estimated. The following design factors were used with the Fischer & Porter data:

- End of Lamp Life Factor (EOLL): 0.80 based on lamp hours ranging from 10,282 to 14,198 hours during testing.
- Fouling Factor (FF): 0.90 based on lamps being cleaned within a couple weeks of the testing.

It is important to note that the MS2 RED does not factor in the effect of water level due to headloss. The design water level is assumed to be 24-inches. Therefore, when the water levels are greater than 24-inches the actual UV dose will be lower than estimated.

Table 9 contains the water level measurements taken during the testing with the estimated MS2 RED and whether the test passed or failed to meet permit requirements (2.2 MPN/100 mL). Test 11 would be expected to meet permit since it resulted in a higher estimated dose at lower water levels. These results could be due to a lab error in measurement of UV effluent total coliforms. The resulting performance of Tests 14 and 24 can be attributed to marginal UV dose and higher water level.

Table 9 UV System Performance and Water Level Analysis - Testing May 14, 15, and 17, 2018

Test ID	UVT Bench - top Meter, %	Flow mgd	Banks On	Water Level, inches(1)	Headloss Across Banks, inches(2)	Water Level Between Banks, inches	Height Above Design Water Level, inches	Estimated MS2 UV Dose(3) (4), mJ/cm^2	Sample Met Permit, Pass/Fail
11	67.7	1.48	A, B	24.24	0.28	24.52	0.52	100	FAIL
12	68.0	2.86	A, B	24.78	0.28	25.06	1.06	56	PASS
13	67.9	2.78	A, B	24.66	0.28	24.94	0.94	57	PASS
14	68.0	2.69	A, B	24.60	0.28	24.88	0.88	60	FAIL
21	68.2	2.20	B, C	24.36	0.21	24.57	0.57	72	PASS
22	68.3	2.85	B, C	24.60	0.21	24.81	0.81	57	PASS
23	68.2	2.86	B, C	24.60	0.21	24.81	0.81	57	PASS
24	68.4	2.79	B, C	24.60	0.21	24.81	0.81	58	FAIL
31	68.3	3.50	C, D	24.60	0.20	24.80	0.80	48	PASS
32	68.0	3.57	C, D	24.54	0.20	24.74	0.74	46	PASS
33	68.2	3.53	C, D	24.60	0.20	24.80	0.80	47	PASS
34	68.4	1.52	C, D	24.18	0.14	24.32	0.32	100	PASS

Notes:
(1) Converted to inches from HMI water level reported in feet.
(2) Headloss is based on validated headloss values. It is calculated based on the number of banks downstream of the operating banks plus one.
(3) Estimated MS2 Dose for the F&P 70UV6000 system is based on F&P data, not third party validated. Lamp hours ranged from 10,282 to 14,198 hours; therefore, used an End of Lamp Life (EOLL) factor of 0.80 for the dose calculation. Since lamps were cleaned within the last couple of weeks prior to testing, used a fouling factor (FF) of 0.90 for the dose calculation.
(4) Estimated MS2 Dose does not factor in the effect of water level due to headloss. Assumes water level is at the design level of 24.0 inches. Therefore, for water levels above 24.0 inches the MS2 Dose will be lower than estimated.

The data in Table 9 suggests:

- Water level impacted performance. As you test downstream banks the system complies with the permit limit even though the estimated dose is lower due to the higher flows.

To reliably meet the permit criteria (2.2 MPN/100 mL total coliform), there are a few recommended approaches:

- Water Level. The water level needs to be lowered so that at high flows the water level does not exceed 2.00 feet. Presently, at low flows the water level is 2.00 feet and at high flows the water level is 2.06 feet. It is recommended that the level gate be tuned so that the water level at low flows is 1.93 feet. This is sufficient to cover the top lamp of the rack. This will also increase the efficiency of each UV bank since the water level will be reduced by approximately 0.8 inches. Operation staff indicated that when they what to ensure proper disinfection they turn on UV Bank E, the last bank. The reason this bank is the most efficient is because it has the lower surface water level. Reducing the water level in the channel will increase the disinfection efficiency of each bank.
- Accurate dose-based control. The UV dose to maintain total coliform counts at or below the 2.2 MPN/100 mL target requires a minimum UV dose of 30 mJ/cm^2. As a necessary measure of conservatism, a minimum UV dose of 40-50 mJ/cm^2 is recommended. However, this dose is only

relevant if the UV system utilizes a validated monitoring and control system with a high degree of accuracy. Because the existing UV system has not been validated in accordance with industry standards (NWRI), because the control system does not utilize accurate sensor (UV intensity) or UVT data for real time control, this dose target of 40-50 mJ/cm^2 is not relevant to OVSD. Based on the estimated MS2 RED analysis a minimum MS2 RED of approximately 60 mJ/cm^2 is reasonable to meet permit when combined with lowering the water level.

