Occidental County Sanitation District WWTF Reclaimed Water Project – Alternatives Analysis – FINAL

Prepared for:
Sonoma County Water Agency

Prepared by:
Stantec Consulting Services Inc.

November 30, 2015

Sign-off Sheet

This document entitled Occidental County Sanitation District WWTF Reclaimed Water Project - Alternatives Analysis - FINAL was prepared by Stantec Consulting Services Inc. ("Stantec") for the account of Sonoma County Water Agency (the "Client"). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Prepared by _____________________________________
(signature)

Leila Sermek, PE

Prepared by _____________________________________
(signature)

Beth Cohen, PE

Prepared by _____________________________________
(signature)

Akram Botrous, PhD, PE, BCEE

Reviewed by _____________________________________
(signature)

Steven L. Beck, PE

Occidental County Sanitation District WWTF Reclaimed Water Project Alternatives Analysis - FINAL

Table of Contents

LIST OF TABLES

LIST OF FIGURES

List of Abbreviations

°C	degrees Celsius
AAF	average annual flow
AAL	average annual load
ADWF	average dry weather flow
BOD	biological oxygen demand
CCR	California Code of Regulations
CDO	Cease and Desist Order
CMDF	Cloth Media Disk Filter
CT	concentration time
DO	dissolved oxygen
ES	Executive Summary
gpd or gal/d	gallons per day
gpm	gallons per minute
HDPE	high density polypropylene
I/I	Inflow and Infiltration
LCRS	leachate collection and removal system
MBR	Membrane Bio Reactor
MCC	Motor Control Center
MG	million gallons
MGD or Mgal/d	million gallons per day

mg/L	million gallons per liter
mg N/L	milligram nitrogen per liter
MPN	most probable number
MS2	a type of bacterio-phage
NPDES	National Pollutant Discharge Elimination System
NWRO	National Water Research Institute
O&M	Operations and Maintenance
OCSD	Occidental County Sanitation District
PDF	peak day flow
PDL	peak day load
PHF	peak hour flow
PMF	peak month flow
PML	peak month load
PWF	peak week flow
PWL	peak week load
RAS	return activated sludge
RFP	Request for Proposal
SBR	Sequencing Batch Reactor
SCWA	Sonoma County Water Agency
SND	Simultaneous Nitrification and Denitrification
SRT	solids retention time
TKN	total Kjeldahl nitrogen
TSS	total suspended solids

US EPA	United States Environmental Protection Agency
UV	ultraviolet
VSS	volatile suspended solids
WAS	waste activated sludge
WDR	Waste Discharge Requirement
WWTF	Wastewater Treatment Facility

EXECUTIVE SUMMARY

ES.1 PURPOSE

The purpose of this executive summary is to summarize the investigations and recommendations developed in Sections 1 through 5 for the upgrades needed at the Occidental County Sanitation District (OCSD) Wastewater Treatment Facility (WWTF) to comply with California Title 22 standards, and meet stringent regulatory restrictions. This project will allow the plant to comply with the existing waste discharge permit at process flow rates corresponding to an average dry weather flow rate of 0.025 Mgal/d.

The following is a list of sections that are included in this Alternatives Analysis Report:

- Section 1: Flows and Loads
- Section 2: Site Layout and Yard Piping
- Section 3: Facility Improvements Alternatives Analysis
- Section 4: Disinfection System Improvements Alternatives Analysis
- Section 5: Solids Handling Alternative Analysis

ES.2 BACKGROUND

The town of Occidental is a small rural community, located in western Sonoma County, with a population of approximately 1,100 people. Because a large sector of the local economy is made up by tourism, commercial properties (retail, restaurants, and hotels) are a significant source of wastewater generation and utility use. The sanitary sewer collection system has approximately 100 existing connections, with the remaining residents on independent septic systems. The Occidental County Sanitation District (OCSD) collects wastewater through a series of gravity sewer pipelines and one centralized pump station on Occidental Camp Meeker Road. The pump station sends raw sewage to the OCSD Wastewater Treatment Facility (WWTF), located adjacent to the Druid's cemetery.

The existing OCSD WWTF was originally constructed in 1973 and consists of a two stage pond treatment system (one aeration pond and one settling pond) with chlorine disinfection, dechlorination, and disposal to a 10 million gallon (MG) storage reservoir on a nearby private property, called Graham's Pond. From the storage reservoir, disinfected secondary recycled water can be discharged to Dutch Bill Creek, a tributary to the Russian River, or to irrigate surrounding fodder crops. In 1995, responsibility for operating and maintaining the OCSD WWTF and collection system was transferred to the Sonoma County Water Agency (SCWA).

The WWTF operates under a waste discharge permit issued by the North Coast Regional Water Quality Control Board (order number R1-2012-0101) and has received a Cease and Desist Order (CDO) for violating various discharge requirements (R1-2012-0102). The permit requires completely eliminating discharges to Dutch Bill Creek from May 15th through September 30th, as well as imposes stringent effluent limitations that require significant upgrades in level of treatment. Complying with the new regulations is made difficult due to a number of conditions specific to the town of Occidental, including site constraints, limited rate payers, and current level of treatment. As a result, the plan has been to proceed in an orderly yet cautious manner to develop a project that (1) complies with current regulations, (2) is affordable to the existing ratepayers, and (3) provides service for minor development/infill within the town.

There have been several interim projects conducted between 2002 and 2007 that improved the treatment ponds (dredging and baffling) and collection system inflow and infiltration (I/I). Further, over the past decade, SCWA has conducted a number of investigations to determine the most viable strategy for compliance with the Regional Board's mandates. Those studies determined there were two potentially viable solutions for treating the town's wastewater: 1) improving the local WWTF and relocating the final effluent storage pond to Morelli Lane, or 2) trucking sewage to a nearby fully compliant wastewater plant that has the capacity to treat the additional regional sewage.

The purpose of this report is to analyze the first compliance alternative and to recommend the most appropriate process and site improvements necessary to comply with the permit and maintain a local WWTF.

ES.3 SUMMARY OF FINDINGS AND RECOMMENDED IMPROVEMENTS

The findings and recommended improvements for the OCSD WWTF are summarized in the following paragraphs:

Section 1: Flows and Loads

Influent flow data for the period from January 2000 through October 2014 were obtained and analyzed. Historical plant data from 2000 through 2014 were analyzed to establish appropriate design values for existing average and peak period influent flows and loads to the wastewater treatment plant. Incremental increases to the existing flows and loads were then calculated based on the anticipated buildout average dry weather flow (ADWF) rate of 0.025 Mgal/d. Existing and future design flows and loads are summarized in Table ES-1.

The WWTF influent data includes hourly flow rate information and monthly biochemical oxygen demand and total suspended solids (BOD and TSS) sampling results. Because the District does not monitor influent temperature and there are limited availability of Total Kjeldahl Nitrogen (TKN) samples, the historical influent database must be supplemented with typical data for similar communities and appropriate assumptions based on engineering judgment.

Table ES-1 Existing and Design Flows and Loads

Parameter	Unit	Project Design Criteria
Flow		
ADWF	Mgal/d	0.025
AAF	Mgal/d	0.038
PMF(1)	Mgal/d	0.083
PWF(1)	Mgal/d	0.115
PDF(1)	Mgal/d	0.183
PHF(2)	Mgal/d	0.229
BOD Loads		
AAL	lb/day	139
PML	lb/day	222
PDL	lb/day	278
TSS Loads		
AAL	lb/day	139
PML	lb/day	222
PDL	lb/day	278
TKN Loads		
AAL	lb/day	28
PML	lb/day	44
PDL	lb/day	56
BOD Concentration		
ADWF and AAL	mg/L	666
ADWF and PML	mg/L	1,065
ADWF and PDL	mg/L	1,331
TSS Concentration		
ADWF and AAL	mg/L	666
ADWF and PML	mg/L	1,065
ADWF and PDL	mg/L	1,331
TKN Concentration		
ADWF and AAL	mg/L	133
ADWF and PML	mg/L	213
ADWF and PDL	mg/L	266

(1) Reconciled flow is based on post 2008 data plus 5% allowance for storms with return frequency > 17 yrs

(2) No data for PHF. PHF is assumed to be 1.25 times the PDF

(3) Range of 365-day average load

(4) Range of 30-day average load

(5) Range of daily load

(6) Measured concentration at dry summer months

Section 2: Site Layout and Yard Piping

There are two potential site locations available for the Improvement Project processes:

Alternative 1 is situated at the collection system pump station (off Occidental Camp Meeker Road), and

Alternative 2 is within the existing treatment plant boundary (off of Occidental Road).

The only area available for new facilities at the pump station site is the space currently occupied by an abandoned package plant. The treatment plant site is restricted by the existing ponds, which must remain in service during construction, but has a small area west of the control and lab building. Both alternatives were analyzed to determine the most suitable site; the recommended location was based on space limitations, facility access, odors/noise/safety concerns, and life cycle costs.

Regardless of site location and process selection, Title 22 regulations require 24 hours of emergency storage to capture partially treated sewage during unpredictable plant upset conditions when duty equipment is out of service and standby equipment is available. The abandoned Imhoff tank and package plant basin, at the lift station site, can be used for storage of untreated wastewater. However, the available tanks volume, total capacity of 42,000 gallons, is insufficient to store peak 24-hours of flow. Therefore, raw or partially treated wastewater will need to be pumped to the pond plant site for storage and then returned to the lift station site for final treatment. Because of this limitation, the lift station site alternative requires an additional return pump station and longer effluent pipe line to discharge reclaimed water at the Morelli Lane Storage Reservoir, which adds approximately $200,000 to the construction and operational cost of the facility.

The lift station site has several other disadvantages, including being sandwiched between Dutch Bill Creek and Occidental Camp Meeker Road. Being located off a busy street diminishes site security (allowing higher volumes of citizens' access to sensitive infrastructure) and restricts vehicular ingress/egress (necessary for chemical deliveries, solids and biosolids waste removal, and routine operations). Further, this site is positioned across from residential homes that will be sensitive to additional odors and noise. Due to the additional costs, complications, and safety concern of using the lift station site for wastewater treatment, it is recommended that upgrades occur at the pond plant site facility.

The proposed site layout for the OCSD WWTF is shown on Figure ES-1. The overall layout is intended to accommodate plant process components. Proposed improvements are shown in bold.

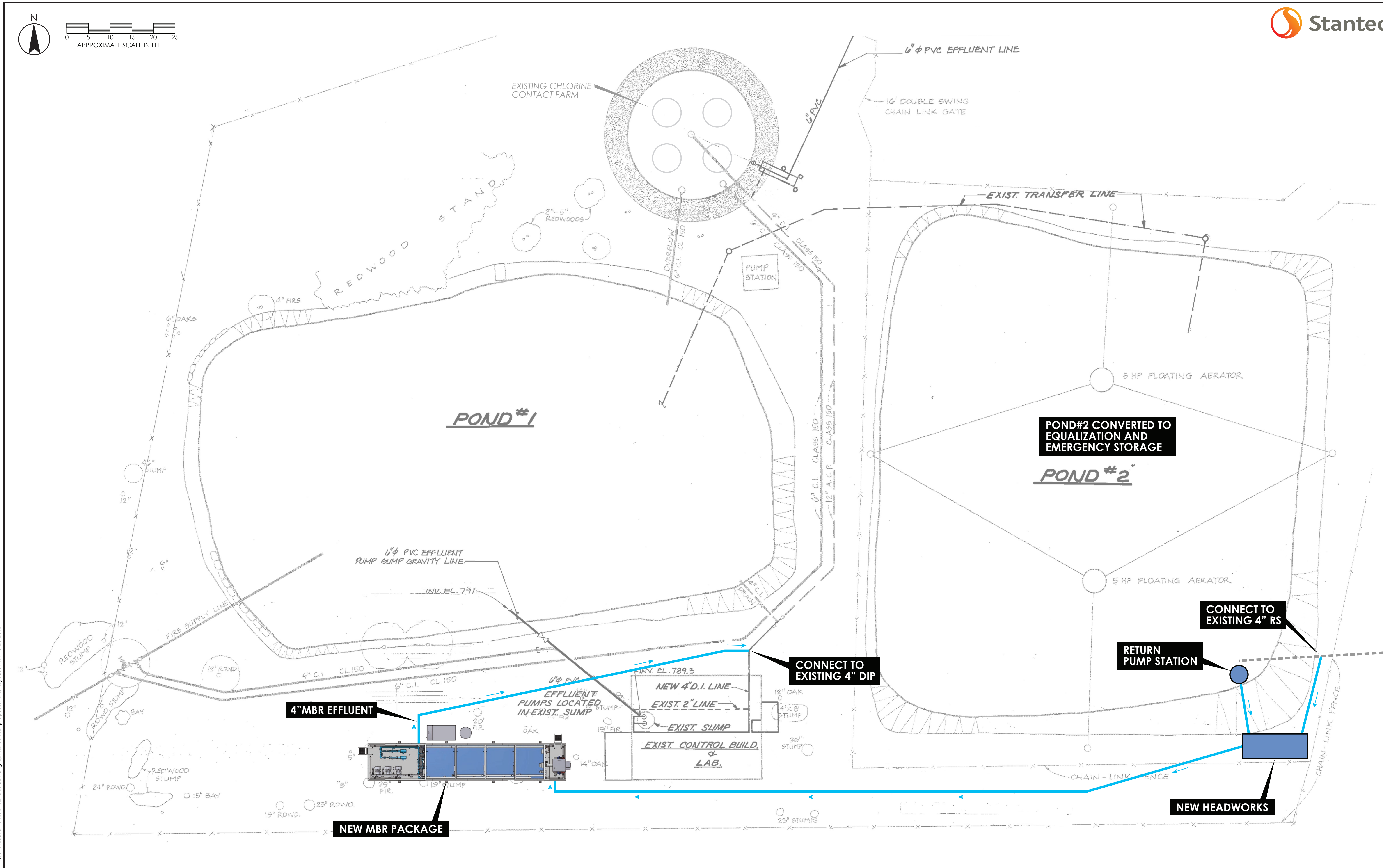


Figure ES-1
Proposed Site Layout

Section 3: Facility Improvements Alternatives Analysis

The OCSD wastewater treatment facility is required to provide a high level nitrification (ammonia removal) and denitrification (nitrate removal). Key effluent limitations that govern the process design are monthly average ammonia-nitrogen and nitrate-nitrogen concentrations of 1.2 mg/L and 10 mg/L, respectively. Unfortunately, the existing plant uses an aerated pond system which cannot be operated to fully nitrify or denitrify. Therefore, the treatment plant needs to be upgraded to an activated sludge facility where robust nitrification and denitrification can be achieved.

Based on previous evaluations conducted by SCWA, it was established that the three preferred alternatives for treatment processes upgrades are 1) a packaged MBR facility, 2) an SBR process with coupled disk filter, or 3) an Aeromod system with coupled disk filter. All three alternatives have various associated interdependent plant components that require improvements for permit compliance, including installation of a new headworks and flow splitting structure, potential modifications to the disinfection system, and site layout alternatives. The comparative evaluation of the process alternatives included consideration of all other aspects of the plant that would be impacted by the choice of the biological treatment option.

The various alternative combinations were rated with respect to several key economic and non-economic criteria, each of which were assigned an “importance weighting” factor. The analysis matrix was developed with the input and review of SCWA staff in an effort to assure that the criteria included in the table and the relative weighting factors appropriately reflect the interests and concerns of SCWA and OCSD. Selection of the apparent best project was based on the two alternatives with the highest overall weighted score. After preparing a life cycle cost analysis for all options, it became apparent that the best suited process selection for OCSD is the MBR facility, as it far outweighs the other two alternatives and has the cheapest installation and life cycle costs. Based on the evaluation, the recommended process for the OCSD WWTF is a membrane bioreactor.

After SCWA confirms the recommendations made herein and directs Stantec to proceed, Stantec will prepare a request for proposal to solicit guaranteed pricing from MBR equipment manufacturers. These proposals will be used in the basis of design report, which will fully develop the design criteria for the project. The basis of design report will also include details on project components that are independent of the alternatives analysis, including an effluent pump station, final effluent storage reservoir and pipe alignment, solids handling, electrical improvements, and pond decommissioning.

Section 4: Disinfection System Alternatives Analysis

The purpose of this section is to assess the chlorine disinfection process and determine the improvements to the chlorination system and analyze ultraviolet (UV) disinfection as an alternative to the existing process. Currently the OCSD WWTF disinfects its effluent by mixing its secondary effluent with sodium hypochlorite. Disinfection contact time is provided by series of chlorine contact tanks (5,000 gallons each) that provides approximately 50 minutes of contact time at current maximum monthly flows. After the two contact tanks, sodium bisulfite is injected into the piping manifold and sent through the last contact tanks (3000 gallons) for dechlorinating the effluent prior to discharge.

To achieve Title 22 compliance, the chlorine system must provide a CT (the product of total chlorine residual and modal contact time measured at the same point) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow. For either the UV system or chlorine system, the disinfection process must be able to inactivate and/or remove 99.999 percent of the plaque-forming units of F-specific bacteriophage MS2, or polio virus in the wastewater, and coliform bacteria measured in the disinfected effluent must not exceed an MPN of 2.2 per 100 milliliters for the last 7 days.

There is a higher potential for the formation of chlorine disinfection byproducts, such as dichlorobromomethane (a known carcinogen), when the plant is upgraded to remove nitrogen. Even though the current NPDES permit allows for low levels of disinfection byproducts to be discharged into Dutch Bill Creek, these requirements are likely to become more restrictive in the future, which will make compliance increasingly difficult. Thus, if the Water Agency wants to maintain the use of the existing surface discharge, for flexibility when land application or reclaimed water storage is not available, the risk of violation should be measured against the added costs of installing a UV disinfection system.

Ultimately, SCWA must decide whether the risk of violation during surface water discharge is worth continued operation of the existing chlorine disinfection system. Assuming the effluent can be fully reused or discharged to Dutch Bill Creek, it is recommended to maintain the existing chlorine disinfection system because all associated equipment and tanks were recently installed and will remain compliant with the existing permit. However, when the new chlorine tanks begin to degrade, instead of installing new tanks, it is recommended to upgrade to a UV disinfection system, as this will allow the WWTF more flexibility in discharging final effluent to either the reclamation system or to Dutch Bill Creek.

Section 5: Solids Handling Alternatives Analysis

The purpose of this section is to evaluate different options for solids handling. As a result of the proposed improvements to the OCSD WWTF solids will be produced on a daily basis. With construction of new secondary treatment facilities, the existing ponds will become available for other uses. Since the existing aeration pond is already allocated to be used as an emergency and equalization storage basin, the remaining settling pond is available to be reused for the solids handling facilities.

The selection of the solids handling alternative depends on the current waste discharge requirements and available disposal options. Considering that the current NPDES permit does not allow OCSD to land apply their biosolids and allows only disposal to a landfill or another appropriately permitted facility, there is no monetary incentive for the OCSD to produce Class A or Class B biosolids for land application. Therefore, the options considered in solids handling alternative analysis include only those options that will meet minimum requirements for a landfill disposal, i.e. to produce sludge with a minimum 15% solids content.

The options evaluated for OCSD include Solids Stabilization / Storage Lagoon with sludge dredging option and sludge drying option; and solids dewatering using a dewatering tube and a dewatering box. The selection of solids handling alternative was based on life cycle cost analysis and non-monetary criteria such as potential to attract vectors and create nuisance odors which is specifically prohibited by the current NPDES permit.

