



LOS ANGELES COUNTY
SANITATION DISTRICTS
Converting Waste Into Resources

Joint Outfall System 2050 Master Facilities Plan Long Beach Water Reclamation Plant

TECHNICAL MEMORANDA • JANUARY 2025





**LOS ANGELES COUNTY
SANITATION DISTRICTS**
Converting Waste Into Resources

Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 1
**LONG BEACH WATER RECLAMATION
PLANT POPULATION, FLOW, AND
LOADS**

FINAL | January 2025



**LOS ANGELES COUNTY
SANITATION DISTRICTS**
Converting Waste Into Resources

Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 1
LONG BEACH WATER RECLAMATION PLANT
POPULATION, FLOW, AND LOADS

FINAL | January 2025

Contents

Technical Memorandum 1 - Long Beach Water Reclamation Plant Population, Flow, and Loads

1.1 Introduction	1-1
1.1.1 Purpose	1-1
1.1.2 Background	1-1
1.1.3 Definitions	1-4
1.2 Population Projection	1-5
1.2.1 Historical Population	1-5
1.2.2 Population Projection	1-8
1.3 Historical Flow and Loading	1-11
1.3.1 Influent Sources to LBWRP	1-11
1.3.2 Historical Flow and Loading	1-11
1.3.3 Historical Flows to LBWRP	1-12
1.4 Flow and Load Projection	1-15
1.4.1 Approach	1-15
1.4.2 Per Capita Generation Rate (PCGR)	1-15
1.4.3 Residential, Commercial, and Industrial Flows	1-16
1.4.4 DWR Indoor Residential Water Use Study (IRWUS) Goals	1-16
1.4.5 Projected PCGR	1-17
1.5 Flow and Load Projections	1-17
1.6 Summary	1-19
1.7 References	1-21

Tables

Table 1.1	Long Beach Design Flows	1-2
Table 1.2	Wastewater Flow and Load Definitions	1-4
Table 1.3	Tributary Area Names	1-6
Table 1.4	Tributary Area Relationships	1-7
Table 1.5	Historical Population for LBWRP Tributary Area	1-7
Table 1.6	Annual Population Growth Rate 1970 - 2045	1-8
Table 1.7	Projected Population by WRP Tributary Area	1-8

Table 1.8	Projected Population for LBWRP Tributary Area	1-9
Table 1.9	Historical Flows to LBWRP Summary (Per Year) in mgd	1-12
Table 1.10	Historical Flow Peaking Factors at LBWRP Summary (Per Year)	1-13
Table 1.11	Historical Influent LBWRP BOD5 Load Summary - Influent	1-14
Table 1.12	Historical Influent LBWRP COD Load Summary - Influent	1-14
Table 1.13	Historical Influent LBWRP TSS Load Summary - Influent	1-14
Table 1.14	Historical IW Loads to LBWRP	1-15
Table 1.15	JOS Districts' Sewage Units	1-16
Table 1.16	California State Standards and DWR Recommended Standards	1-17
Table 1.17	Current and Projected PCGR	1-17
Table 1.18	Flow and Load Projection for LBWRP	1-18

Figures

Figure 1.1	Aerial Image of LBWRP	1-2
Figure 1.2	LBWRP Tributary Area	1-3
Figure 1.3	Long Beach Process Flow Diagram	1-4
Figure 1.4	WRP Tributary Map	1-6
Figure 1.5	Historical Population for LBWRP Tributary Area	1-8
Figure 1.6	Projected Population for LBWRP Tributary Area	1-10
Figure 1.7	Historical Flow Trend at LBWRP	1-13
Figure 1.8	Flow Projections for LBWRP Tributary Area	1-19
Figure 1.9	BOD ₅ Loading Projections	1-19
Figure 1.10	COD Loading Projections	1-20
Figure 1.11	TSS Loading Projections	1-20

Abbreviations

AMEL	average month effluent limit
Basin Plan	Los Angeles Region Water Quality Control Plan
cfs	cubic feet per second
DDW	Division of Drinking Water
EPA	U.S. Environmental Protection Agency
F	Fahrenheit
gpm/ft ²	gallons per minute per square foot
GWR	groundwater recharge
IPR	indirect potable reuse
JOS	Joint Outfall System
LACDPW	Los Angeles County Department of Public Works
LACSD	Los Angeles County Sanitation Districts
lbs/day	pounds per day
LBWRP	Long Beach Water Reclamation Plant
LCWRP	Los Coyotes Water Reclamation Plant
MFPs	Master Facilities Plans
mgd	million gallons per day
NPR	non-potable reuse
NPDES	National Pollutant Discharge Elimination System
POTW	Publicly Owned Treatment Work
POWRP	Pomona Water Reclamation Plant
Regional Water Board	Regional Water Quality Control Board
RO	reverse osmosis
SDWA	Safe Drinking Water Act
SJCWRP	San Jose Creek Water Reclamation Plant
SCCWRP	Southern California Coastal Water Research Project
State Water Board	State Water Resources Control Board
TM	technical memorandum
Warren Facility or WWRF	A.K. Warren Water Resource Facility
WDR	waste discharge requirement
WNWRP	Whittier Narrows Water Reclamation Plant
WRD	Water Replenishment District of Southern California
WRP	water reclamation plant
WRR	Water Reclamation (or Recycling) Requirement
Vander Lans WTF	Leo Vander Lans Water Treatment Facility