- Flow Pacing Control. The data presented above indicates that two UV banks, fully operational with lamps near their end of lamp life and with cleaned quartz sleeves, are sufficient for compliance ~75 percent of the time under the flow test conditions. As quartz sleeves will foul and because there will be occasions where lamps will be out in a bank (see note on this below), the UV system must operate with two or more banks in service. Based upon the data in hand, the recommended bank operational strategy is shown in Table 10.

Table 10 Recommended Flow Pacing

Flow Ranges at UVT(1)	Number of Operating Banks(2)
65 percent ≤ UVT < 70 percent	
≤ 1.8 mgd	2
>1.8 mgd to ≤2.8 mgd	3
>2.8 mgd to ≤3.8 mgd	4
>3.8 mgd	5
UVT ≥ 70 percent	
≤ 2.1 mgd	2
>2.1 mgd to ≤3.3 mgd	3
>3.3 mgd	4

Notes:
(1) UVT is based on the portable UVT meter reading until the online UVT analyzer is recalibrated so that it matches the portable meter.
(2) For any combination of 2 and more operating banks, Bank D or Bank E must be operating. This will ensure that one of the two most effective banks is in use due to the lower water level.

For the recommended flow pacing approach, success will be dictated by careful tracking of the following parameters:

- UV influent total coliform concentrations. As influent total coliform concentrations rise, OVSD staff should consider increasing the number of banks in operation proactively. Concurrent with this increase in UV dose, methods to reduce the influent total coliform concentration should be implemented, such as shock chlorination or cleaning of the filter feed channel.
- UV Transmittance. The data collected here illustrated the inaccuracy of the online UVT analyzer by more than 10 percent. The data also illustrated a UVT of 67 to 68 percent with a calibrated bench-top meter. The online meter should be routinely checked and calibrated as discussed herein.
- Quartz Sleeve Cleaning. As recommended previously, the quartz sleeves for operational banks should be chemically cleaned, at a minimum, every 3 weeks.
- UV Channel Cleaning. Under normal operations OVSD cleans one UV bank per week. OVSD currently cleans the UV channel using a scouring wand once a week and periodically the channel is taken off line and power washed if needed due to algae or bryozoan growth.

- Lamp Replacement. Maintaining 100 percent UV lamps in operation for each bank should be the maintenance goal. There will be occasions where lamps are out of service. At a minimum, and in accordance with the NWRI UV Guidelines, avoid:
  - Adjacent lamps out of service in a single bank
  - >5 percent of lamps out in a single operational bank
- UV Intensity. The UV intensity results should be trended and inspected weekly for differences between probe readings (from bank to bank), with a goal of maintaining values between probes within ~10 percent. Further, should UV intensity values trend down, OVSD staff should evaluate the cause, which may include reduced UV lamp output, fouled sensor tubes, dropping UVT, or other factors, all of which may require more UV banks to go into service or parts replacements (e.g., UV lamp replacement).
- Filter Effluent Turbidity. While filter effluent turbidity did not correlate with UV disinfection compliance, rising filter effluent turbidity should be closely monitored. Weekly analysis of turbidity trends is recommended.

Quartz Sleeve Fouling

The conclusions regarding sleeve fouling and the impact on disinfection performance are:

- External fouling of the sleeves was reasonable, losing ~20 to 30 percent of UV transmittance through the sleeves over a one month period.
- As shown in Figure 11, all six sleeves were brought back close to "new sleeve" SFF levels (SSF of approximately 100 percent) after cleaning the outside of the sleeves.
- Our suggestion is to have the quartz sleeves from all operational UV banks chemically cleaned every three weeks to maintain a sleeve fouling factor of 0.80 (80 percent).