The solids handling alternative proposed for the OCSD is a dewatering tube with a container. The waste sludge from secondary process is first conditioned with polymer and pumped directly into the dewatering tube which is located in the watertight roll-off container. The permeable material used to construct the tube allows excess water to seep out while retaining the solids and fine particles. From the container water flows onto the concrete pad or into a drainage pipe connected to the container drainage outlet from where it is further conveyed into the return pump station. When the tube fills with sludge it is replaced with a new tube. The improvements required for this alternative include a polymer blending unit, 25 ft x 10 ft concrete pad, a watertight 20-yard roll-off container (rented), purchase of one-year supply of dewatering bags, and piping for solids feed and decant drainage.

ES.4 ENGINEERS OPINION OF PROBABLE CONSTRUCTION COSTS

An overall alternative cost analysis including the combined costs of equalization storage and headworks, biological treatment and filtration, and solids handling is shown in Table ES-2.

Table ES-2 Engineers Opinion of Probable Construction Cost for WWTF Improvements

ITEM	COSTS, $
WWTF Improvements	
Headworks Screens	321,000
Emergency Storage Basin	149,000
Packaged MBR	976,000
Solids Handling	58,000
Subtotal 1	***1,504,000***
Paving and Grading, 7%	105,000
Site Piping, 15%	226,000
Electrical, 25%	376,000
Subtotal 2	***2,211,000***
Contingencies, 25%	553,000
Total Construction Cost	**2,764,000**

Note: Constructions costs do not include effluent pump station and pipeline to new effluent storage reservoir.

SECTION 1 INFLUENT FLOWS AND LOADS

1.1 PURPOSE

The purpose of this Section is to evaluate the existing and estimate the projected future wastewater flows and loads at the Occidental County Sanitation District (OCSD) Wastewater Treatment Facility (WWTF).

1.2 ANALYSIS OF HISTORICAL FLOWS AND LOADS

1.2.1 Historical Flows

Influent flow data for the period from January 2000 through October 2014 were obtained and analyzed. There is a significant inflow and infiltration (I/I) component in the influent flow during the wet weather flow, as shown in Figure 1-1. Some improvements were done to the collection system in 2007 to repair and replace the most problematic infrastructure, which has helped reduce some levels of I/I. However, it is difficult to estimate the true impacts of the investment because no large rain events have occurred after the capital improvements projects were complete. While Figure 1-1 shows a peak storm event in October 2010 similar to an event occurring in December 2005 (7.75-inches and 7.2-inches of precipitation respectively), the scale of the figure makes it difficult to see that the 2005 storm had two weeks of heavy rain that totaled more than 30-inches of precipitation (increasing effects of I/I and resulting in an influent flow rate of 0.2 Mgal/d), while the 2010 storm had only 0.75-inches of precipitation in the two weeks leading up to rainfall (resulting in an influent flow rate of 0.05 Mgal/d). Therefore, to ensure the new treatment facility will be able to handle the unpredictable surge event, hydraulic peaking factors will be based on all historically available data.

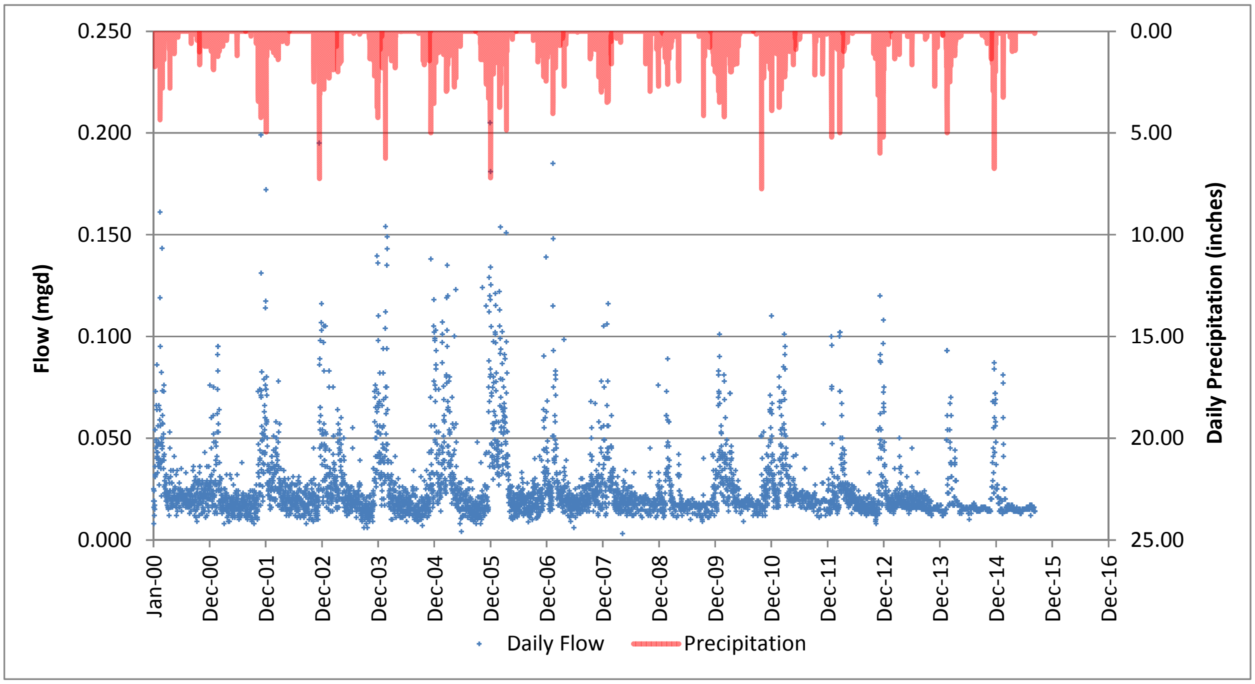


Figure 1-1 Influent Flow and Precipitation

The average dry weather flow (ADWF), calculated as the average flow during the driest three months of the year (July, August, and September), has remained nearly constant during the last 14 years, at approximately 0.018 Mgal/d, as shown in Figure 1-2. The ratios of the annual average, monthly, weekly, and daily flows to the ADWF are shown in Figures 1-3 through 1-6. It must be noted that the peak flows were reduced after 2008 due to the collection system rehabilitation efforts that started around 2008. The storm events occurred after 2008 were in the range of 10 to 17 years return frequency as shown in Table 1-1.

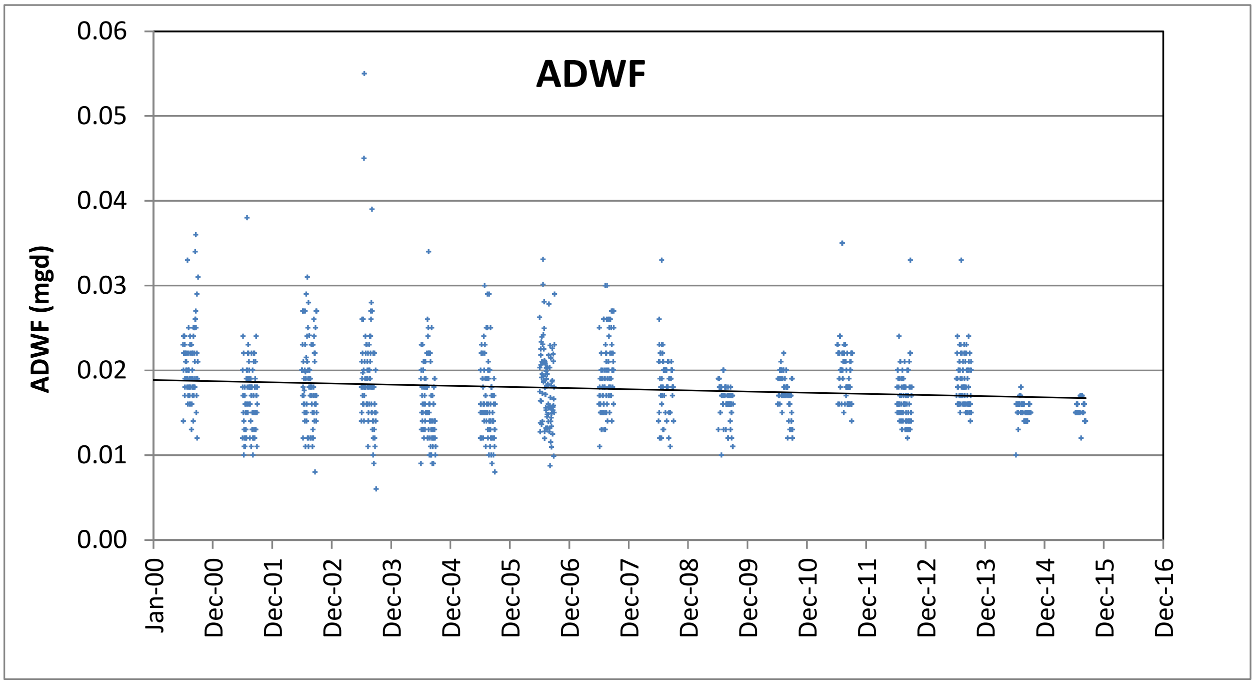


Figure 1-2 Average Dry Weather Flow

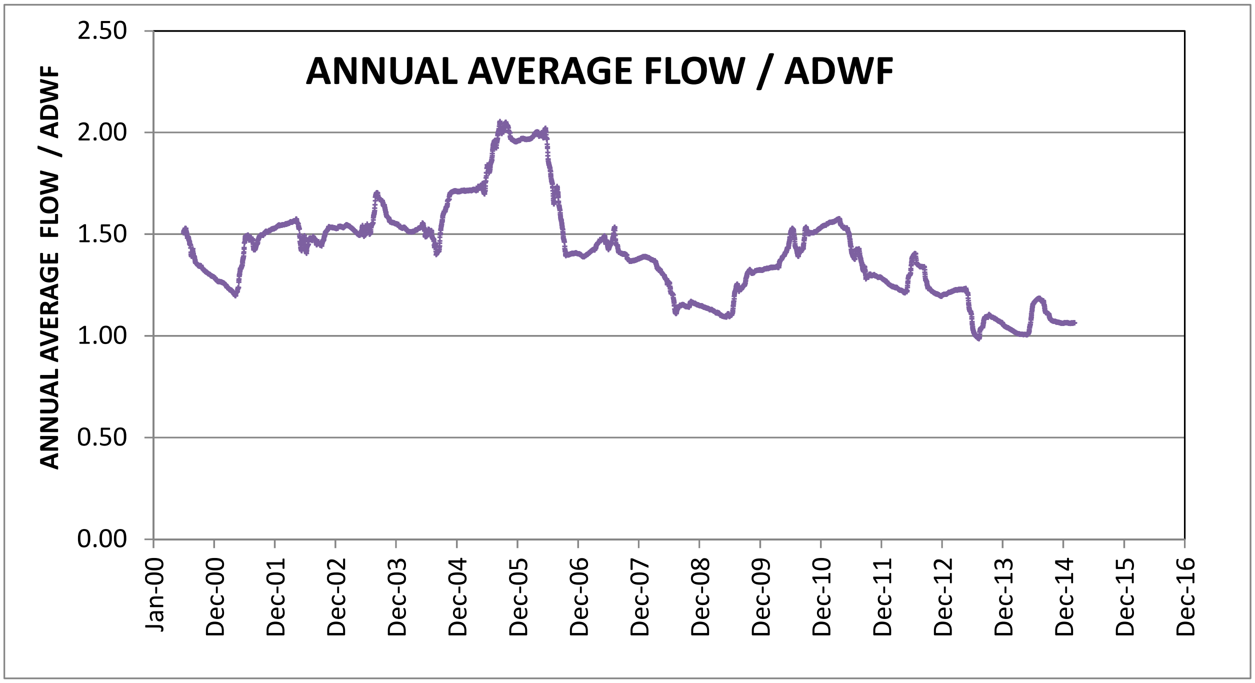


Figure 1-3 Ratio of Annual Flow to ADWF

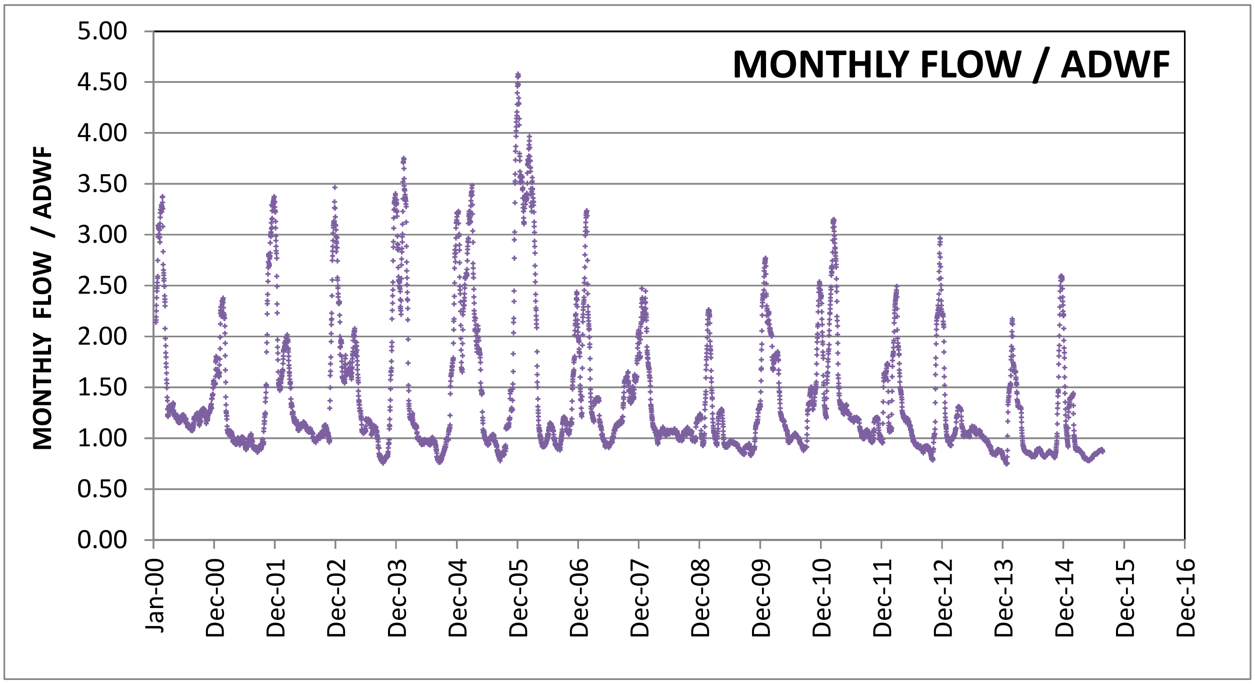


Figure 1-4 Ratio of Monthly Flow to ADWF

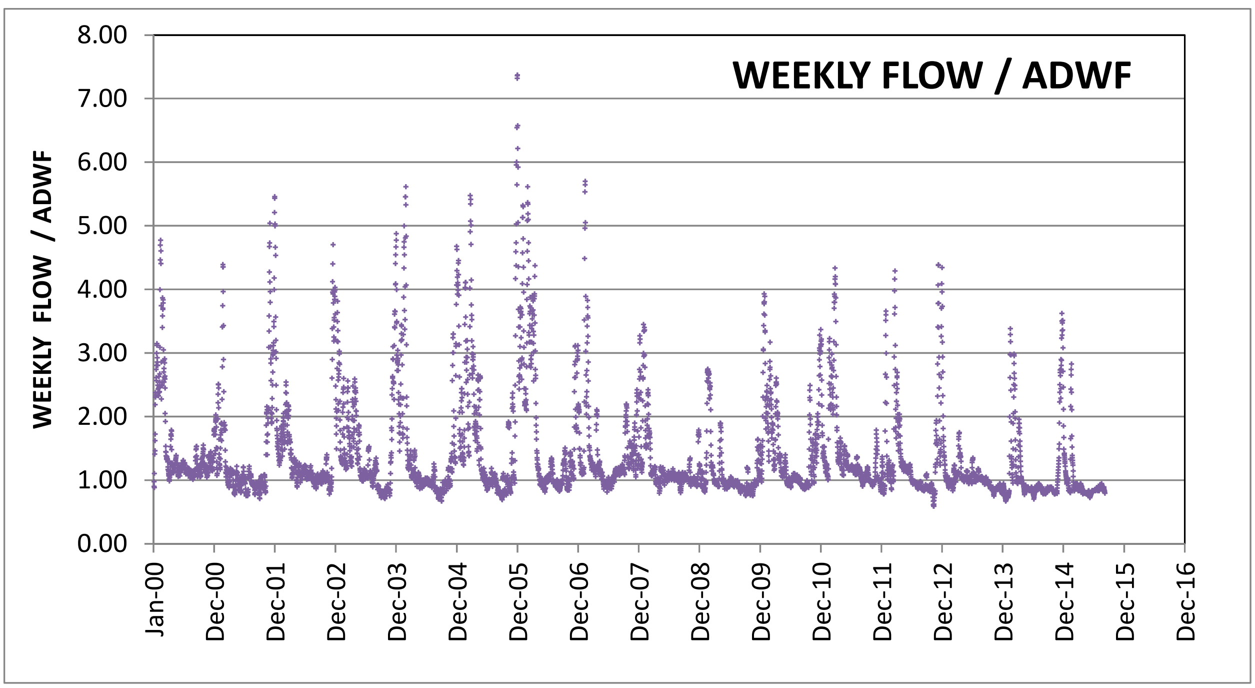


Figure 1-5 Ratio of Weekly Flow to ADWF

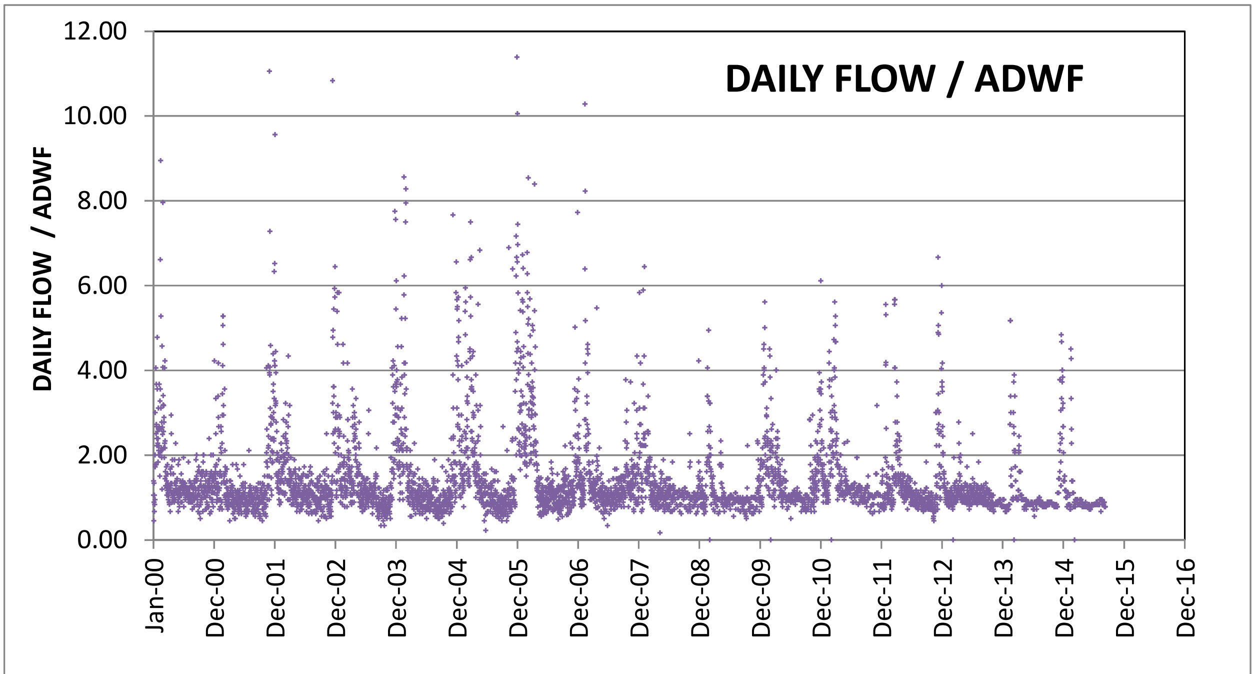


Figure 1-6 Ratio of Daily Flow to ADWF

Table 1-1 Precipitation Return Frequency after 2008

Precipitation (inches)				Return Frequency (yrs)	Date
24-hr	**2-day**	**3-day**	**4-day**		
7.75				16.9	10/23/2010
	9.5			9.9	10/24/2010
		10.7		9.4	12/1/2012
			13.1	15.6	1/22/2012

Since the Occidental is a resort community, it is typical to have higher occupancies and transient populations on weekends than on weekdays. This phenomenon was evaluated by analyzing median flows for each day of the week over the entire period of record. The results are shown in Figure 1-7. As indicated, weekend flows are typically around 115 percent of midweek flows. This phenomenon is very important when considering an influent monitoring program to correctly represent the OCSD wastewater characteristics. This topic is discussed further later in this section.