Technical Memorandum 1

LONG BEACH WATER RECLAMATION PLANT POPULATION, FLOW, AND LOADS

1.1 Introduction

The Los Angeles County Sanitation Districts (Districts) are a confederation of 24 independent special districts serving approximately 5.6 million people in Los Angeles County (County). The Districts' service area covers approximately 850 square miles and encompasses 78 cities and unincorporated territory within the County. The Districts construct, operate, and maintain facilities to convey, treat, recycle, and dispose of sewage and industrial wastes and generate recycled water, electrical power and biosolids as products of the treatment process.

The Districts operate 10 water reclamation plants (WRPs) and the A.K. Warren Water Resource Facility (Warren Facility). Seventeen sanitation districts provide sewerage services in the metropolitan Los Angeles area and are signatory to a Joint Outfall Agreement that provides for the regional, interconnected systems of facilities known as the Joint Outfall System (JOS). The service area of the JOS encompasses 73 cities and unincorporated territory. Six of the ten WRPs plus the Warren Facility are within the JOS. The WRPs within the JOS include: La Cañada (LaCa) WRP, Long Beach (LB) WRP, Los Coyotes (LC) WRP, Pomona (PO) WRP, San Jose Creek (SJC) WRP, and Whittier Narrows (WN) WRP.

The Districts are preparing a Master Facilities Plan (MFP) in phases for (1) the SJCWRP, (2a) the LBWRP and LCWRP, and (2b) the POWRP and WNWRP. The MFP for the SJCWRP was prepared during Phase 1. The MFP for the LBWRP was prepared during Phase 2a. Preparation of a MFP for the remaining WRPs will be completed during Phases 2a and 2b, utilizing the process developed during Phase 1.

1.1.1 Purpose

The purpose of this Technical Memorandum (TM) is to provide population, flow, and loading projections for the LBWRP tributary area. The population projections were used to derive a Per Capita Generation Rate (PCGR) for wastewater flow and produce flow estimates and wastewater quality characteristics projections.

1.1.2 Background

The Long Beach WRP is located at 7400 E. Willow Street in the city of Long Beach. It is bounded by Willow Street to the north, Coyote Creek to the south and east, and the San Gabriel River to the west. Land uses surrounding the plant include the Water Replenishment District's Leo J. Vander Lans Advanced Water Treatment Facility (AWTF) and El Dorado Park to the north, El Dorado Park Golf Course to the west, and residential areas to the south and east. The permitted capacity of the Long Beach WRP is 25.0 mgd. Table 1.1 lists the plant design flows from the tributary sewers at LBWRP. Figure 1.1 is an aerial overview of the Long Beach WRP. Figure 1.2 presents the LBWRP tributary area.

Table 1.1 Long Beach Design Flows

Plant Flows	
Daily Average Flow	25 mgd
Daily Minimum Plant Flow (Factor = 0.24)	6 mgd
Daily Maximum Peak Sanitary Flow (Factor = 1.36)	34 mgd
Daily Maximum Peak Storm Flow (Factor = 2.4)	60 mgd