Appendix A

SUMMARY OF INVESTIGATIONS

Summary of Investigation

Technology	Potential Issue	Investigation	Conclusion	Issue Resolved?
Filtration	• Filter Bypass, resulting in elevated TSS, turbidity, and particle loading to the UV system	• Examination of filter influent and effluent quality as well as filtration operational parameters	• Filter adequately reduces particles and turbidity and preconditions the water properly for subsequent UV disinfection	• Yes • For the dates tested, the filter is operated and performs as necessary for effective UV disinfection
Filtration	• High filtered effluent total coliform concentrations	• Periodic shock chlorination to reduce filter effluent total coliform	• Results are inconclusive • Recommend repeating efforts and monitoring filter influent and effluent total coliform concentrations	• No • Uncertain if shock chlorination will solve periodic high filter effluent total coliform concentrations
Filtration	• High filtered effluent total coliform concentrations	• Recommended Efforts (not implemented) • Investigate the chemical feed station upstream of filtration • Determine the extent of coagulation and sedimentation of solids ahead of filtration and the resulting buildup of total coliform concentrations	• No work completed on this recommended task	• No
UV Disinfection	• Biofilms, resulting in sloughing that shields UV light and causes regrowth at sample compliance locations	• No Investigation, issue well handled by OVSD staff	• OVSD staff samples after Bank E for compliance sampling. • Under normal operations OVSD cleans one UV bank per week. Lamp sleeves are cleaned by hand. OVSD currently cleans the UV channel using a scouring wand once a week and periodically the channel is taken off line and power washed if needed due to algae or bryozoan growth.	• Yes • Channel cleaning interval and approach is acceptable • Sample location is acceptable
UV Disinfection	• UV lamp outages, if sufficiently numerous, will reduce UV system effectiveness	• No Investigation, issue well handled by OVSD staff	• OVSD staff works to maintain all UV lamps in service for operational UV banks • For future maintenance of UV lamps, at a minimum, do not operate a bank with adjacent lamps out of service and do not operate a bank with more than 5 percent of lamps out of service	• Yes • Lamp maintenance sufficient
UV Disinfection	• Analytical methods impact accuracy of measurements	• Comparison of Colilert with Standard Methods for total coliform analysis	• Colilert readings typically are higher than Standard Methods for total coliform analysis • Use Colilert for internal investigations only, recognizing the inherent conservative bias of this method	• Yes • Colilert is used for internal investigations only
UV Disinfection	• Fouled quartz sleeves results in reduced UV intensity for disinfection	• Extended period sleeve fouling study to demonstrate the rate and impact of sleeve fouling	• Chemical cleaning of quartz sleeves every 3 weeks for operational banks will minimize sleeve fouling impacts to 20 percent or less	• Yes • Chemical cleaning interval has been determined and will minimize the impact of quartz sleeve fouling
UV Disinfection	• Inaccurate UV System control	• The installed UV system has not been extensively evaluated per industry standards	• The control system is based upon a Point Source Summation method, incorporating flow and UVT into dose calculation • The dose shown on the UV HMI is grossly overestimating actual UV dose • Reliance by OVSD on the HMI UV dose numbers, which are well in excess of the target UV dose, is misleading to staff	• No • Extensive validation testing of the installed UV system would be required, resulting in an approximate cost of $50,000. Once validation testing is completed, reprogramming costs would be in the $20,000 range.
UV Disinfection	• UVT analyzer (1 unit) out of calibration, contributing to inaccurate UV System Control	• Online UVT analyzer compared to calibrated bench-top UVT meter	• Online UVT analyzer overestimates UVT by >10 percent, resulting in over prediction of UV dose by the UV system, and misleading staff	• Yes • Routine calibration of an online UVT analyzer can be readily done

Technology	Potential Issue	Investigation	Conclusion	Issue Resolved?
UV Disinfection	• UV intensity meters (4 tested, 5 in total) out of calibration	• Four different UV intensity meters were compared with each other for uniformity	• Some variability was seen, ~10 percent or less. While not ideal, that variability is acceptable	• **Yes** • UV intensity meters data should be trended and cross-checked on a weekly basis for uniformity
UV Disinfection	• Excessive head or increased water level can reduce UV disinfection efficiency	• Water levels were evaluated over a range of flow and compared with the DESIGN value of 24-inches for this particular UV system design	• The water level is set to 24-inches at the effluent of the UV system • Upstream of the last bank, step by step, water level increases for each upstream UV bank • For high flow, upstream banks see water level of greater than 24.8-inches	• **No, issue understood, but not solved.** • The water level needs to be set to ~23-inches after the last UV bank, which allows for head to increase on upstream banks without exceeding the design target of 24-inches upstream of the level control gate.
UV Disinfection	• Non-uniform or insufficient power draw to the UV banks results in reduced UV disinfection performance	• Power draw was estimated to the system and compared with expected values	• Power draw is reasonably close to the anticipated power draw	• **Yes** • UV lamps are drawing the appropriate power
UV Disinfection	• Reduced UV lamp output, often due to lamp aging, correlates directly to reduced UV disinfection performance	• This work did not examine UV intensity values with time	• This work did document the current UV intensity values • Long-term trending of UV intensity values, coupled with UVT at the time of measurement, allows for evaluation of loss of lamp intensity as lamps age	• **Partially** • Long term UV intensity monitoring will determine the impact of aging lamps on performance