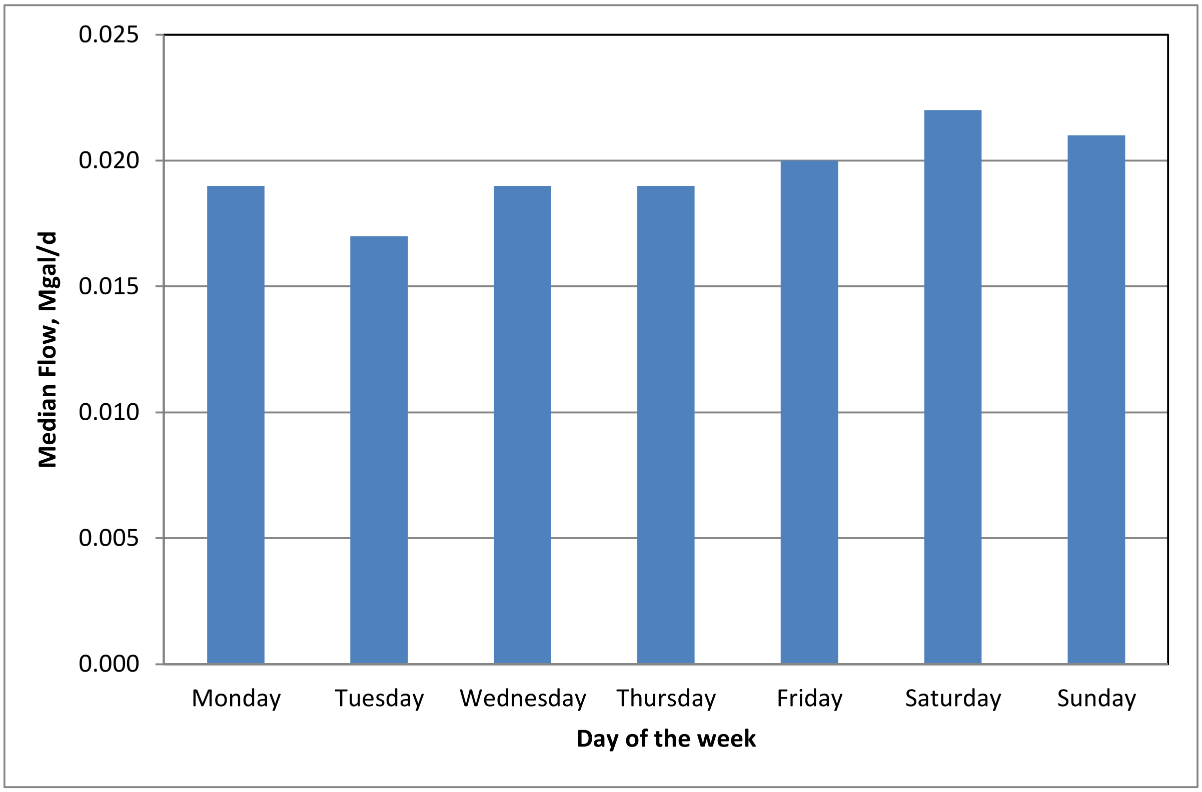


Figure 1-7 Median Flow by Day of Week

1.2.2 Historical Loads

For wastewater treatment plant design, it is essential to have a good understanding of the constituent loadings that the plant will receive. The term load or loading refers to the total mass or weight of a constituent entering the wastewater treatment plant over a specific period of time. Loadings are normally expressed in units of pounds per day. A constituent load for a given time period is determined by multiplying the average flow times the flow-weighted average constituent concentration during that period, and then applying a conversion factor to get the desired units (pounds per day).

The main constituents of concern are the biochemical oxygen demand (BOD5, or simply BOD) and total Kjeldahl nitrogen (TKN). TKN includes organic nitrogen (typically about 1/3 of the TKN) and ammonia nitrogen (typically about 2/3 of the TKN). Since there is normally no nitrite or nitrate nitrogen in the plant influent, TKN usually comprises the total influent nitrogen. Other influent parameters that are also important include total suspended solids (TSS) and alkalinity.

Plant influent BOD and TSS concentrations from same time period (January 2000 through July 2014) were collected and analyzed. Samples were 24-hour time composites and were taken weekly until the end of 2003. Starting from the beginning of 2004, the influent samples were collected once a month. Influent samples have always been collected on weekdays, not on weekends. Just as is the case for flows, it is believed that weekend loads can be much higher than weekday loads and therefore historical sampling data may not be an accurate representation of peak loading characteristics.

BOD and TSS concentrations and the daily influent flows were used to calculate influent load. The monthly and annual loadings were calculated as the 30-day rolling average and 365-day rolling average, respectively. The maximum annual average BOD load observed in the analyzed period was about 110 lb/d and was observed around 2009 and 2010 (See Figure 1-8).

The TSS data shown in Figure 1-9 exhibits a pattern similar to the BOD data. On average, the ratio of influent TSS to BOD is approximately 0.76 (See Figure 1-10), which is less than a typical ratio of 0.9-1.1. This may indicate that that the collected samples are perhaps unrepresentative.

Only limited recent TKN data are available, as shown in Figure 1-11. TKN has generally ranged from about 34 to 130 mg/L. The TKN high value of the 130 mg/L as N is a probably an outlier. A ratio of TKN to BOD of around 0.2 would be expected and has ranged from 0.1 to 0.4. Because limited historical data exist, standard TKN values at 20% of influent BOD were used as the design loading. More samples are needed to confirm that these high TKN values are not typical.

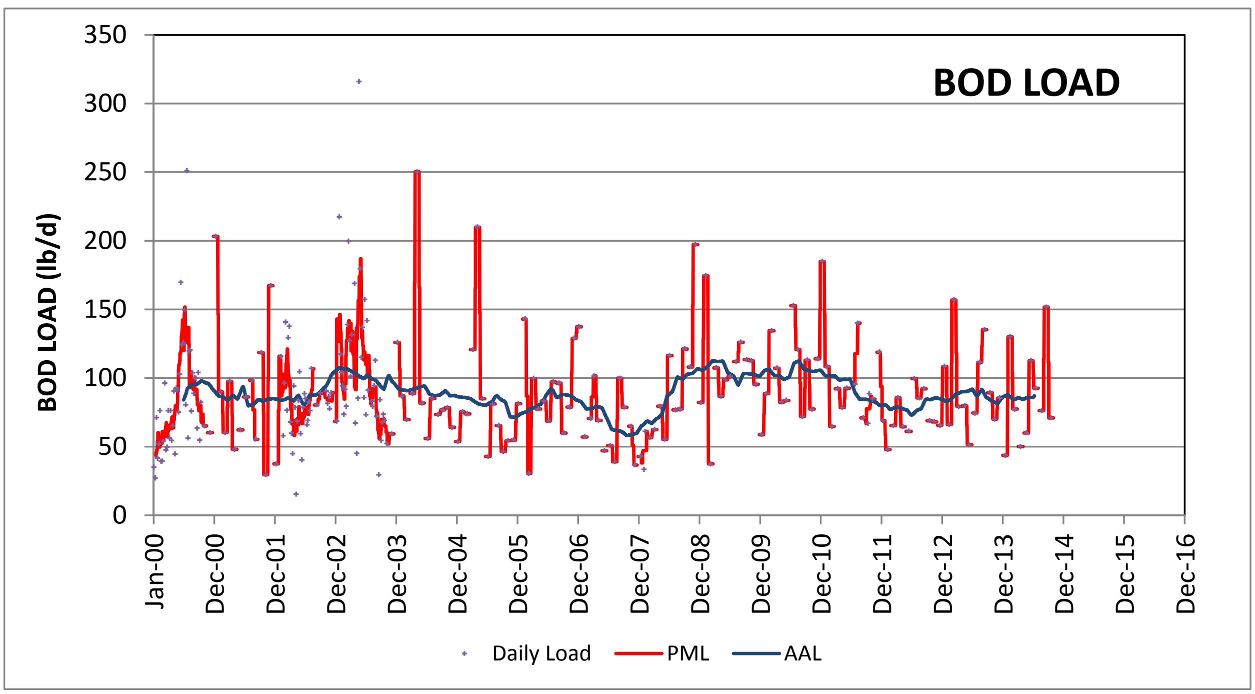


Figure 1-8 Midweek Influent BOD Load

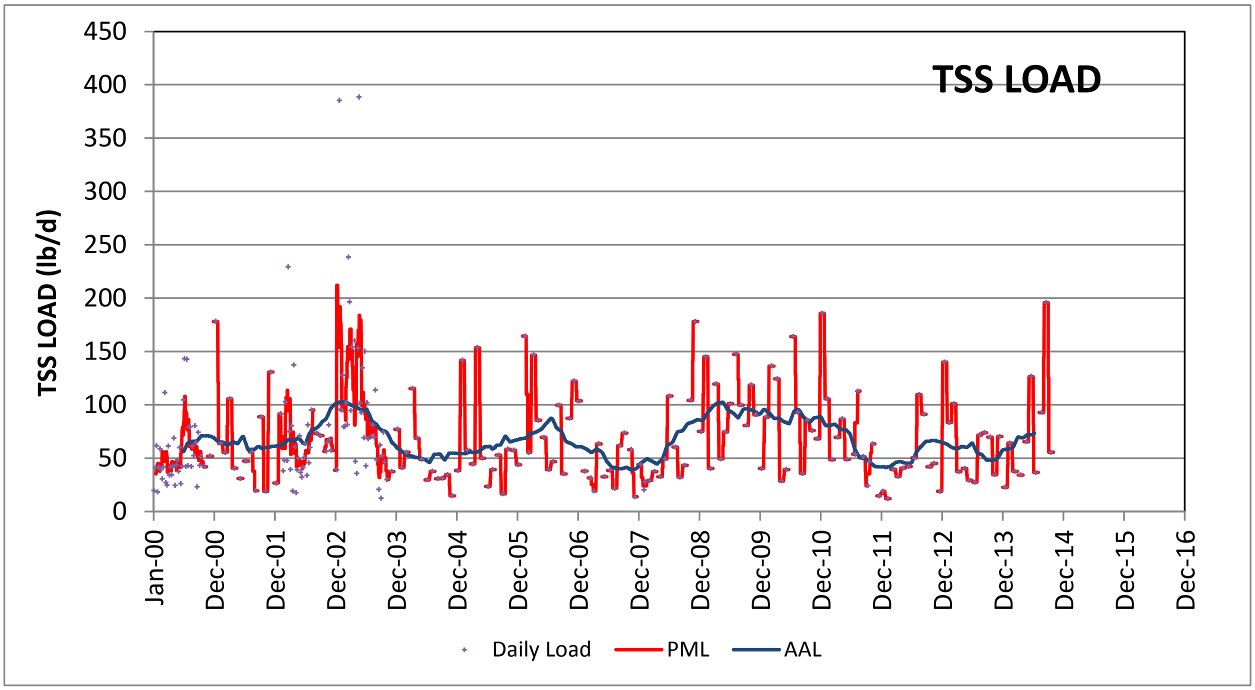


Figure 1-9 Midweek Influent TSS Load

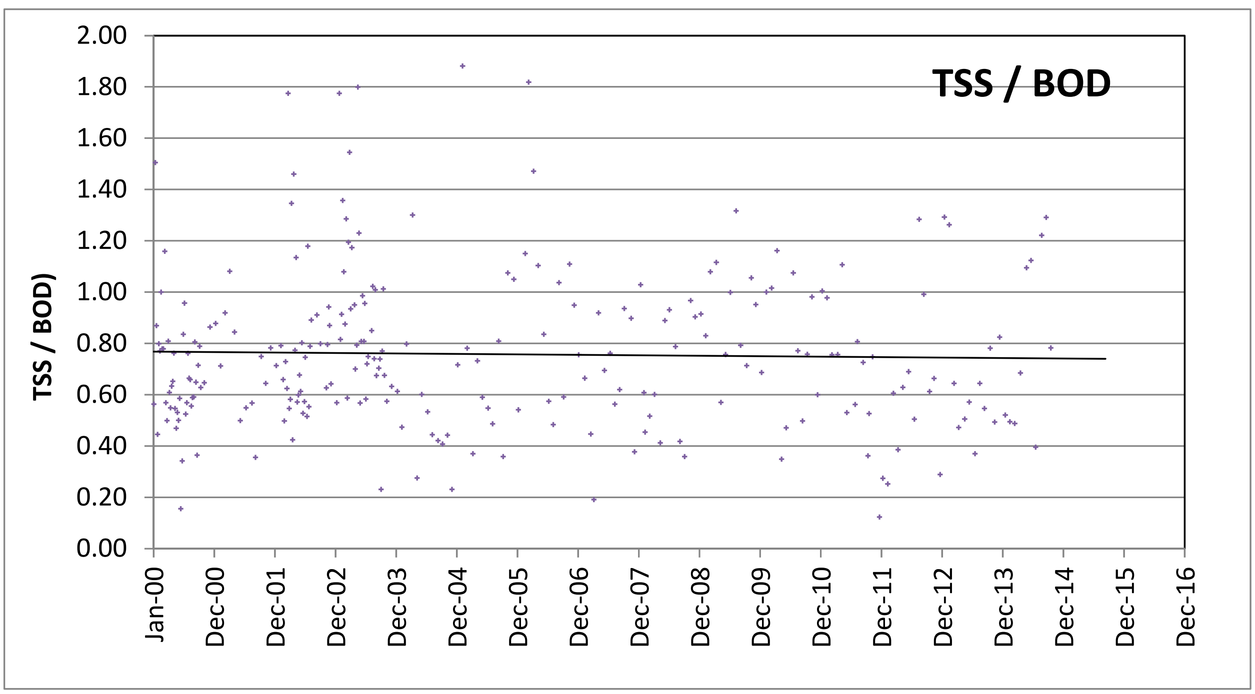


Figure 1-10 Ration of TSS/BOD

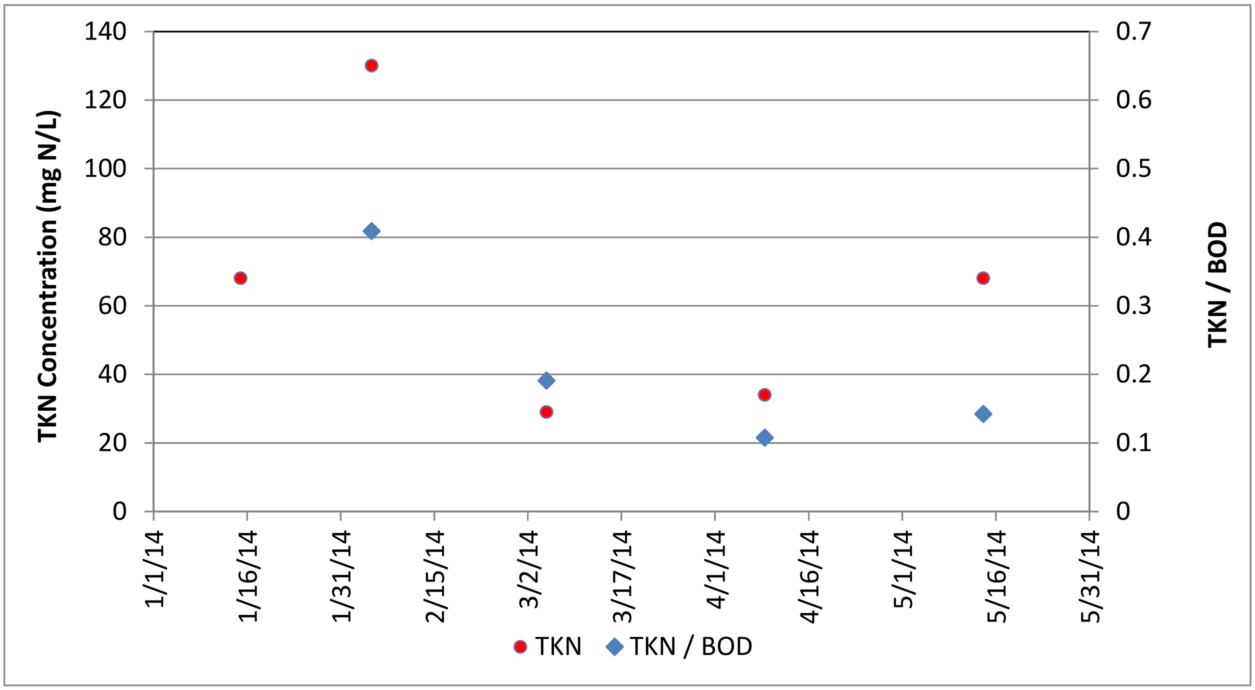


Figure 1-11 Influent TKN and Ration of TKN/BOD

1.2.3 Flows and Loads Reconciliation

1.2.3.1 Influent Flow Rates

As a means to confirm the historical data, the flow rate was calculated using textbook values (76 gal/capita-day for residential connections and 1500 gal/acre-day for commercial connections, adopted from Metcalf and Eddy, 4th edition). Based on the 2010 census data, the town of Occidental has a population density of 2.1 people/home. SCWA records indicate the sanitary sewer collection system has 55 existing residential connections and 11.3 acres of commercial space. Therefore, the average flow rate estimated for the existing connections is 25,725 gpd. The estimated flow rate is effectively the same as the annual flow rate estimated by historical data and is therefore verifiable. Further, it was confirmed by operational staff that the existing influent flow meter, located on the collection system pump station discharge manifold, is regularly calibrated and reliable.

Peak flows have been reduced after 2008 due to collection system rehabilitation. It is assumed that the collection system improvements will continue to prevent excessive inflow/infiltration at wet weather events. There were some reasonably large storms (10 to 17 year storms) in the period from 2008 through 2014 (See Table 1-1). However, it is desirable to design for larger storm events, perhaps 25 year storm. Since there is no correlation between precipitation and the peak flow, it is assumed that the design peak flows will be 5% larger than the 2008-2014 data analyzed.

1.2.3.2 Influent Loading Characteristics

Because a large sector of the local economy is tourism, commercial properties (including retail, restaurants, and hotels) are a significant source of wastewater generation and influent constituent concentrations. Therefore, it is expected that the OCSD WWTF will correspond to a high strength domestic wastewater. Further, samples have always been collected mid-week (not on the weekends) and because of the transient nature of the tourist community, it is believed that weekend loading may be even higher than suggested by the historical data.

The observed average BOD concentration at the periods of dry weather flow is about 666 mg/L as shown in Figure 1-12. To adjust for the lack of weekend data, the peak month and peak day BOD are assumed to be slightly higher than observed data as shown in Figure 1-12. The selected BOD concentration corresponding to peak month, and peak day loadings (all in conjunction with the average dry weather flow) are 1,065 and 1,331 mg/L, respectively.

Also, as indicated above, the observed TSS/BOD ratio were highly variable (as shown in Figure 1-10) with a mean value of 0.76. A typical TSS/BOD ratio for domestic raw wastewater ranges between 0.9 and1.1. For conservative design, it is reasonable to assume that the TSS is equal to the BOD (i.e., the ratio of TSS/BOD is 1.0).

These adjustments are based on engineering judgment on the available data. However, it is recommended to provide a two week intensive sampling program during the peak tourist season, with flow proportional composite samples that are withdrawn from a well-mixed location. This will ensure the samples are representative of the influent flow characteristics and allow the process to be sized appropriately during detail design.

The remainder of the alternatives analysis report is based on the assumption that historical monitoring data are reliably accurate.

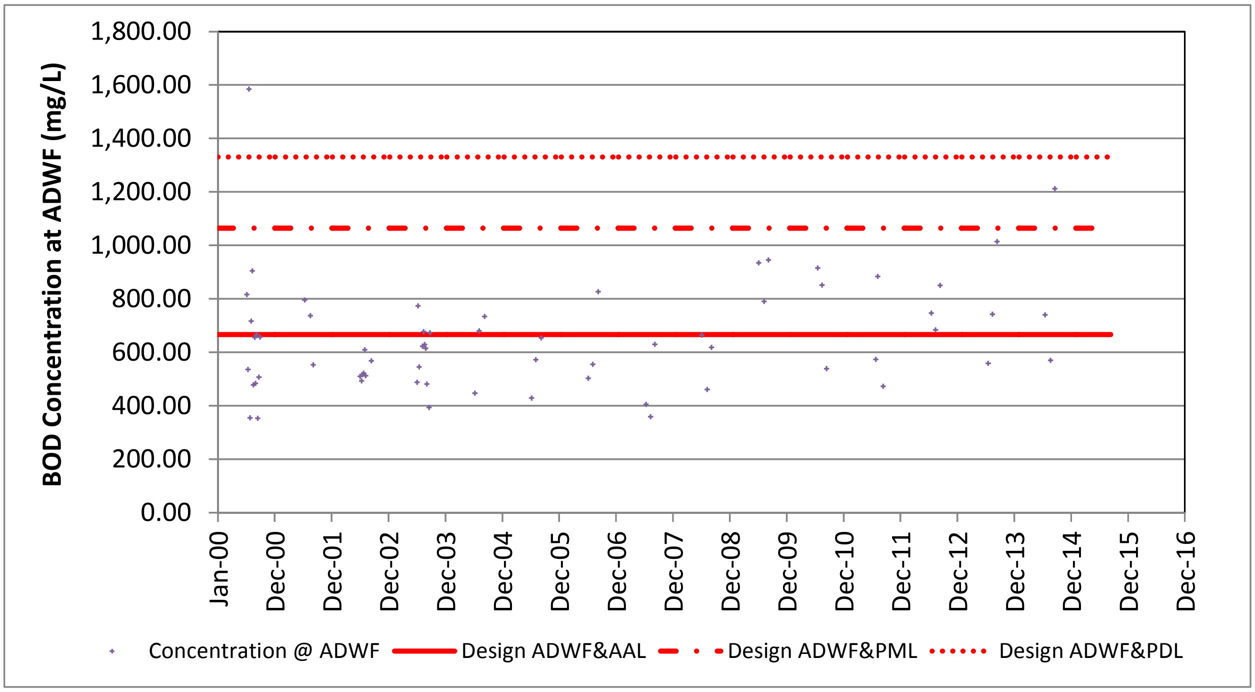


Figure 1-12 BOD Concentration at ADWF – Observed Data vs Design Assumptions

1.3 DESIGN FLOWS AND LOADS

Based on the analysis presented above, current and projected flows and loads are summarized in Table 1-2.

Table 1-2 Existing and Design Flows and Loads

Parameter	Unit	2000-2008	2008 - 2014	Reconciled Current Conditions	Project Design Criteria Based on Reconciled Loading
Flow					
ADWF	Mgal/d	0.018	0.018	0.018	0.025
AAF	Mgal/d	0.036	0.027	0.027	0.038
PMF(1)	Mgal/d	0.081	0.058	0.060	0.083
PWF(1)	Mgal/d	0.126	0.079	0.083	0.115
PDF(1)	Mgal/d	0.198	0.126	0.132	0.183
PHF(2)	Mgal/d			0.166	0.229

Parameter	Unit	2000-2008	2008 - 2014	Reconciled Current Conditions	Project Design Criteria Based on Reconciled Loading
Flow Peaking Factors					
AAF/ADWF	-	2.00	1.50	1.50	1.50
PMF/AAF	-	4.5	3.2	3.3	3.3
PWF/AAF	-	7.0	4.4	4.6	4.6
PDF/AAF	-	11.0	7.0	7.3	7.3
PHF/AAF	-	0.0	0.0	9.2	9.2
BOD Loads					
AAL	lb/day	58-108 (3)	73-113 (3)	100	139
PML	lb/day	30-250 (4)	37-185 (4)	160	222
PDL	lb/day	15-315 (5)	37-185 (5)	200	278
TSS Loads					
AAL	lb/day	39-103 (3)	41-102 (3)	100	139
PML	lb/day	14-212 (4)	12-195 (4)	160	222
PDL	lb/day	12-388 (5)	12-195 (5)	200	278
TKN Loads					
AAL	lb/day			20	28
PML	lb/day			32	44
PDL	lb/day			40	56
Load Peaking Factors					
PML/AAL	-			1.6	1.6
PDL/AAL	-			2.0	2.0
BOD Concentration					
ADWF and AAL	mg/L	352-1584 (6)	472-1212 (6)	666	666
ADWF and PML	mg/L			1,065	1,065
ADWF and PDL	mg/L			1,331	1,331
TSS Concentration					
ADWF and AAL	mg/L	184-900 (6)	206-1564 (6)	666	666
ADWF and PML	mg/L			1,065	1,065
ADWF and PDL	mg/L			1,331	1,331
TKN Concentration					
ADWF and AAL	mg/L		29-130	133	133
ADWF and PML	mg/L			213	213
ADWF and PDL	mg/L			266	266

(1) Reconciled flow is based on post 2008 data plus 5% allowance for storms with return frequency > 17 yrs
(2) No data for PHF. PHF is assumed to be 1.25 times the PDF
(3) Range of 365-day average load
(4) Range of 30-day average load
(5) Range of daily load
(6) Measured concentration at dry summer months

SECTION 2 SITE SELECTION AND LAYOUT

2.1 PURPOSE

The purpose of this section is to evaluate the two options for locating the upgraded processes for Occidental County Sanitation District (OCSD) Wastewater Treatment Facility (WWTF) to comply with the California Code of Regulations, Title 22 unrestricted reuse.

The remainder of this section is organized into the following sub-sections:

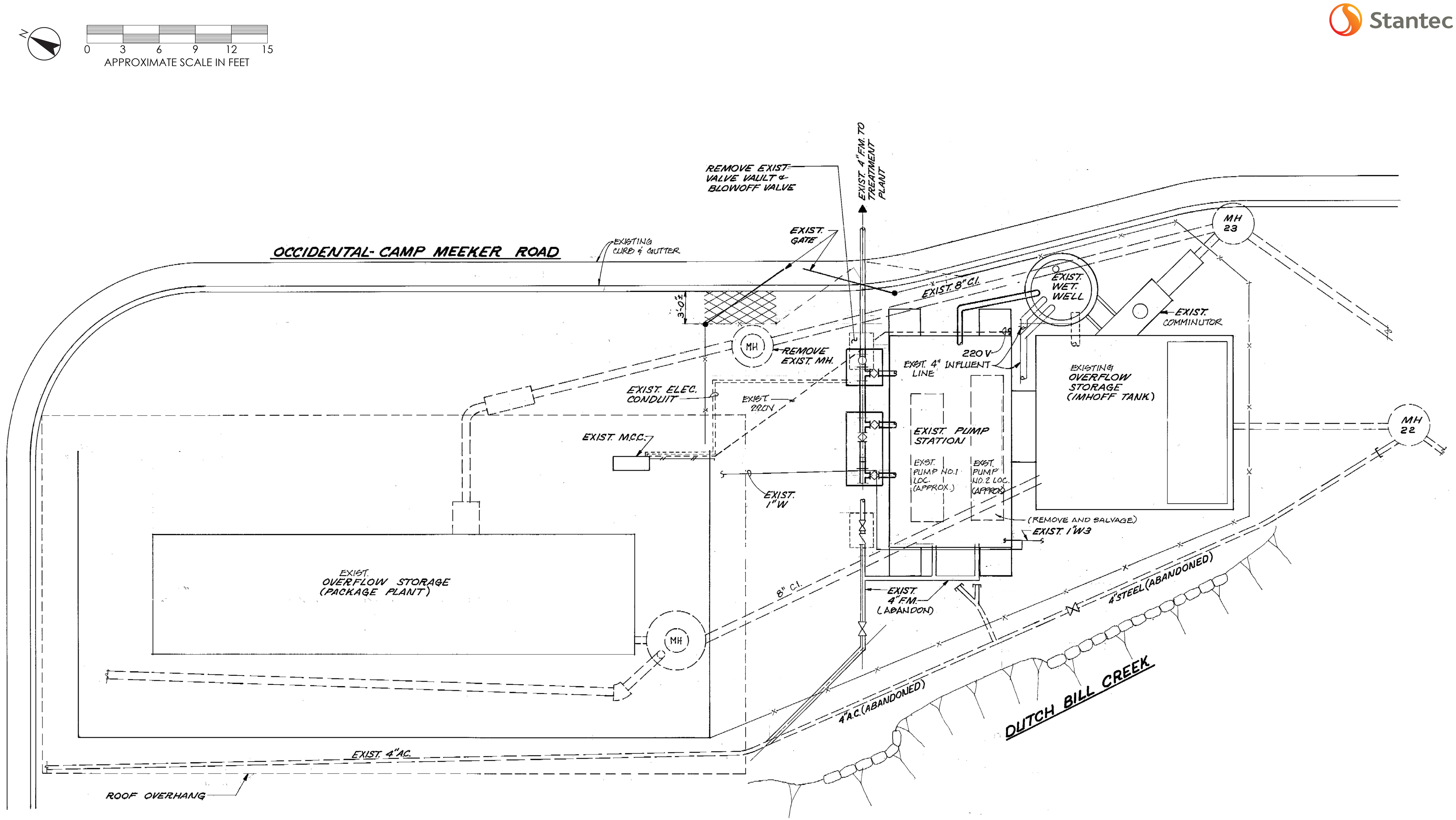
- Existing Sites and Facilities
- Site Selection Alternatives
- Site Selection Evaluation and Recommended Design

2.2 EXISTING SITES AND FACILITIES

Based on previous evaluations conducted by Sonoma County Water Agency (SCWA), it was established that the two alternative sites available for locating the new treatment processes are: 1) the centralized collection system pump station site and 2) the remote pond plant.

OCSD collects wastewater through a network of gravity sewer pipelines. The collection system combines influent wastewater into one centralized pump station, located on Occidental Camp Meeker Road. The pump station is sandwiched between a two lane public road and Dutch Bill Creek, as shown in Figure 2-1. Approximately half of the property is used to house an abandoned package plant that is situated under an existing canopy. The remainder of the site includes a functioning collection system wetwell, an overflow structure, and dry pit pump station that sends wastewater to the OCSD WWTF for treatment.

The existing WWTF, as shown in Figure 2-2, was originally constructed in 1973 and has gone through several subsequent improvement projects. The site consists of a two-stage pond treatment system (one aeration pond and one settling pond), a chlorine disinfection tank farm, and a small operational control/lab building. The pond plant site is located adjacent to the Druid's cemetery and is accessed from Lu-Dan Road, a dead-end street that is used mostly by cemetery visitors and treatment plant personnel.



Stantec

Figure 2-1
Existing Pump Station Site Plan

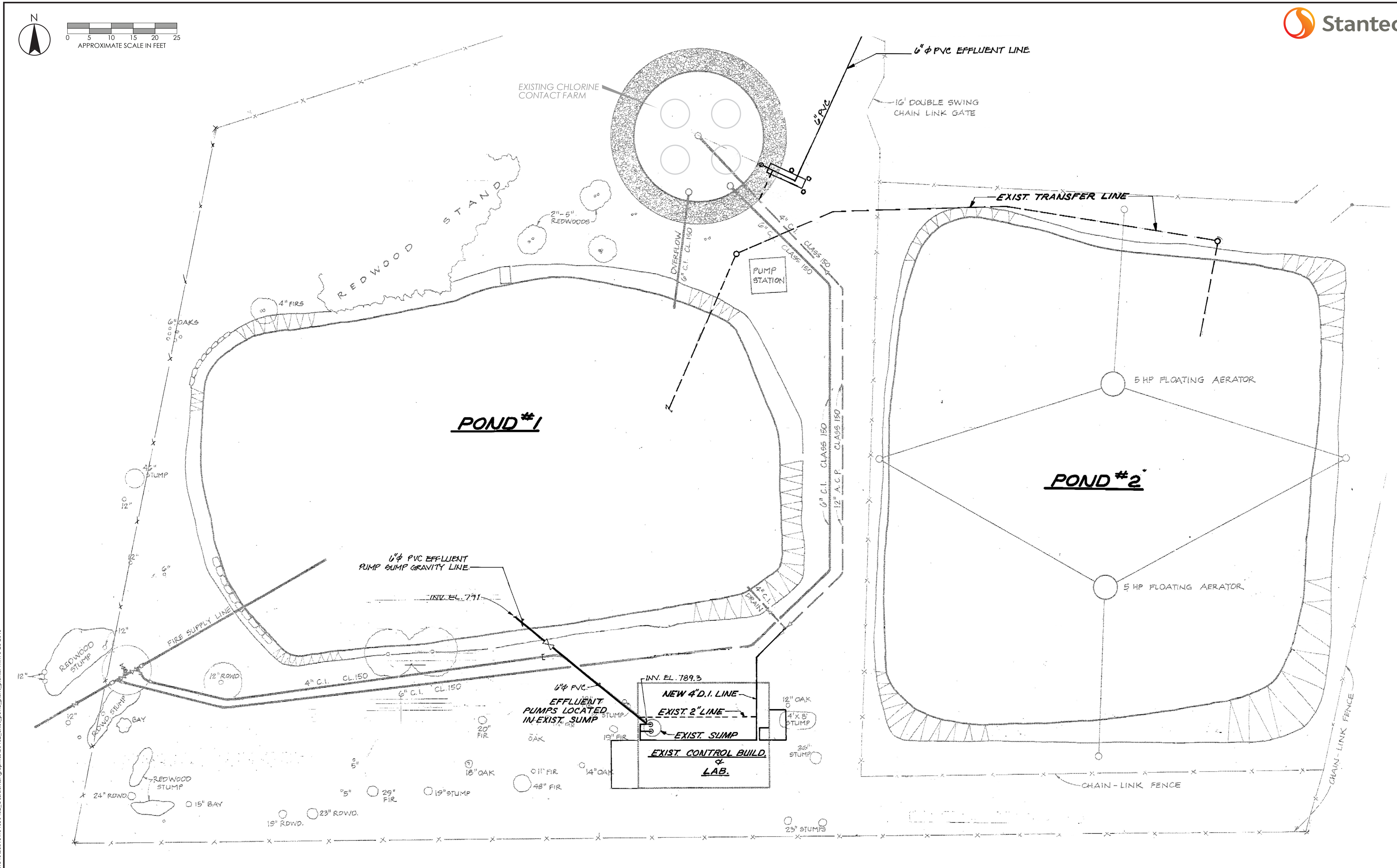


Figure 2-2
Existing Pond Plant Site Plan

2.3 SITE SELECTION ALTERNATIVES

The established alternative sites available for locating the new wastewater treatment processes are 1) the pump station site and 2) the pond plant site. Both sites have benefits and disadvantages associated with locating new treatment facilities within the property's boundary. Both site location alternatives are discussed further below.

2.3.1 Alternative 1: Pump Station Site

One benefit of repurposing the pump station site to house the new treatment processes is the ability to reuse an existing canopy to cover any new processes and provide protection of both equipment and operational personnel from exposure to harmful environmental elements. Additionally, this site allows the continued use an abandoned package plant as an overflow structure to capture influent wastewater flows that exceed the plant capacity. Further, its centralized location allows for relatively easy upgrades to third party utility connections (including the electrical service).

However, using the pump station site has several disadvantages, including restricted egress/ingress onto the property with only a small zone for parking and deliveries. The limited site access will become a safety concern for large commercial trucks used to deliver chemicals and haul solids away from the plant. Although unlikely, there are additional safety concerns associated with installing a new treatment facility in direct contact with the public. While the existing fence provides a certain level of protection for all equipment, there is no fully enclosed structure on site that will completely insulate the equipment, controls, or instrumentation from malicious vandals.

Additionally, the site is surrounded by residential neighbors who are sensitive to both noise and odors, which are common to wastewater treatment plants. With little buffer between the treatment plant and residential properties, additional costs must be added to the overall scope of the project to include sound baffling enclosures on equipment and treat any noxious odors emanating from the raw sewage and stored sludge. Further, if the new processes are to be constructed below the existing canopy, they will sit on top of the existing concrete basin that currently houses the abandoned package plant and is used for emergency overflow. This complicates construction of the new processes because they either must be structurally separated from the existing concrete basin (supported by new steel beams and perimeter concrete footings) or the existing basin must be improved to meet more stringent seismic and structural building codes. Either installation technique will increase the construction costs by approximately $50,000.

The most critical disadvantage of using the pump station site to house the treatment processes is that there is no additional room to provide emergency storage beyond the existing overflow storage basins. Currently overflow is sent to the abandoned package plant and Imhoff tank concrete basins, with a total storage volume of 42,000 gallons. When backup equipment is not available during unpredictable plant upset conditions, Title 22 regulations require 24 hours of emergency storage to capture partially treated sewage. Because there is insufficient volume available to store the peak day flow, 275,000 gallons, raw or partially treated wastewater will need to be pumped to the pond plant site for storage and then returned to the lift station site for final treatment. This can be achieved by using the existing pump station and raw sewage piping to send flow up to the pond plant, but will require an additional return pump station and a longer effluent pipe line to discharge reclaimed water at the Morelli Lane Storage Reservoir. These items increase to the construction and operational cost of the facility by approximately $200,000.