Appendix B

RECOMMENDED WORK EFFORTS TO IMPROVE DISINFECTION RELIABILITY

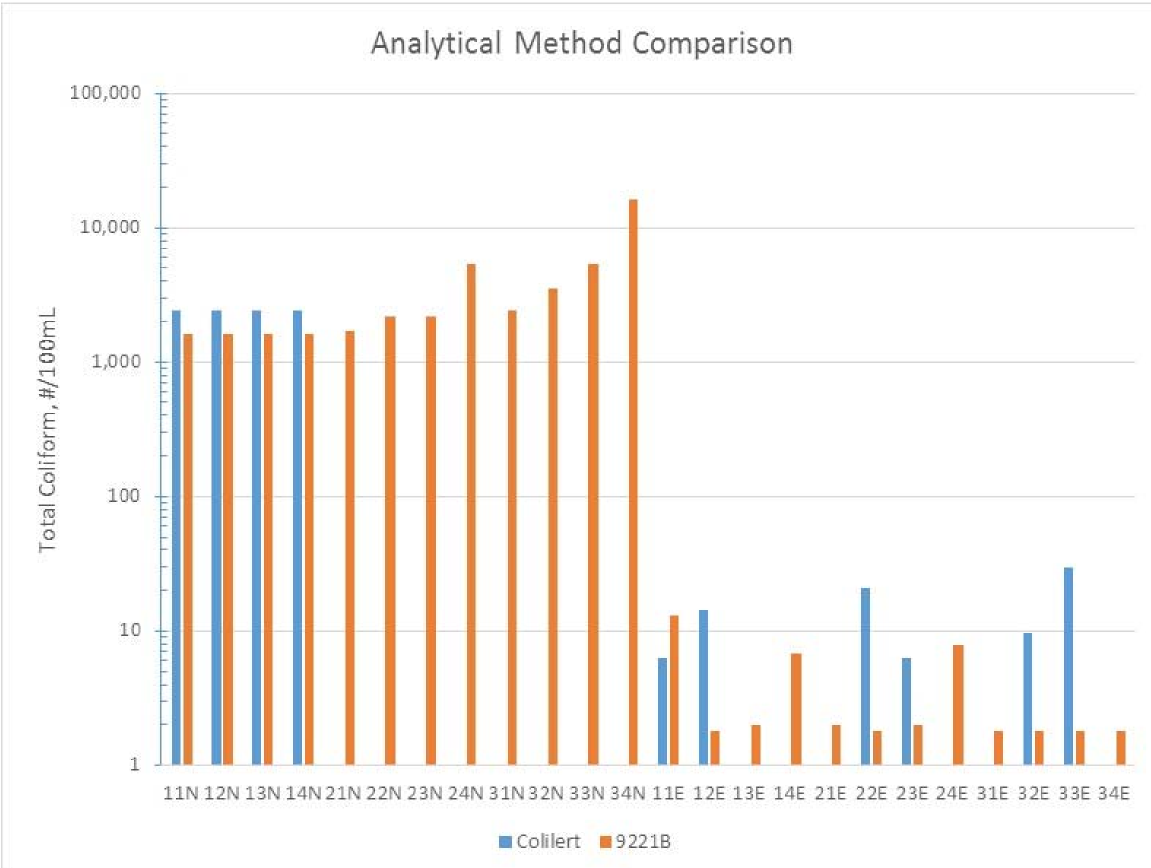
Recommended Work Efforts to Improve Disinfection Reliability