The site is further complicated with the surrounding environmentally sensitive resources. For example, old-growth redwood trees line the street and makes off-site modifications (including new reclaimed water discharge piping) more cumbersome to permit and construct. Further, Dutch Bill Creek is directly adjacent to the property boundary and is a tributary to Russian River (a highly restricted waterway regulated under the North Coast Region's Basin Plan, which prohibits discharging potential toxins to the Creek). Therefore, all potentially contaminated stormwater that falls on the property, during construction and final operations, would need to be treated before running off the site.

2.3.2 Alternative 2: Pond Plant Site

There are several benefits associated with upgrading the wastewater treatment processes on the existing pond plant site. One advantage to this alternative is that there is sufficient volume in the existing ponds (approximately 270,000 gallons in Pond 1 and 500,000 gallons in Pond 2) to use as emergency storage of partially treated wastewater. As stated above, to maintain compliance with Title 22 regulations, the facility must have 24-hours of backup storage available during equipment downtime. This site offers nearly three days of emergency storage volume during the peak flow event. This storage capacity reduces permit compliance risk, increases operational flexibility, and minimizes construction costs. Additionally, the property is located in a rural area and is surrounded by a cemetery and agricultural land. There is a 1,000 foot buffer between the pond plant site and the closest residential property, allowing onsite generated odors to dissipate prior to reaching occupants. Further, this site offers the ability to reuse the existing control and lab building, for water quality sampling and electrical Motor Control Center (MCC) and instrumentation housing. Another benefit to the pond plant site is the potential to reuse the current effluent pump station and chlorine disinfection facility.

However, there are disadvantages in using the pond plant site for the new treatment processes. One such drawback to the site is that the ponds must remain in service to allow for continued treatment of sewage during construction or additional construction costs must be incurred to fill in Pond 1 (the settling pond) and provide temporary tanks with enough volume to settle the wastewater. To reduce construction costs, the potential building locations are limited to a small area west of the control building, as the remainder of the site is densely populated with redwood trees or is used as the internal access road. Another complication associated with this site is that it needs to be re-graded to prevent stormwater from flooding the roads. Further, SCWA has an agreement with the owners of the Druid cemetery that the County will pay for the continued maintenance of the ¼ mile long private access road and, therefore, periodic road replacement must be budgeted into the treatment plant operational costs. Lastly, in order to send reclaimed water effluent piping to the new Morelli Lane Reservoir, the existing final effluent pipe will need to be extended through private property. Therefore, the Water Agency must obtain an easement or maintain a working relationship with the Graham's Pond property owners, to ensure smooth installation and maintenance of the effluent pipe alignment.

2.4 SITE SELECTION EVALUATION AND RECOMMENDED DESIGN

Both locations offer a unique set of advantages and disadvantages with respect to housing the new wastewater treatment facilities and none of the shortcomings are insurmountable. However, as described in the sections above, the pump station site has a higher level of complicated limitations in its ability to transform into a fully compliant Title 22 treatment process. Because of these limitations, the pump station site has an expanded project scope to mitigate odor and noise, structurally separate the new processes from the existing concrete basin, and to install a longer effluent pipe alignment and overflow return pump station. The additional scope of work necessary at the pump station site increases the construction costs by approximately $250,000 over the costs of using the pond plant site. Further, there are site safety and access concerns associated with the pump station site that do not exist at the pond plant site.

Based on the extensive list of disadvantages, safety concerns, and additional costs required to use the pump station site, it is recommended to install the new treatment facilities at the pond plant site.

The recommended preliminary site layout is shown in Figure 3-4 and is based on the analysis and recommendations found in Section 3. Note that this layout only includes the improvements to the wastewater treatment processes and does not show the upgrades needed to the supporting facilities (effluent pump station, electrical service and backup power, or solids handling). The additional improvements will be detailed in the next stage of the project during the basis of design report.

The main objectives of the site layout include:

- Maximize use of available space constraints and existing topography.
- Centralize process and electrical facilities for distribution to main electrical loads within the WWTF and locate high electrical use facilities (aeration blowers) near primary electrical switchgear.
- Preserve the use of main access road.

- Retain the use of the existing raw sewage pump station, influent and effluent pipelines, and control/lab building.
- Utilize pond #2 for equalization / emergency storage and pond #1 for solids storage.
- Provide adequate stormwater drainage
- Allow for continued operations of the pond plant during construction of the new facilities.

SECTION 3 SECONDARY WASTEWATER TREATMENT PROCESS SELECTION

3.1 PURPOSE

The OCSD WWTF is required to provide a high level of nitrification (ammonia removal) and denitrification (nitrate removal) in order to meet their surface water discharge permit requirements. Key effluent limitations that govern the process design are monthly average ammonia-nitrogen and nitrate-nitrogen concentrations of 1.2 and 10 mg/L, respectively. Unfortunately, the existing plant uses an aerated pond system that cannot be operated to fully nitrify or denitrify and has resulted in several fines and a Cease and Desist Order (CDO) issued by the Regional Board. Therefore, the treatment plant needs to be upgraded to an activated sludge facility where robust nitrification and denitrification can be achieved.

Based on previous evaluations conducted by SCWA, three preferred secondary nitrification/denitrification treatment process alternatives were identified by SCWA for future consideration. The purpose of this section is to evaluate the three treatment process alternatives: (1) AeroMod, (2) Sequencing Batch Reactor (SBR), and (3) Membrane BioReactor (MBR). The three alternatives would require differing associated plant improvements, including a new headworks, emergency storage basin, a new filtration system (not needed for MBR), and site layout alternatives. The comparative evaluation of the process alternatives includes consideration of all other aspects of the plant that would be impacted by the choice of the biological treatment option.

The remainder of this section is organized into the following sections:

- Secondary Treatment Design Criteria
- Alternative 1: AeroMod
- Alternative 2: Sequencing Batch Reactor (SBR)
- Alternative 3: Membrane Bioreactor (MBR)
- Title 22 Redundancy Requirements and Emergency Storage
- Alternatives Analysis
- Recommended Project

3.2 SECONDARY TREATMENT DESIGN CRITERIA

In order to develop a fair alternatives comparison, it is important to set up common design criteria. In the sections below, secondary influent flows and loads and other common design criteria (including effluent quality) are discussed.

3.2.1 Secondary Influent Flows and Loads

The design flows and loads established for this project are listed in Table 3-1. The secondary treatment flows and loads include preliminary allowances for recycle flows from solids handling facilities, filter backwash (if any), and other process return flows. These preliminary allowances are adequate for this alternative analysis, but will need to be refined for final design of the selected project.

It must be noted that the TKN concentration listed in Table 3-1 are very high and may affect the ability of any process to meet the nitrogen limit. These concentrations are based on very limited historical data as indicated in Section 1. It is strongly recommended to collect more influent TKN data before final design.

Table 3-1 Secondary Process Influent Characteristics

Parameter	Plant Influent	In-Plant Recycle	In-Plant Recycle as a Percent of Influent	With In-Plant Recycle
Flow	Mgal/d	Mgal/d	%	Mgal/d
ADWF	0.025	0.001	5.0	0.026
AAF	0.038	0.002	5.0	0.040
PMF	0.083	0.004	5.0	0.087
PWF	0.115	0.006	5.0	0.121
PDF	0.183	0.005	3.0	0.188
PHF	0.229	0.005	2.0	0.234
Average Loads	lb/d	lb/d	%	lb/d
BOD	139	7	5.0	146
TSS	139	7	5.0	146
TKN	31	0	1.0	31
Peak Month Loads	lb/d	lb/d	%	lb/d
BOD	222	11	5.0	233
TSS	222	11	5.0	233
TKN	49	0	1.0	49
Peak Day Loads	lb/d	lb/d	%	lb/d
BOD	278	0	0.0	278
TSS	278	0	0.0	278
TKN	61	0	0.0	61
Concentrations in mg/L [ADWF with AAL]				
BOD	666			666
TSS	666			666
TKN	147			141
Concentrations in mg/L [ADWF with PML]				
BOD	1,066			1,066
TSS	1,066			1,066
TKN	234			226

3.2.2 Other Common Design Criteria

In addition to flows and loads, other key design parameters are discussed below.

1. **Effluent Requirements**

   The process will be designed to meet a daily maximum ammonia concentration of 2.1 mg N/L and a monthly average nitrate concentration of 10 mg N/L.

2. **Design Temperature**

   It is important to determine the minimum sustained temperature that will exist in the activated sludge process because the maximum growth rate for nitrifiers is reduced at low temperatures. The slower the growth rate of nitrifiers, the larger the reactor volume needs to be to guarantee successful nitrification. The maximum temperature is also important for design because microbial activities increase at high temperatures, resulting in higher rates of oxygen utilization, which impacts the sizing of the aeration equipment.

   Unfortunately, no historical temperature data were available for Occidental. A reasonable minimum design temperature of 14 ^{o}C was selected based on similarly sized wastewater treatment facilities in the region. A maximum process temperature of 25 °C is assumed.

3. **Minimum Sludge Retention Time**

   The selection of the design solids retention time (SRT) or sludge age probably is the most important design decision because it determines the size of the treatment facilities. The aerobic SRT (the average mass of mixed liquor solids maintained in an aerobic environment divided by the solids wasting rate) must be sufficiently long to allow the nitrifying organisms to grow and metabolize virtually all of the available ammonia-nitrogen. The anoxic SRT (the average mass of mixed liquor solids maintained in an anoxic environment divided by the solids wasting rate) must be long enough to attain the required amount of denitrification.

   The required aerobic SRT varies with temperature, desired effluent ammonia concentration, dissolved oxygen concentration (DO), and with the fraction of the total reactor volume that is aerobic (averaged over time). Figure 3-1 shows the theoretical aerobic SRT requirement for a completely mixed reactor as a function of temperature when the DO is 2 mg/L, the effluent ammonia-nitrogen concentration is 1 mg/L, and with different fractions of aerobic to total reactor volumes (or aerobic fraction of time). As shown in Figure 3-1, the theoretical aerobic SRT at the minimum design temperature of 14 °C varies from 6 to 8 days, depending on the percentage of the aerobic to total reactor volume. Assuming an aerobic mass fraction of about 70 percent (to be verified for the selected project), a minimum theoretical aerobic SRT of 6.5 days is indicated. However, this theoretical SRT does not include a factor of safety. To assure reliable nitrification performance under actual field conditions, it is appropriate to apply a safety factor of at least 1.5. Accordingly, a design aerobic SRT of 10 days is selected, resulting in a total SRT of about 14 days (based on 70% aerobic volume). In the final detail design stage of the project, the selected SRTs will be confirmed by running a dynamic model of the new facilities through a BioWin simulation. However, the selected design SRT is considered a very good estimate for the alternatives evaluation.

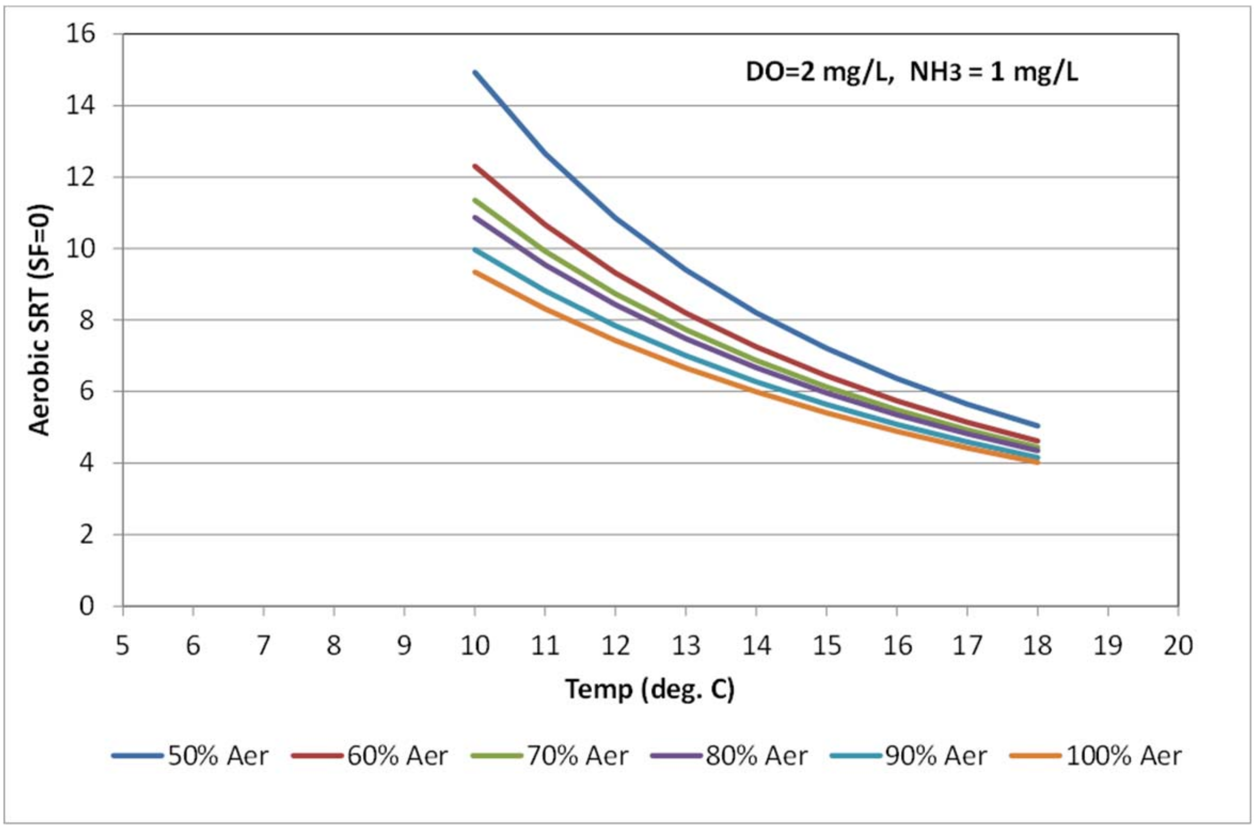


Figure 3-1 Effect of Temperature and Aerobic Volume Fraction on Aerobic SRT

4. **Aeration Basins**

   The sizing of the aeration basins will be based on peak month loads to accommodate the needed sludge inventory for this critical design condition.

5. **Sludge Yield**

   The sludge yield is another important parameter for sizing the aeration basins and is defined as the ratio of the suspended solids produced (in waste activated sludge) to the BOD load removed. Sludge yield is a function of the SRT and the temperature. A sludge yield of 1.0 lb TSS /lb BOD was determined according to the Water Environment Federation Manual of Practice 8 (4^{th} edition, Figure 11.7) at total SRT of 14 days and minimum temperature of 14 °C, and is believed to be conservative.

6. **Aeration Requirements**

   The aeration system shall be designed to provide sufficient oxygen to the process during peak day loading conditions with the largest blower unit out of service.

7. **Design Flexibility**

   Ideally, at least two sets of reactor basins (or trains) should be provided to allow continued operation during times when a basin has to be taken out of service for major maintenance or repairs. It must be noted that all of the volume would be needed at the design condition. Taking a reactor or a train out of service must be scheduled in the summertime when the mixed liquor temperature is high (which would allow adequate treatment at a lower SRT) and the flows are low.

3.2.3 Title 22 Redundancy Requirements and Emergency Storage

Title 22 of the California Code of Regulations (CCR) requires that wastewater treatment plants producing reclaimed water meet reliability requirements for each unit process. Several optional methods of compliance are allowed. For example, one means of compliance is to have more than one method of discharge, such that an effluent that would be non-compliant for reuse could be diverted to a less restrictive discharge alternative. Another method of compliance is to have multiple treatment units in service, such that, even if the largest unit must be taken off-line, the remaining units would be able to meet reuse requirements at the full design flow. Alternatively, there are two reliability options that are based on the use of emergency storage of untreated or partially treated wastewater:

- A short-term emergency storage facility with a capacity to store untreated or partially treated wastewater for at least twenty-four hours and standby equipment that can be used to replace a treatment unit that is taken off-line.

- A long-term emergency storage facility with capacity to store untreated or partially treated wastewater for at least twenty days in case a treatment unit is off-line.

The Title 22 redundancy requirements are different for each considered process alternative, as discussed in the following sections.

3.3 ALTERNATIVE 1: AEROMOD SYSTEM

The Sequox system, a proprietary product manufactured by AeroMod, is an activated sludge treatment process that takes place in one common-wall concrete structure. The concrete structure is subdivided into individual process basins and clarifiers as shown in Figure 3-2. The layout shown includes aerobic digesters, which are optional, and are not part of the secondary treatment system. Therefore, for the purposes of the secondary treatment cost evaluations developed herein, no costs were included for aerobic digesters.

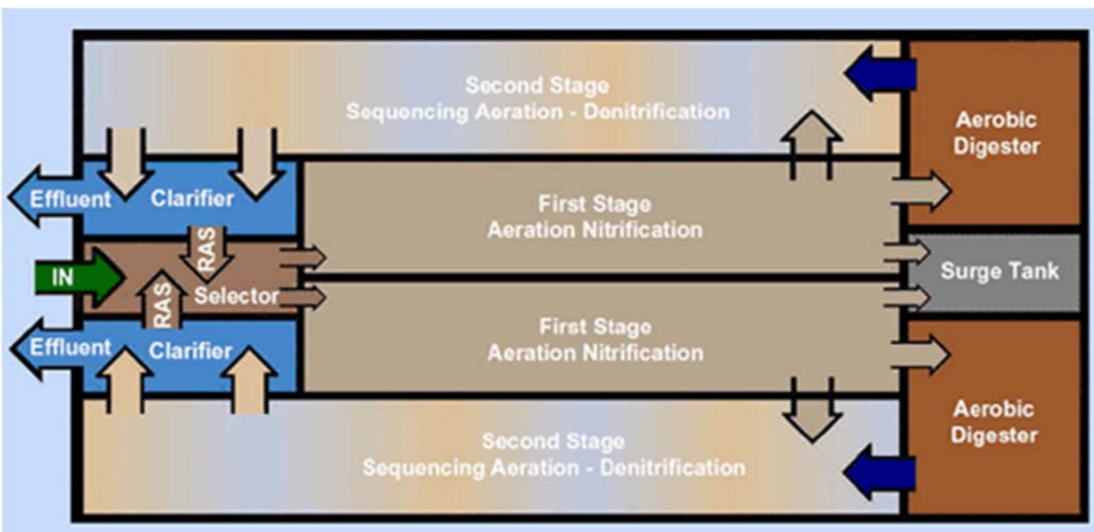


Figure 3-2 AeroMod Typical Layout

3.3.1 Process Description

Screened raw sewage from the headworks enters the anoxic selector tank where it is combined with return activated sludge (RAS) from the secondary clarifiers. The term “selector” refers to its role in selecting activated sludge organisms with excellent settling characteristics. The mixed liquor from the selector is then split into two first-stage and then two second-stage aeration basins where fine bubble aerators are operated intermittently to promote simultaneous nitrification and denitrification.

From the second-stage aeration tanks, the mixed liquor flows into two parallel rectangular secondary clarifier basins fitted with specially designed stationary stainless steel clarification assemblies (proprietary to AeroMod) used to settle and separate the mixed liquor solids from the treated effluent. The solids are returned to the selector using air lift pumps that are integral to the clarifier assemblies, while the clarified effluent overflows to the next process.

At predetermined intervals, mixed liquor is typically wasted from the first stage aeration tanks and sent to the aerobic digesters or other sludge handling facilities. On a daily basis, the amount of waste activated sludge (WAS) should equal solids production. Taking the WAS directly from the aeration basins (rather than from RAS) is the simplest and most accurate method for controlling the sludge age. However, waste from the aeration tank is more dilute than the waste from the RAS, resulting in increased flows to solids handling facilities. If the higher flows of dilute sludge were to be a problem (depends on the type of solids handling facilities), a revised scheme for wasting from RAS could be developed.

3.3.2 AeroMod Design Features

Unit Sizes and Footprint: Based on the AeroMod proposal, the treatment process would include two trains contained in one concrete structure that is 68-ft x 30-ft.

Headworks Screens: Like any wastewater treatment plant, the AeroMod system requires the influent wastewater to be screened. Two 6-mm perforated screens (not included in the AeroMod proposal) will be required. It is assumed that there will be a grade difference in the wastewater treatment site so that wastewater can flow by gravity from the screens to the treatment plant.

Influent Equalization: The AeroMod proposal indicates that peak flows can be treated. At peak hour flow, the secondary clarifier overflow rate is approximately 1,100 gpd/sf, which is acceptable. No influent equalization will be required for the AeroMod process.

Secondary Effluent Filtration: Secondary effluent filtration would be accomplished by two Cloth Media Disk Filter (CMDF) trains. Each train would be capable of filtering the entire peak hour flow, allowing for one completely redundant train.

Title 22 Redundancy Requirements: The AeroMod system consists of two trains: two aeration basins and two clarifiers. The secondary clarifiers do not have motorized mechanical sludge collection mechanisms that can fail. Instead, RAS is collected via air lift pumps powered by the aeration blowers. Since there is a standby blower, a short term emergency storage may not be required. A summary of the AeroMod project components is listed in Table 3-2.

Table 3-2 AeroMod Project Components

Item	Unit	Value
Headworks Screens		
Number of 6-mm Perforated Plate Screens	EA	2
Capacity (Each)	Mgal/d	0.354
Aeration Tank		
Number	EA	2
Volume (Total)	Mgal	0.129
Secondary Clarifiers		
Number	EA	2
Surface Area (Total)	SF	320
Aeration Blowers		
Number of Blowers	EA	2
Horsepower	HP	12
Filtration		
Number of Tank Mounted Filters	EA	2
Capacity (Each)	Mgal/d	0.354

3.4 ALTERNATIVE 2: SEQUENCING BATCH REACTOR (SBR)

The Sequencing Batch Reactor (SBR) system is an activated sludge process designed to operate under non-steady state conditions. The SBR process is a fill and draw process that operates in a true batch mode with aerobic conditions, anoxic conditions, and sludge settlement all occurring in the same tank but at different times. The SBR process eliminates the need for secondary clarifiers, RAS pumping, and internal mixed liquor pumping but adds complexity of the sequence process control and the effluent removal "decant" system.

3.4.1 Process Description

Screened wastewater is introduced into the SBR tank, one batch at a time. Normally, a minimum of two SBR tanks are provided so that one tank receives wastewater while the other tank is processing. Wastewater treatment is achieved in each tank by a timed sequence of operations, consisting of filling, reaction (with aeration and/or mixing), settling, decanting, idling, and sludge wasting. The various stages in the sequence are described further below:

<u>Stage 1: Fill:</u> During fill, the influent valve is opened, allowing raw influent to enter the basin. At the beginning of the fill period, no aeration is allowed to promote anoxic conditions, which discourages the growth of filamentous bacteria and promotes nitrate removal. During the later part of the fill period, the aeration system is turned on to allow BOD oxidation and Simultaneous Nitrification/ Denitrification (SND). The high oxygen uptake creates an aerated anoxic condition where blowers are operated yet residual DO levels remain near zero.

Stage 2: Reaction: Once the SBR tank is filled to its high water level or the designated fill time has been reached, the flow will be diverted to the other SBR tank. Aeration and/or mixing occur in the rector until complete biodegradation of organics has occurred. Since no flow enters the tank during react, no short circuiting of raw, untreated waste can occur. The aeration system can be cycled on / off to help promote denitrification, if needed.

Stage 3: Settling: Following react, the contents of the SBR tank will begin a settle mode in which liquid/solid separation occurs. No influent enters the tank during this period allowing for a perfect quiescent condition. All of the reactor volume/surface area is used for solids separation. The settle period typically lasts for 45 minutes, but is field adjustable.

Stage 4: Decanting: The effluent withdrawal (decant) begins once the settling period is completed. A floating decanter is used to maximize separation between the effluent withdrawal ports and the settled biomass. The decanter is designed to remove effluent from below the water surface to prevent the inclusion of foam, scum, or floatables. Typical systems will have roughly 25%-35% of the tank contents removed from the upper portion of the tank during the decant period.

Stage 5: Idling: During idle, waste activated sludge is typically removed to maintain the correct biomass population in the tank. The aeration and mixing systems typically are not operated during idle, and the tank simply waits for the next fill cycle to begin.

Aqua-Aerobic Systems, Inc., the manufacturer of the Aqua-SBR system, was invited to submit a design proposal for Occidental. Aqua-Aerobic proposed a two-train SBR system. The scope of supply included all mechanical equipment needed for the SBR, plus all equipment needed for post equalization.

3.4.2 SBR Design Features

Unit Sizes and Footprint: Based on Aqua-Aerobic's proposal, the treatment process would include two batch tanks. Each tank would be 25-ft wide x 25-ft long x 17 ft side water depth.

Headworks Screens: The SBR system requires the influent wastewater to be screened. Two 6-mm perforated screens (not included in the SBR proposal) will be required. It is assumed that there will be a grade difference between the SBR and the screen so that wastewater can flow by gravity from the screens to the SBR.

Influent Equalization: The SBR proposal indicates that peak influent flows can be treated. No influent equalization will be required for the SBR process.

SBR Effluent Equalization: Since the SBR is a fill and draw process, the effluent discharge is intermittent, which would affect downstream processes such as tertiary filtration and disinfection. A 15,000 gal effluent equalization is included in the SBR proposal and is accounted for in the cost estimate.

Filter Feed Pump Station: Since the SBR would require effluent equalization, it is unlikely to gravity flow secondary effluent to the filters. Therefore, a filter feed pump station will be required. Two rotary lobe pumps would be required.

Secondary Effluent Filtration: Secondary effluent filtration would be accomplished by two Cloth Media Disk Filter (CMDF) trains. Each train will be capable of filtering the entire flow, allowing for one completely redundant train. Since there will be an effluent equalization, the tertiary filters can be designed for the equalized peak cycle flow instead of the peak hour flow.

Title 22 Redundancy Requirements: The SBR system consists of two trains. Each train has its own dedicated blower, mixer, and decanter. The Aqua-SBR original proposal includes a stand-by blower but no stand-by decanter or mixer. If a decanter is broken, the whole flow cannot be treated in the other train. After further discussions, the manufacturer agreed to provide on-the–shelf spare mixer and decanter. In this case, only short-term emergency storage would be required. However, because the spare equipment is not installed, a larger emergency storage basin would be appropriate to allow time for installing the equipment. The existing 500,000 gallon pond will provide 4.1 days of storage at the peak week flow. It is recommended to line the entire existing Pond 2 and use it for emergency storage.

A summary of the SBR components is listed in Table 3-3.

Table 3-3 SBR Project Components

Item	Unit	Value
Headworks Screens		
Number of 6-mm Perforated Plate Screens	EA	2
Capacity (Each)	Mgal/d	0.354
SBR Tanks		
Number	EA	2
Volume (Total)	Mgal	0.158
Aeration Blowers		
Number of Blowers	EA	2
Horsepower	HP	15
Filtration		
Number of Tank Mounted Filters	EA	2
Capacity (Each)	Mgal/d	0.354
Emergency Storage		
Volume (Total)	Gal	500,000

3.5 ALTERNATIVE 3: MEMBRANE BIOREACTOR (MBR)

The MBR alternative is based on Ovivo's MicroBLOX™ packaged system. The system would be delivered to the site, factory assembled, with little interconnecting piping or ancillary equipment to install (See Figure 3-3).

Figure 3-3 Packaged MicroBLOX MBR System

3.5.1 Process Description

The MBR system includes two fine screens and one biological train with one anoxic tank followed by three membrane/aeration tanks. The tanks are epoxy coated steel tanks with long service life. Each tank can be individually taken out of service while the wastewater process is in operation. The tanks, as well as all the equipment, are skid mounted as shown in Figure 3-3.

3.5.2 MicroBLOX Design Features

Unit Sizes and Footprint: Based on Ovivo's MicroBLOX proposal, the footprint of the treatment process is 45-ft X 8.5-ft.

Headworks Screens: MicroBLOX has two integrated screens but they are not rated for the peak hour flow. Two 2-mm perforated screens (not included in the MicroBLOX proposal) will be added. It is assumed that there will be a grade difference in the wastewater treatment site so that wastewater can flow by gravity from the screens to the treatment plant.

Influent Equalization: The Ovivo MicroBLOX proposal indicates that a peak flow of 175,000 gal/d can be sustained for a week with all three membrane zones in service. To trim the peak day flow of 188,000 gpd to 175,000 gpd, a 13,000 galon volume will be required. To equalize the peak day diurnal flow, about 44,000 gallon will be required, therefore, a total equalization volume of about 57,000 gallon would be needed.

Title 22 Redundancy Requirements: If a membrane unit was taken out of service, one third of the plant capacity is lost. Since the MBR plant hydraulic capacity is 175,000 gal/d, losing one of the three membrane units will reduce the plant capacity to 116,000 gal/d. In the unlikely event that one membrane basin is taken out of service for maintenance during the peak day event, the required emergency storage volume would be 72,000 gal (188,000-116,000). Combining the equalization volume and the emergency storage volume will result in 129,000 gal of required storage, which is less than the available volume in existing Pond 2 (approximately 500,000 gal). It is recommended to build a levy inside Pond 2 to create equalization volume and emergency storage volume.

A summary of the MicroBLOX components is listed in Table 3-4.

Table 3-4 MicroBLOX Project Components

Item	Unit	Value
Headworks Screens		
Number of Fine Screens	EA	2
Capacity (Each)	gpm	150
Equalization Basin		
Number	EA	1
Volume	Gal	57,000
Emergency Storage Basin		
Number	EA	1
Volume	Gal	72,000
Anoxic Tanks		
Number	EA	1
Volume (Total)	Gal	4,000
Membrane Tanks		
Number	EA	3
Volume (Total)	Gal	12,430
Membrane/Aeration Blowers		
Number	EA	3
Horsepower	HP	8.5
Permeate pumps		
Number of Pumps	EA	2
Capacity (Each)	gpm	100
Horsepower	HP	7

3.6 ALTERNATIVES ANALYSIS

In the previous sections, three different processes have been investigated to attain regulatory compliance, while handling the design flows and loads established for the proposed project. In Table 3-5, an overall alternative cost analysis is presented to show the relative costs of the various components for each alternative. Capital, annual, and present worth costs are given. It must be noted that these costs do not include solids handling facilities or shop/office space because they are considered to be similar for all alternatives.

In Table 3-6, the various alternative combinations are rated with respect to several key economic and non-economic criteria, each of which has been assigned an importance weighting factor. Table 3-6 was developed with the input and review of SCWA staff in an effort to assure that the criteria included in the table and the relative weighting factors appropriately reflect the interests and concerns of the District. The criteria, weighting factors and ratings are discussed briefly below.

Life Cycle Cost Costs

Capital and annual costs are used to calculate the life cycle cost of each alternative. Life Cycle cost is assigned a rating factor of 30 percent, meaning, in effect, that 30 percent of the overall decision on which alternative to select is based on life cycle cost. The MBR process has the lowest life cycle cost and was given a rating of 10. The other two alternatives were rate in proportion to their relative cost to MBR alternative.

Robustness and Reliability

Robustness and reliability was assigned a weighting factor of 25 percent. Robustness and reliability represent the degree to which the process is resilient and can perform consistently well, even in problematic conditions, such as influent flow or load spikes, extreme weather, or other challenging biological process conditions. Because the membranes provide an absolute barrier to the escape of particulate matter from the biological treatment system, very consistent performance can be assured. For biological treatment systems that rely on sludge settling (such as AeroMod and SBR), there can be much more variability in effluent quality, which would lead to a higher probability (although still low if properly designed and operated) of potential permit violations. In the case of the OCSD WWTF, the importance of process reliability is amplified because the operational staff are not on site full time and cannot immediately optimize the system during plant upset conditions. The MBR is more resilient and can more readily accommodate challenging conditions, including potential operator error or lack of immediate operator attention, without compromising effluent quality. The MBR alternative is assigned a rating of 10. The SBR alternative is judged to be of similar robustness and reliability as the AeroMod alternative and both were assigned a rating of 7.

Ease of Operation and Maintenance

Ease of Operation and Maintenance (weighting factor of 15 percent) is intended to represent how easy it is to operate the process and to maintain during the life of the equipment. It is also an evaluation of which process takes less operator judgment and has fewer parts requiring maintenance. The AeroMod process was rated higher than the MBR and SBR options because of its design simplicity. The AeroMod process does not have moving parts under water and the only rotating equipment is the blowers; therefore, this alternative is given a rating of 10. The SBR process has more equipment to maintain such as submersible mixers and decanters. It was given a rating of 9. The MBR process has submersible mixers, the membranes, the permeate pumps, the internal recycle pumps. The MBR process was given a rating of 6.

Confidence in Design and Technology

Confidence in design and technology was assigned a weighting factor of 10 percent to reflect the relative importance of this criterion. The SBR alternative was given a rating of 10 because it is an established technology with thousands of plants, by various manufacturers, throughout the world. The AeroMod alternative was assigned a rating of 8 because much of the design is based on proprietary processes. The MBR alternative was also assigned a rating of 8, not because a lack of experience in the MBR technology, but rather because the MicroBlox is a new technology with few plants currently in operation. If the MBR technology proves to be the preferred alternative, more in-depth evaluation would be warranted before final design. Also, other MBR suppliers would be considered during the scheduled request for proposal (RFP) period of the project's preliminary design stage.

Space Requirements

The MBR option would have by far the smallest footprint, resulting in the least disturbance of the natural landscape. As mentioned in Section 1, both treatment plant site alternatives have limited available space and the new processes must fit within the existing boundaries. The MBR alternative is assigned a rating of 10. The other two alternatives are assigned a rating of 8. This criterion was assigned a weighting factor of 7 percent.

Constructability

The MBR is a packaged system that will require minimal connection and concrete work, as all the associated equipment and piping are installed on a skid and shipped to the site. The MBR alternative is assigned a rating of 10. The other two alternatives are assigned a rating of 6, as they require much more field concrete work, field electrical, interconnecting piping to be installed for both the associated equipment and downstream filters. This criterion was assigned a weighting factor of 6 percent.

Adaptability to Future Permit Requirements

The MBR is rated 10 and is higher than the SBR and AeroMod due to improved removals of contaminants that could be of future concerns. This criterion was assigned a weighting factor of 4 percent.

Ease of Future Expansion

Ease of future expansion (weighting factor of 3 percent) is intended to represent how easily additional process basins and equipment could be added to increase capacity. The MBR was rated slightly higher than the other options because it is considered easier to add a package MBR and the associated equipment than to pour concrete basins. Additionally, site availability for future expansion is of less concern for the MBR alternative than for the others, because of its small footprint.

Table 3-5 Alternative Cost Analysis

Item	Unit Cost	Cost, $		
		AEROMOD	SBR	MBR
Capital Cost				
Screens (1)	$	282,000	282,000	321,000
Pumping from Screens to WWTF	$			
24 Hour Emergency Storage Basin (2)	$		119,000	149,000
20 Day Emergency Storage Basin (2.4 Mgal)	$			
WWTF	$	934,000	890,000	976,000
Pumping from WWTF to Filters	$		100,000	
Filters	$	503,000	474,000	
Subtotal 1	**$**	**1,719,000**	**1,865,000**	**1,446,000**
Paving and Grading, 7%	$	120,000	131,000	101,000
Site Piping, 15%	$	258,000	280,000	217,000
Electrical, 25%	$	430,000	466,000	362,000
Subtotal 2	**$**	**2,527,000**	**2,742,000**	**2,126,000**
Contingencies, 25%	$	632,000	686,000	532,000
Construction Cost	**$**	**3,159,000**	**3,428,000**	**2,658,000**
Engineering and Administration, 20%	$	632,000	686,000	532,000
Total Capital Cost	**$**	**3,791,000**	**4,114,000**	**3,190,000**
Annual O&M Costs (3)				
Power Cost (4)	$ / yr	8,230	8,230	15,910
Filter Polymer Consumption (5)	$ / yr	1,644	1,644	
MBR Cleaning	$ / yr			89
Labor Cost	$ / yr	342,000	311,000	328,000
Membrane Replacement	$ / yr			4,000
Total O&M Cost	**$ / yr**	**352,000**	**321,000**	**348,000**
Present Worth Costs				
Present Worth of Annual Cost (5)	$	5,237,000	4,776,000	5,177,000
Total Present Worth (6)	**$**	**9,028,000**	**8,890,000**	**8,367,000**

(1) MBR: 2-mm perforated plate screens. AeroMod and SBR: 6-mm perforated plate screen

(2) Lining the existing 500,000 gal pond. For the MBR alternative, add levy to create equalization basin

(3) Electricity Cost $0.2/kwh. Cost includes power for blowers, screens, and filters

(4) Polymer cost $3.0/lb

(5) 20 years at inflation-adjusted discount rate of 3 percent

(6) Cost does not include certain common elements to all processes, including but not limited to recycled water pipe, R/W, and CEQA.

3.7 RECOMMENDED PROJECT

After consideration of economic and non-economic factors, the MBR process is ranked number 1 (Table 3-6) and is the preferred alternative. The recommended site layout is presented in Figure 3-4.

Table 3-6 Alternatives Evaluation

Criterion	Weighting Factor %	Ratings for Secondary Treatment Alternative [(a)]		
		Aeromod	SBR	MBR
Life Cycle Costs (construction and O&M)	30	9.2	9.3	10
Robustness and Reliability	25	7	7	10
Ease of Operation and Maintenance	15	10	9	6
Confidence In Design and Technology	10	8	10	8
Space Requirements	7	8	8	10
Constructability	6	6	6	10
Adaptability to Future Permits	4	8	8	10
Ease of Future Expansion	3	9	9	10
Overall Weighted Score [(b)]	100	8.32	8.41	9.20
Rank [(c)]		3	2	1

(a) The highest rated alternative is assigned a score of 10. Other alternatives are scored lower, according to the relative concern compared to the highest rated alternative.

(b) Summation of individual ratings multiplied by the corresponding weighting factors.

(c) The alternative with the highest overall weighted score is ranked "1". Other alternatives are ranked "2" through "3", according to overall score.