Concept	Relevant to	Issue	Approach	Potential Response	Cost or Level of Effort to Implement	Notes
Improved Water Quality Monitoring	Filtration	• Within the existing filter effluent turbidity range, filter effluent turbidity does not correlate to water quality challenges with UV • A better method to track UV feed water quality changes is needed	• Install particle counting on filter effluent to track increase in particles that can correlate to high total coliform loading	• Chlorinate filters or increase UV dose	• Already implemented	Work implemented, results TBD
Understand Impact of Filter Water Quality Upset	Filtration	• Filter upsets may send higher particle counts and higher total coliform concentrations to UV disinfection	• Collect samples for collimated beam UV testing and particle size distribution testing during a filter upset in the future.	• TBD, pending test results	• Staff time sampling • Laboratory cost	Coolers and bottles have been provided to conduct this testing in the future
Reduce Total Coliform Concentrations in Feed to UV	Filtration	• Solids with high total coliform concentrations may be building up in the feed channel to the filters	• Evaluate solids buildup and cleaning impact on filter influent and effluent total coliform concentration • Recommend test approach detailed in this report	• TBD, pending test results	• Staff time for cleaning feed channel • Staff time for total coliform analysis	
Accurate UV Dose Monitoring (Item #1)	UV Disinfection	• Online UVT analyzers historically drift and are inaccurate • Bench-top UVT meters are readily calibrated and can be relied upon to accurately measure UVT	• Purchase a bench-scale UVT meter to inspect water quality and verify online meter accuracy	• Recalibrate online UVT analyzer when values drift by >2 percent • Potentially adjust UV dose based upon drop in UVT	• Daily grab samples for analysis, assume 15 mins per day • Weekly UVT trending to examine changes in water quality and impact on UV performance, ~30 min per week	OVSD has a bench-top UVT meter manufactured by HF Scientific
Accurate UV Dose Monitoring (Item #2)	UV Disinfection	• The online UVT analyzer, per existing data analysis, is dramatically out of calibration and results in over prediction of UV dose	• Calibrate online UVT analyzer when drift is >2 percent compared to calibrated bench-top unit	• Calibrate UVT analyzer, which is done off-site • Rely upon bench-top device while calibration is underway • Consider a standby online UVT analyzer	• Calibration costs vary by manufacturer-TBD • Purchase of a standby online UVT meter will run between $12,000 and $25,000.	Carollo can advise on online UVT analyzers to be considered
Accurate UV Dose Monitoring (Item #3)	UV Disinfection	• Without improved monitoring and accurate control, the UV system should be operated in a flow-paced setting with number of operational banks a function of flow	• Hand Control System	• Improved compliance due to increased UV dose delivery	• Increased staff time to adjust UV bank operation based upon flow setpoints	
Accurate UV Dose Monitoring (Item #4)	UV Disinfection	• Control system for your UV system is simplistic and over estimates UV dose. HMI value is not accurate • A properly "Validated" UV system measures, reports, and control based upon a precise dose control system, accurate within 5 percent	• Perform extensive bioassay testing of the UV system in accordance with 2012 NWRI UV Guidelines • Reprogram the UV system control to closely follow the new Validation	N/A	• Bioassay will run ~$50,000 to $60,000 • Programming will run ~$20,000 to $25,000	Carollo can perform the bioassay work if requested

Concept	Relevant to	Issue	Approach	Potential Response	Cost or Level of Effort to Implement	Notes
Improve Hydraulic Conditions on UV System	UV Disinfection	• Water level set to 24-inches on most downstream UV bank, resulting in high water levels on upstream UV banks, and increases with higher flow • Water levels above 24-inches results in a bypass over the top of the UV bank, which will result in less reliable disinfection	• Water level needs to be lowered so that at high flows the water level does not exceed 2.00 feet. Recommend that the level gate be tuned so that the water level at low flows is 1.93 feet. This is sufficient to cover the top lamp of the rack. This will also increase the efficiency of each UV bank since the water level will be reduced by approximately 0.8 inches.	• Improved compliance due to elimination of flow bypass over the top of the UV banks	• This can be as simple as removing a ballast bar from the counter balance weights of the level control gate or the entire weights can be lowered on the moment arm towards the gate shaft. • Need to adjust the gate at low flows and then check the water levels at higher flows. • Estimated time to make the adjustments is a few hours for two operators.	

Appendix C

ANALYTICAL METHOD COMPARISON


Analytical Method Comparison
Total Coliform, #/100mL
100,000
10,000
1,000
100
10
1
11N 12N 13N 14N 21N 22N 23N 24N 31N 32N 33N 34N 11E 12E 13E 14E 21E 22E 23E 24E 31E 32E 33E 34E
Colilert
9221B