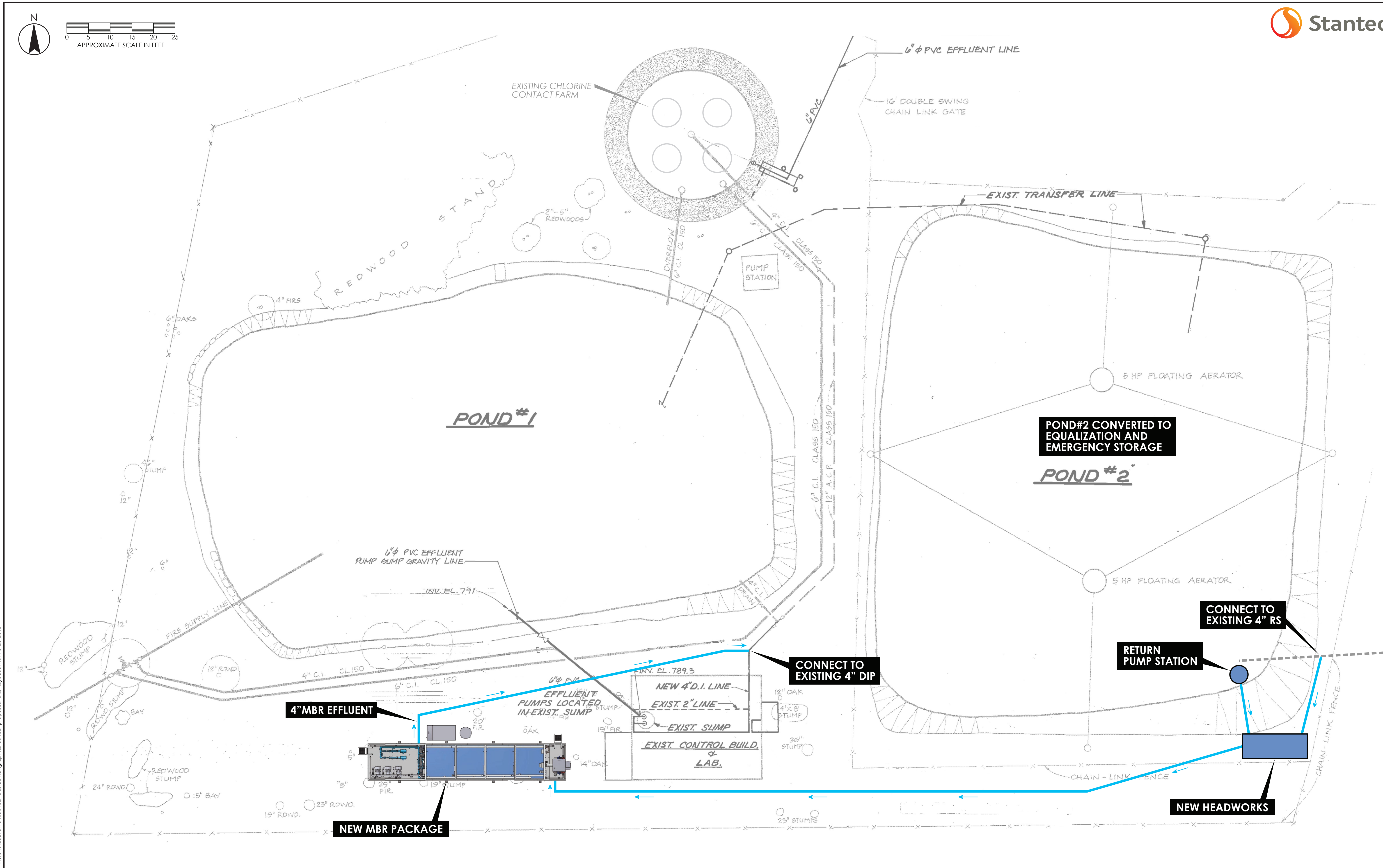


Figure 3-4
Proposed Site Layout

SECTION 4 DISINFECTION SYSTEM ALTERNATIVES ANALYSIS

4.1 PURPOSE

The purpose of this section is to assess the chlorine disinfection process and determine the improvements to the chlorination system and analyze ultraviolet (UV) disinfection as an alternative to the existing process. This evaluation will determine the upgrades needed to disinfect the design flows of 25,000 gal/d (average dry weather flow) and 175,000 gal/d (peak daily flow, equalized, through MBR facility).

4.2 BACKGROUND

Currently the OCSD WWTF disinfects its effluent by mixing its secondary effluent with sodium hypochlorite. Disinfection contact time is provided by series of chlorine contact tanks (5,000 gallons each) that provides approximately 50 minutes of contact time at current maximum monthly flows. After the two contact tanks, sodium bisulfite is injected into the piping manifold and sent through the last contact tanks (3000 gallons) for dechlorinating the effluent prior to discharge.

4.3 CHLORINE DISINFECTION TECHNOLOGY EVALUATION

4.3.1 Chlorine Disinfection Principles

Chlorine is the most widely used disinfectant for municipal wastewater. It is commonly applied as chlorine gas or hypochlorite solutions. Theories that explain the germicidal effect of chlorine include:

- **Oxidation:** Chlorine diffuses into the pathogen's cell and oxidizes its protoplasm.
- **Protein precipitation:** Chlorine precipitates proteins and may change the chemical arrangement of enzymes or inactivates them directly.
- **Modification of cell wall permeability:** Chlorine may destroy the pathogen's cell wall membrane, allowing vital solutes and nutrients, such as nitrogen and phosphorus, to diffuse out of the cell.
- **Hydrolysis:** Chlorine hydrolyzes the pathogen's cell wall polysaccharides. This weakens the cell wall and can lead to its dehydration.

Although the theories mentioned above may all play a part in the destruction of pathogens by chlorine exposure, the primary mechanism depends on the particular type of microorganisms, the chlorine compound (or species) used, the characteristics of the wastewater, and the amount of time that the pathogen is exposed to the disinfectant.

When chlorine gas or hypochlorite salts are added to water, hydrolysis and ionization take place to form hypochlorous acid (HOCl) and hypochlorite ions (OCl-). Free chlorine (a.k.a. free available chlorine) is defined as the concentration of chlorine existing in the form of hypochlorous acid and hypochlorite ions. Free chlorine reacts quickly with ammonia in non-nitrified effluents to form chloramines, principally monochloramine. Free chlorine also reacts with organics in solution to form chloro-organic compounds. Combined chlorine refers to chlorine in the form of chloramines and chloro-organic compounds. The chlorine residual is the total amount of chlorine, both free and combined, remaining after a given contact time.

Some of the advantages of chlorine disinfection include the following:

- Chlorination is a well-established technology.
- The chlorine residual that remains in the wastewater effluent provides disinfection even after initial treatment and can be measured to evaluate the effectiveness.
- Chlorine disinfection is reliable and effective against a wide spectrum of pathogenic organisms.
- Chlorination has flexible dosing control.
- Chlorine can be used for other purposes including odor control, RAS chlorination, etc.

Some of the disadvantages of chlorine disinfection include the following:

- The chlorine residual, even at low concentrations, is toxic to aquatic life and may require dechlorination.
- All forms of chlorine are highly corrosive and toxic. Thus, storage, shipping, and handling pose a risk, which requires increased safety measurements.
- Chlorine oxidizes certain types of organic matter in wastewater, which creates some hazardous compounds (e.g., trihalomethanes [THMs]).
- The level of total dissolved solids (salts) is increased in the treated effluent. In particular, the chloride content of the wastewater is increased.

- Chlorine residuals are unstable in the presence of high concentrations of chlorine-demanding materials. This translates into higher chlorine doses to provide adequate disinfection.
- Some parasitic species have shown resistance to low doses of chlorine, including oocysts of Cryptosporidium parvum, cysts of Entamoeba histolytica and Giardia lamblia, and eggs of parasitic worms.

Chlorine disinfection systems achieve their target degree of disinfection by varying the chlorine dose and the contact time. Chlorine dosage will vary based on chlorine demand, wastewater characteristics, and discharge requirements. The dose usually ranges from 10 to 20 mg/L.

4.3.2 Chlorine Disinfection Design criteria

For the OCSD WWTF, the design for the chlorine disinfection and dechlorination systems will be based on 175,000 gal/d peak flow rate and 25,000 gal/d average dry weather flow rate. The Water Recycling Criteria contained in Title 22 of the California Code of Regulations requires disinfected tertiary recycled water to meet the following criteria:

1. The filtered wastewater has been disinfected by either:

    a. A chlorine disinfection process following filtration that provides a CT (the product of total chlorine residual and modal contact time measured at the same point) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow; or

    b. A disinfection process that, when combined with the filtration process, has been demonstrated to inactivate and/or remove 99.999 percent of the plaque-forming units of F-specific bacteriophage MS2, or polio virus in the wastewater. A virus that is at least as resistant to disinfection as polio virus may be used for purposes of the demonstration.

2. The median concentration of total coliform bacteria measured in the disinfected effluent does not exceed an MPN of 2.2 per 100 milliliters utilizing the bacteriological results of the last 7 days for which analyses have been completed and the number of total coliform bacteria does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30-day period. No sample shall exceed an MPN of 240 total coliform bacteria per 100 milliliters.

4.3.3 Evaluation of Existing Chlorine Disinfection System

Ideally, water entering a basin will travel from the inlet to the outlet in a time equal to the reactor volume divided by the flow rate, which is the theoretical contact time. To minimize short circuiting and to be assured that flow will remain in the basin for the required time, a length to width ratio of at least 20:1 is recommended. Even with ideal conditions, some short-circuiting will still occur and the modal contact time (the time that corresponds to the maximum concentration in a tracer curve for a pulse-input tracer test) will be shorter than the theoretical contact time. In the case of OCSD WWTF, there is no baffling and the length to width ratio is substantially lower than design recommendations, making the assumed ratio of modal to theoretical contact time 0.3 (according to EPA guidance manuals for baffling classifications). Therefore, the theoretical contact time must be at least 90/0.3 = 300 minutes.

Based on Section 3, the improvements project will include an MBR facility with an equalization basin that shaves peak flows to a maximum rate of 175,000 gal/d. Although there will be short-term variations in the instantaneous permeate flow due to membrane relax cycles, the cycle time will be much less than the contact time in the chlorine contact tanks and can be ignored.

In Table 4-1, estimated modal contact times and chlorine doses (based on a CT of 450 mg.min/L) are shown for design dry weather, peak month, and peak day conditions, all based on the existing chlorine contact volumes of 10,000 gallons.

Table 4-1 Evaluation of Existing Chlorine Contact Tanks

	Unit	Design Dry Weather Flow	Design Average Day Peak Month Flow	Design Peak Day (Equalized) Flow
Flow to New Contact Basin	gal/d	25,000	113,000	175,000
Chlorine Contact Basin				
Volume	gal	10,000	10,000	10,000
Theoretical Contact Time	minutes	576	127	82
Estimated Modal Contact Time	minutes	173	38	25
Chlorination				
Required Chlorine Residual	mg/L	2.6	11.8	18.2
CT	mg.min / L	450	450	450

As the table indicates, the existing tank farm can meet Title 22 disinfection requirements. However, during the peak monthly and daily events, the amount of chlorine dose required to adequately disinfect the wastewater becomes quite high.

The estimated 30-year present worth value of the operation and maintenance costs associated with the existing disinfection system is $255,000, based on daily chemical costs, routine pump maintenance, and tank replacement costs (plastic tank life expected to be around 15 years).

4.3.4 Evaluation of Existing Chemical Storage and Feed storage

In order to store 30 days' worth of sodium hypochlorite and bisulfite at the peak month demand during design flow conditions, a 350 gallon tank is required. The hypochlorite and bisulfite pumps must be able to deliver between 1 and 150 gpm of chemical. While no data exists on the recently improved storage and feed system, it is assumed to have the required capacity needed for the project. This will be confirmed during final design.

4.4 UV DISINFECTION TECHNOLOGY EVALUATION

4.4.1 UV Technology Principals

UV disinfection technology is an effective bactericide and virucide for tertiary effluent of wastewater treatment plants. A UV disinfection system transfers electromagnetic energy in the form of UV radiation to the pathogens suspended in the influent water. When UV radiation penetrates the cell wall of a microorganism, it destroys the cell's ability to reproduce by disrupting its genetic material (DNA and RNA). The effectiveness of a UV disinfection system depends on the characteristics of the wastewater, the intensity of UV radiation, the time the microorganisms are exposed to the radiation, and the reactor configuration.

Typically, the main components of a UV disinfection system are mercury arc lamps packed in modules, a reactor, and ballasts. The ballasts provide a starting voltage for the lamps and maintain a continuous current.

The sources of UV radiation are usually either low-pressure or medium-pressure mercury arc lamps. Low-pressure lamps produce low-intensity radiation and medium-pressure lamps produce high-intensity radiation. Medium-pressure lamps, generally used for large facilities, have approximately 15 to 20 times the germicidal UV intensity of low-pressure lamps. The medium-pressure lamps disinfect faster and have greater penetration capability because of their higher intensity. However, these lamps operate at higher temperatures with higher energy consumption.

The optimum wavelength to effectively inactivate microorganisms is in the range of 250 to 270 nm. The intensity of the radiation emitted by the lamp dissipates as the distance from the lamp increases.

Some of the advantages of UV disinfection include the following:

- UV disinfection is effective at inactivating bacteria and most viruses, spores, and cysts.
- UV disinfection is a physical process rather than a chemical disinfectant, which eliminates the need to generate, handle, transport, or store toxic/hazardous or corrosive chemicals.
- There are no residual chemicals that can be harmful to humans or aquatic life.
- UV disinfection systems are usually simple to operate.
- UV disinfection has a shorter contact time when compared with other disinfectants (approximately 20 to 30 seconds with low-pressure lamps).
- UV disinfection equipment requires less space than other methods (e.g., chlorine contact basins), in particular when using in-line UV systems.

Some of the disadvantages of UV disinfection include the following:

- Relatively high energy consumption
- Low dosages may not effectively inactivate some viruses, spores, and cysts.
- UV systems are proprietary.
- Turbidity and total suspended solids (TSS) in the wastewater can render UV disinfection ineffective.
- There is no measurable residual to indicate the efficacy of UV disinfection.

The following definitions[1] are used consistently throughout the remainder of this technical memorandum to describe components of the UV disinfection system:

- **Module:** One or more UV lamps with a common electrical feed.
- **Bank:** One or more UV modules that the entire flow through a reactor train must pass through. Banks are composed of one or more modules.
- **Reactor:** An independent combination of single or multiple bank(s) in series with a common mode of failure (e.g., electrical, cooling, cleaning system, etc.).

---

[1] Extracted from Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, Third Edition, National Water Research Institute (NWRI).

- **Reactor train:** A combination of reactors in series, including inlet, outlet, and level controlling arrangements.

4.4.2 UV Disinfection Design Criteria

For the OCSD WWTF, the design of the UV disinfection system will be based on a maximum (equalized) flow rate of 175,000 gal/d. The UV disinfection system shall be designed to comply with the following Title 22 requirements for unrestricted reuse:

- The UV system, in combination with the membrane filtration system, shall be able to inactivate and/or remove 99.999 percent of the plaque-forming units of F-specific bacteriophage MS2, or polio virus in the wastewater. A virus that is at least as resistant to disinfection as polio virus may be used for purposes of the demonstration.

- The median concentration of total Coliform bacteria measured in the disinfected effluent does not exceed an MPN of 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed and the number of total Coliform bacteria does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30 day period. No sample shall exceed an MPN of 240 total Coliform bacteria per 100 milliliters.

The National Water Research Institute (NWRI) has developed *UV Disinfection Guidelines for Drinking Water and Water Reuse* (last updated in 2012). These guidelines present design constraints for use of UV disinfection on reclaimed wastewater suited for unrestricted reuse. A UV system designed in accordance with NWRI guidelines will provide a level of disinfection that offers the most flexibility for disposal and reuse possibilities.

A summary of UV design criteria consistent with the Title 22 requirements and NWRI guidelines is tabulated in Table 4-2.

Table 4-2 Summary of UV Design Criteria

Design Criteria	Units	Value
Average Dry Weather Flow	gal/d	25,000
Peak Daily Flow	gal/d	175,000
Design UV Does	mJ/cm2	80
Transmittance (at 254 nm)	%	65
Turbidity	NTU	< 0.2

4.4.3 In-Pipe UV Disinfection Systems

Several manufacturers offer low-pressure high-intensity closed vessel reactor, creating an in-pipe UV system that is Title 22 compliant. Three manufacturers were contacted to take part in the evaluation, all of which are currently approved through the Department of Health Services to comply with Title 22 regulations. Based on the proposals received, Trojan Technologies was the cheapest option and their design parameters are presented below.

The disinfection process will consist of two parallel reactors, one duty and one standby, each with 18 ultraviolet lamps. Permeate pumps will pump membrane effluent into the UV system. Each UV reactor train can treat the maximum flow rate, with one train out of service. Pneumatic valves are required to take a UV process train in and out of service to switch between lead and lag reactors. Each train also has a magnetic flow meter that transmits flow to the UV reactors, which is used to adjust the UV intensity to maintain a minimum UV dosage. The UV control panel, provided by the manufacturer, adjusts the UV power based on flow and transmittance. Further, each reactor will be programmed to run approximately the same amount of time in a lead/lag mode, to allow for consistent bulb aging.

Table 4-3 summarizes the design criteria for the UV system.

Table 4-3 UV Disinfection System Design Criteria

Parameter	Value
Manufacturer*	Trojan
Type	Closed vessel
Model	TrojanUVFit, 18AL40
Number of Reactor Trains	2 (one duty, one standby)
Number of Reactors Per Train	1
Number of Lamps per Reactor	18
Design Flow, Mgal/d	0.175
Design UVT, %	65
Design UV Dose, mJ/cm2.	80
Lamp Type	Low Pressure

*The UV manufacturer (Trojan) is identified for the purpose of evaluating relative feasibility. Competitive bidding or preselection process will be allowed if UV disinfection is considered.

The estimated installation cost of the Trojan UV disinfection system is $200,000.

The estimated 30-year present worth value of the operation and maintenance costs associated with the existing disinfection system is $177,000, based on lamp and ballast replacement and energy costs, making the total life cycle costs (for construction and O&M) $377,000.

4.5 ANALYSIS AND RECOMMENDED IMPROVEMENTS

There is a higher potential for the formation chlorine disinfection byproducts, such as dichlorobromomethane (a known carcinogen), when the plant is upgraded to remove nitrogen. Even though the current NPDES permit allows for low levels of disinfection byproducts to be discharged into Dutch Bill Creek, these requirements are likely to become more restrictive in the future, which will make compliance increasingly difficult. Thus, if the Agency wants to maintain the use of the existing surface discharge, for flexibility when land application or reclaimed water storage is not available, the risk of violation should be measured against the added costs of installing a UV disinfection system. When discharging to the reclamation system, the NPDES permit does not require compliance with low levels of disinfection byproducts. The relaxed discharge requirements, associated with land irrigation, allow for the continued use of chlorine disinfection without additional improvements to the existing system because it is capable of complying with the existing permit and Title 22 regulations. Therefore, no significant costs for construction are required for this alternative. However, the present worth life cycle O&M costs associated with the continued use of chlorine and sodium bisulfide are higher than for UV disinfection.

Non-economic factors must be considered when determining whether to upgrade the OCSD WWTF disinfection system, including public perception, protection of human and aquatic life, likelihood of surface water discharge, and potential for future permit violations. Ultimately, SCWA must decide whether the risk of violation during surface water discharge is worth continued operation of the existing chlorine disinfection system. Assuming the effluent can be fully reused and not discharged to Dutch Bill Creek, it is recommended to maintain the existing disinfection system because all associated equipment and tanks were recently installed and will remain compliant with the existing permit. However, when the new chlorine tanks begin to degrade, instead of installing new tanks, it is recommended to upgrade to a UV disinfection system, as this will allow the WWTF more flexibility in discharging final effluent to either the reclamation system or to Dutch Bill Creek.

SECTION 5 SOLIDS HANDLING ALTERNATIVE ANALYSIS

5.1 PURPOSE

The proposed improvements to the OCSD WWTF will produce solids on a daily basis. The purpose of this section is to evaluate different options for solids handling including temporary on site solids storage and daily solids dewatering.

The remainder of this memorandum is organized into the following sections:

- Background
- Alternative Evaluation
- Life Cycle Cost Analysis and Recommended Improvements
- Engineer's Opinion of Probable Construction Costs

5.2 BACKGROUND

Key elements that govern a development and screening of the solids handling alternatives for OCSD WWTF are outlined in the following section.

5.2.1 Existing Facilities

At present, secondary wastewater treatment is accomplished with one aerated treatment pond and one settling pond where all settleable solids accumulate over time. After continued operation over many years, the settling pond has to be drained, and the accumulated solids removed for disposal. With construction of new secondary treatment facilities, the existing ponds will become available for other uses. Since the existing aeration pond is already allocated to be used as an emergency and equalization storage basin, as described in Section No.3, the remaining settling pond is available to be reused for the solids handling facilities. The settling pond design parameters are listed in Table 5-1.

Table 5-1 Existing Settling Pond Design Parameters

Parameter	Value
Water Surface Area, sft	8,460
Side Water Depth, ft	5 to 6
Side Slope	2:1
Liner Type	Clay

5.2.2 Solids Production and Waste Sludge Flows

Average annual and peak month solids production and waste sludge flows for OCSD WWTF were developed by performing solids balances under design loading conditions. The design solids production quantities that are used for the solids handling alternative evaluation and design of proposed solids handling facilities are shown in Table 5-2.

Table 5-2 Solids Production and Waste Sludge Flows

Parameter	36,000 gal/d Average Annual Flow and Load	113,000 gal/d Max Month Flow and Load
Dry Solids, lb/day	144	224
Flow, gal/d	2,660	3,580

5.2.3 Regulatory Requirements

The major regulations that govern the solids handling and disposal for the OCSD WWTF are the applicable waste discharge requirements (WDRs) Order No. R1-2012-0101 issued by the Regional Water Quality Control Board. The current WDRs provide general solids disposal requirements and reference the US EPA Sewage Sludge Regulations 40 CFR Part 503 for land application of biosolids and Criteria for Municipal Solid Waste Landfills 40 CFR Part 258 for disposal of sewage sludge to the municipal solids waste landfills.

As stated within the current NPDES permit, the beneficial use of biosolids by application to land is not covered by the current WDR and any biosolids land application as soil amendment within the North Coast Region would have to comply with State Water Board Water Quality Order No. 2004-12-DWQ (General Waste Discharge Requirements for the Discharge of Biosolids to Land as a Soil Amendment in Agricultural, Silvicultural, Horticultural, and Land Reclamation Activities).

To meet Order No. 2004-12-DWQ and US EPA 40 CFR Part 503 requirements for land application, all biosolids must satisfy limitations for specific metals, and requirements for pathogen and vector attraction reduction and must comply with requirements for Class A or Class B biosolids for land application.

In addition, the current permit requires that solids and sludge treatment, storage, and disposal or reuse shall not cause nuisance including odors and vector attraction, and shall not result in groundwater contamination. Further, runoff from the solids handling facilities should be contained and treated within the site and shall not be allowed discharge into any waters regulated and owned by the State.

5.3 ALTERNATIVE EVALUATION

A typical sludge handling program consists of up to four steps: solids stabilization, solids dewatering, solids drying, and solids disposal. The method of disposal will dictate what must be done in the previous steps. Therefore, it makes sense to identify the applicable disposal options and then to consider stabilization, dewatering, and solids drying alternatives that could be combined with these disposal options.

Based on the current NPDES permit the OCSD WWTF is allowed to discharge their solids in one of the following three methods:

1. Disposal of dewatered sludge to Class III landfill under the following conditions (as specified in Title 27, Chapter 3, CCR for Waste Classification and Management, unless DTSC determines that the waste must be managed as hazardous waste:
    a. The landfill is equipped with a leachate collection and removal system (LCRS);
    b. The sludge contains at least 20 percent solids (by weight) if primary sludge, or at least 15 percent solids if secondary sludge, mixtures of primary and secondary sludges, or water treatment sludge; and
    c. Minimum solids to liquid ratio of 5:1 by weight shall be maintained to ensure that the codisposal will not exceed the initial moisture holding capacity of the nonhazardous solid waste. The actual ratio required by the RWQCB shall be based on site specific conditions.
2. Disposal of biosolids to another appropriately permitted facility. The potential options include Synagro and Waste Management.
3. Disposal to land assuming that OCSD WWTF meets State Water Board Water Quality Order No. 2004-0012-DWQ discharge requirements and has obtained authorization to land apply the biosolids. The above mentioned Order follows the requirements and regulations specified in US EPA Sewage Sludge Regulations 40 CFR Part 503 for land application of sewage sludge.

Considering that the OCSD is not currently permitted to land apply their biosolids the most appropriate alternative for biosolids disposal is either to haul dewatered solids to the closest municipal Class III landfill that accepts wastewater sludge or contract with Synagro or Waste Management. The minimum requirement for any disposal options of solids that do not meet requirements for land application (do not meet Class A or Class B requirements per EPA 40 CFR Part 503) is to produce sludge with a minimum 15% solids content. The life cycle cost evaluation was developed assuming that the sludge will be taken to the landfill operated by Waste Management, located in Livermore. This is the closest Waste Management facility that accepts less than 50% dry sludge.

5.3.1 Solids Stabilization / Storage Lagoons

A lagoon is a lined earthen basin with a depth of 10 to 18 feet. Waste sludge is pumped or drained into the basin and occupies up to about 2 to 4 feet of depth. The liquid above the sludge is typically influent wastewater or treated effluent maintained in an aerobic condition by surface mechanical aerators. In the lagoons with untreated solids, the organics are stabilized by aerobic and anaerobic degradation. The stabilized solids settle to the bottom and accumulate. Lagoon volume is determined to provide a sludge detention time from one to two years, during which a low rate stabilization occurs. Excess liquid from the lagoon is returned to the headworks for further treatment.

Once the lagoon fills, the sludge needs to be removed. Different options are available to remove and treat the biosolids. Some lagoons are dredged in service with solids dewatered using on-site mechanical dewatering facilities or by hiring a dewatering contractor. Others are designed to allow for the solids to dry within the basin. These basins are provided with hard bottoms to allow the equipment access during the summer months to tilt the sludge. Both options are described in more detail below.

Because the existing settling pond has only 5 to 6 ft side water depth it will have to be excavated to minimum water depth of 8 ft and relined. The liner options include:

a. Asphalt liner with HDPE sides. The asphalt liner provides a durable hard surface which allows heavy equipment to drive at the pond bottom during the drying stage of the solids handling. Due to its high installation cost this liner option is more appropriate for larger facilities where hauling and disposal savings exceed capital costs.

b. Lime treated clay. This options provides a hard surface that can be driven on but is less durable than the asphalt liner and has a higher potential to be damaged. As a result, the liner may have to be repaired every second or third time that the pond is emptied. The lime-treated clay liner is approximately 40% less expensive than the asphalt liner

c. HDPE liner. This liner type does not provide a hard surface for heavy equipment and as result the sludge lagoons cannot be used for sludge drying. Instead, the sludge is removed from the lagoon using dredges and has to be dewatered prior being hauled off for final disposal.

A major concern with the sludge lagoons is the potential for odors. As stated in the EPA Process Design Manual for Sludge Treatment and Disposal (625/1-79-011) the sludge lagoons should only be used following an anaerobic stabilization. They cannot function properly (without major environmental impacts) when supplied with either unstabilized or aerobic sludge. In those instances where there is no upstream stabilization of the sludge, odors that are quite unacceptable to the surrounding community are produced. However, despite this warning, there are many successful facultative sludge lagoon installations that do not produce objectionable odors. This can be accomplished with relatively low solids surface loading rates and by providing an aerobic water cap (typically with surface aerators) at least 6 feet over the sludge blanket.

The recommended maximum loading rate for the solids stabilization lagoons to insure that odors are not generated is 20 lbs VSS per 1,000 ft^2 per day. Assuming the waste sludge volatile fraction is 80 percent, the total volatile solids loading at average day maximum month design conditions is 224 x 0.80 = 179 lb/day and the minimum required basin area is 8,950 square feet which roughly equals to surface area of the existing settling pond. Lagoon design will, however, ultimately depend on desired solids retention time and removal method and is analyzed in more detail later in this section.

5.3.1.1 Solids Stabilization Lagoon – Dredging Option

Based on this design, the solids stabilization lagoon mostly serves as a solids storage basin. After the lagoon capacity is reached, i.e. the maximum allowed solids depth is achieved, the solids are removed from the lagoon using a solids pump and a floating dredge. Thereafter the solids can either be treated on site using the mechanical dewatering facility or by the dewatering contractor which is preferred option for the OCSD WWTF due to its small solids production quantities. Because the lagoon can remain in service while the settled sludge is being dredged out, only one lagoon is required for this alternative. Design criteria for the solids stabilization lagoon with dredging option are presented in Table 5-3.

Table 5-3 Design Criteria for Solids Stabilization Lagoon – Dredging Option

Parameter	Value
Number of Basins	1
Surface Area, sft	8,460
Side Water Depth, ft	8
Fill Phase, months, ft	10
Dredging and dewatering Phase, months	1
Solids Loading, lb VSS/day-1000 ft^2	20
Liner	HDPE

The solids stabilization/storage lagoon includes feed piping, overflow/decant piping, and existing aerators relocated from the aeration pond. Biosolids are typically fed to the basins regularly. During the 1 year accumulation period, biosolids undergo low rate digestion which results in a reduction of solids volume. It is assumed that only 60% of the volatile biosolids remains after the digestion period, thus reducing the dewatering, hauling and disposal requirements. When the solids accumulate at the bottom with a maximum sludge depth up to two feet, the sludge is pumped out and dewatered using a mechanical dewatering unit. A dredge and dewatering equipment would be rented from a local supplier.

5.3.1.2 Solids Stabilization Lagoon – Drying Option

Solids are fed to the storage/stabilization lagoon for approximately 12 to 24 months and then the lagoon is rested for the 3 month during which drying occurs. Based on this design a minimum of two basins are required. Due to a compact site and relatively shallow existing settling pond the only option to divide pond into two basins is to construct a vertical concrete wall. In addition, the existing settling pond would have to be deepened to at least 8 ft side water depth to allow for minimum of 6 ft of water cap and provide a sufficient storage volume for the sludge produced in 12 month period. With the pond being so deep there would not be adequate area to maneuver heavy equipment that is usually employed to tilt solids during a drying phase. For these reasons the solids storage/stabilization lagoon with drying option is not good fit for the OCSD WWTF and will not be evaluated further.

5.3.2 Solids Dewatering using Dewatering Tube or Dewatering Box

An alternative to solids stabilization/storage lagoon is solids dewatering. Because of small flows and solids production rates at OCSD WWTF, mechanical dewatering equipment such as screw press, fan press, belt filter press, or centrifuges would not be cost effective alternatives and were not evaluated. Typically, wastewater treatment facilities with flows comparable to the Occidental WWTF use geotextile tubes or dewatering boxes. Both systems operate on a same principle; the waste sludge from secondary process is conditioned with polymer and pumped directly into the tube or dewatering box. The permeable material used to construct the tube or is placed within the box allows excess water to seep out while retaining the solids and fine particles.

The main difference between a dewatering tube and a dewatering box is that the dewatering box comes permanently installed within a roll-up container whereas the dewatering tube can be placed in any 20-yard or 30-yard standardized container or on a drying bed. When the drying box gets filled with dewatered sludge either a second dewatering box needs to be provided or sludge wasting has to be temporarily stopped until the box is emptied. On the other hand, when the container with the dewatering tube fills up with sludge, the container can be removed and replaced with an empty container. The new tube is then installed in an empty container and reconnected to the sludge feed line. If the tube is installed within the drying bed, when full, the bag is cut open and sludge is loaded into the trucks and hauled off.

Because containers can be easily rented as part of disposal contract the dewatering tube has a lower capital cost. On the other hand, the tubes cannot be reused unlike it is the case with the dewatering boxes, which results in higher O&M cost for the dewatering tube. Options with both, the dewatering tube and the dewatering box, are evaluated further in the remainder of this report. The dewatering tube is evaluated for the option with a tube within the container and option with a tube placed within the existing settling pond.

5.3.2.1 Dewatering Tube within Container

Capital improvements required for this alternative include a polymer blending unit and 25 ft x 10 ft concrete pad. The dewatering tube is simply placed into a 20-yard watertight roll-off container which can be either rented or purchased. The container is placed on top of the concrete pad sloped to drain by gravity into a sump and further into the return pump station. The WAS discharge from the MBR process is first conditioned with polymer and then fed into the dewatering tube. As the tube fills with wet sludge, excess water drains through the tube fabric into watertight container. From container water flows onto the concrete pad or into drainage pipe connected to the container drainage outlet from where it further conveyed into the return pump station. The 20-yard tube would have to be replaced approximately once every month. The dewatering tube within roll-off container is shown on Figures 5-1 and 5-2.

Figure 5-1 Dewatering Tube within a Container - Schematic

Figure 5-2 Dewatering Tube within a Container

5.3.2.2 Dewatering Tube within the Existing Settling Basin

Another option is to place one 60-ft or two 45-ft dewatering tubes within the existing settling pond. To minimize vector attraction due to standing water in the pond, the pond bottom would have to be covered with a minimum 6" of drainage rock wrapped into the geotextile fabric. As the tube is filled with conditioned wet sludge, water seeps through tube fabric and into the drainage rock. At one end of the pond a decant liquid is collected into a drainage pipe and sent into the return pump station from where it is pumped back into the process. This option also requires polymer blending unit. Assuming that two 45-ft tubes would be used, the replacement frequency for both tubes is once every three years. The dewatering tube on a sludge drying bed is shown on Figures 5-3.

Figure 5-3 Dewatering Tube on a Sludge Drying Bed

5.3.2.3 Dewatering Box

Similar to the dewatering tube with the container, the dewatering box would be placed on a 25 ft x 10 ft concrete pad which drains by gravity to the return pump station. Once the box fills with sludge it is disconnected from the feed line, hauled off site, and emptied at the landfill. The proposed 15-yard dewatering box would have to be emptied approximately once every 20 days. The sludge fed into the dewatering box also requires polymer conditioning which is accomplished by adding polymer to the sludge feed line. The recommended system for OCSD WWTF is a dewatering box system which includes 15-yard dewatering box, polymer blending unit, and aluminum working platform. The dewatering box is shown on Figure 5-4.

Figure 5-4 Dewatering Box

5.4 LIFE CYCLE COST ANALYSIS AND RECOMMENDED IMPROVEMENTS

The four feasible solids handling alternatives include:

1. Solids stabilization / storage lagoon with dredging option
2. Solids dewatering using dewatering tube within the container
3. Solids dewatering using dewatering tube in the existing settling pond
4. Solids dewatering using dewatering box

Cost analysis for these four alternatives is summarized in Table 5-4.

As indicated, the alternative with the lowest present worth cost is the dewatering bag placed into the existing settling basin which has capital cost of about $121,000 and a total present worth cost of $557,000. The alternative with lowest capital cost is with the dewatering bag placed in the container with capital cost of $79,000 and total present worth cost of $640,000.

Table 5-4 Solids Handling Alternatives Life-Cycle Cost Analysis

Item	Solids Storage / Stabilization Lagoon - Dredging Option	Dewatering Bag with Container	Dewatering Bag within Existing Settling Pond	Dewatering Box
Base Construction Costs[a]				
Existing Settling Pond Improvements	$55,000	$0	$29,000	$0
Solids Handling Failities	$17,000	$31,000	$24,000	$93,000
Site Piping	$13,000	$13,000	$14,000	$13,000
Subtotal 1	$85,000	$44,000	$67,000	$106,000
Markups[i]	$68,000	$35,000	$54,000	$85,000
TOTAL PROJECT COST				
Subtotal	**$153,000**	**$79,000**	**$121,000**	**$191,000**
Annual O&M Costs at Design Condition				
Electrical[c]	$3,266	$0	$0	$0
Chemical[d]	$0	$2,186	$2,186	$2,186
Equipment Rental[e]	$32,000	$6,000	$0	$0
Labor[f]	$8,580	$6,240	$6,500	$9,360
Maintenance and Materials Replacement	$5,000	$7,500	$2,067	$1,500
DISPOSAL[g]	$19,267	$28,334	$28,334	$28,334
TOTAL O&M COST				
Annual O&M Cost	**$68,100**	**$50,300**	**$39,100**	**$41,400**
Total Present Worth 20-Year O&M Costs adjusted for lower initial residuals production[b] [h]	**$760,000**	**$561,000**	**$436,000**	**$462,000**
TOTAL Present Worth Costs - Residuals Handling Alternatives	**$ 913,000**	**$ 640,000**	**$ 557,000**	**$ 653,000**

Assumptions

(a) Cost estimate is based on 20-Cities ENR of 10,500

(b) Present worth O&M is calculated assuming 3% annual inflation

(c) Electrical power costs are $0.15 per kW-hr.

(d) Polymer cost is estimated at $3.50 per lb active.

(e) Equipment Rental includes annual fees to rent container, dredge, or dewatering equiment

(f) Labor cost is estimated at $65 per person-hour.

(g) Assumed that all the solids will be disposed at Livermore WM facility at fee of 125$/wet ton. Assumed transportation fee is 50$/wet ton.

(h) Annual O&M cost is estimated at 75% of annual O&M cost at buildout.

(i) Markups include 20% for General Conditions, Overhead, and Profit, 25% for Contingency, and 20% for Engineering and Administration

The main disadvantage of the option with the dewatering tube within the settling pond compared to the tube in the container is its potential to attract vectors. Even though the pond bottom will be covered with a minimum of 6-inches of drainage rock wrapped into the geotextile fabric, water that drains out of the bag may attract flies and mosquitos. On the other hand, the tube within the container can be covered to reduce odors and vector attraction. Another disadvantage of the tube within the pond is that, once the tube is full, it has to be opened on site. Because the sludge collected in the bag is not properly stabilized it may create odors and become nuisance to the plant operators and neighbors.

For these reasons even though the dewatering tube within the pond has lower present worth cost, the alternative with tube placed within the container is recommended for OCSD WWTF. The recommended solids handling improvements for OCSD WWTF are:

- Construct 10 ft x 25 ft concrete drainage pad for 20-yard container
- Provide a new polymer dilution system.
- Rent water-tight 20-yard container.
- Purchase one-year supply of dewatering bags.
- Provide piping for solids feed and decant drainage.

5.5 OPINION OF PROBABLE CONSTRUCTION COSTS

Table 5-5 shows the engineer's opinion of probable construction costs for the recommended Improvements.

Table 5-5 Opinion of Probable Construction Costs for Solids Handling Improvements [(a)]

Item	Cost [(b)]
Solids Handling Improvements	31,000
Yard Piping	13,000
One year 20-yard Container Rent Fee	6,000
Dewatering Tube – One Year Supply	7,500
Total Base Construction Cost	**57,500**

(a) Not including general conditions, overhead, profit, contingencies, engineering, and administration.

(b) Estimated at Mid Construction cost level, ENR 20-Cities CCI = 10,500.