Figure 1.1 Aerial Image of LBWRP

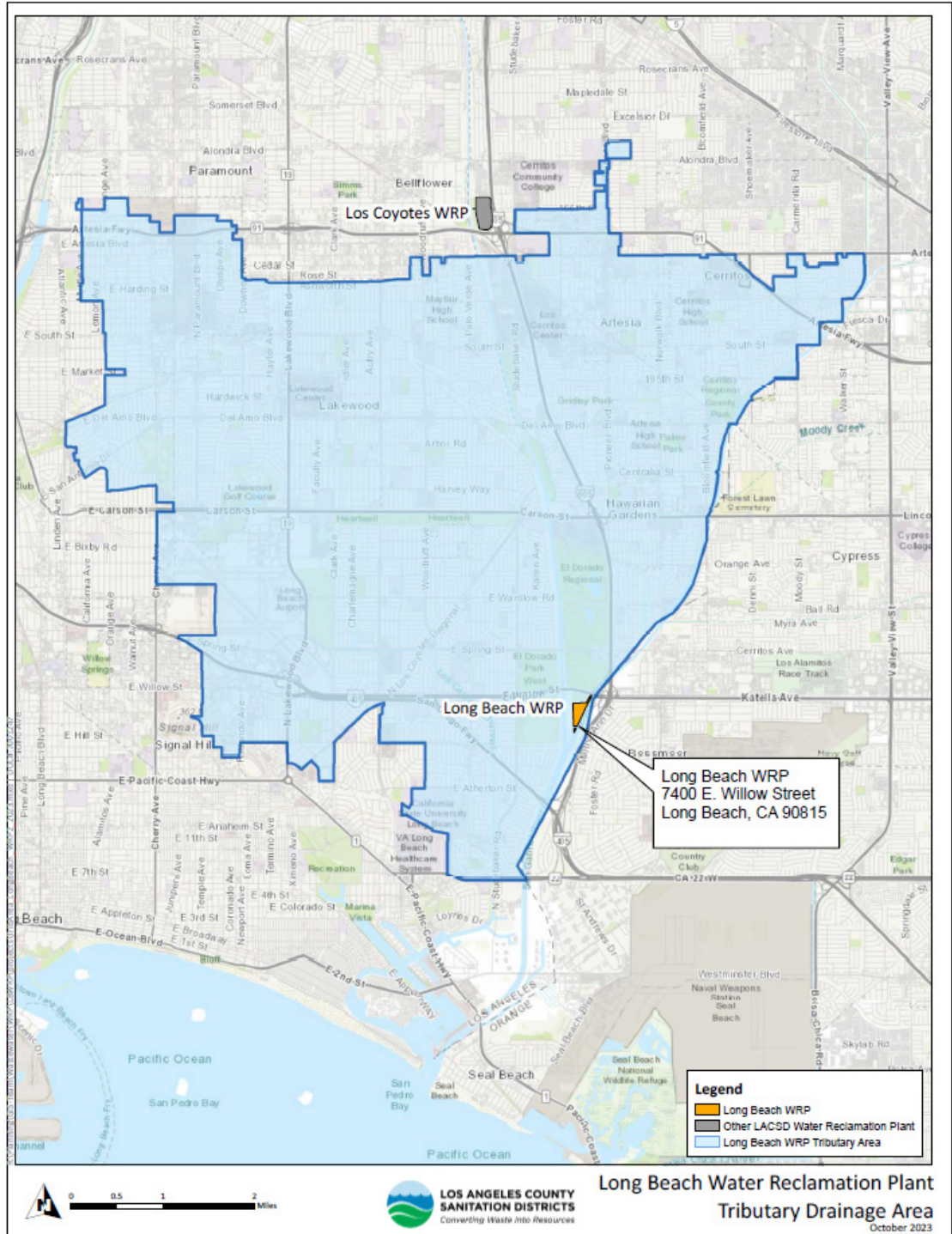


Figure 1.2 LBWRP Tributary Area

The Long Beach WRP consists of two treatment modules, each having a capacity of 12.5 mgd. Primary, secondary, and tertiary treatment are provided with all solids conveyed to Warren Facility for processing (Figure 1.3).

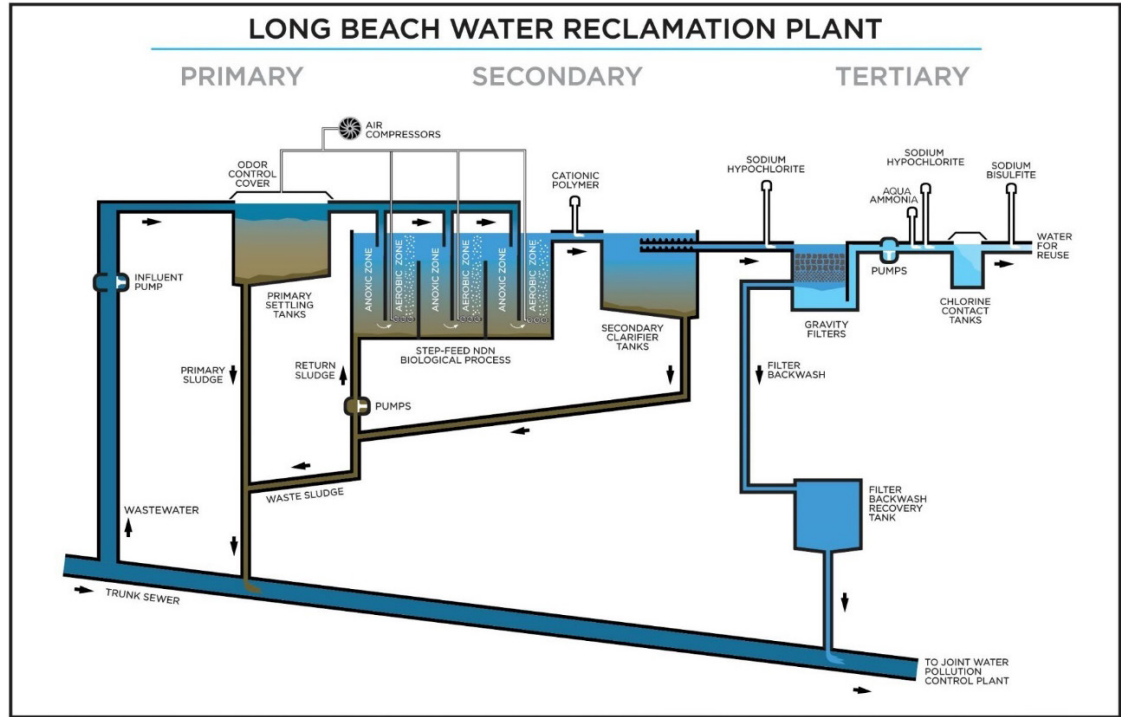


Figure 1.3 Long Beach Process Flow Diagram

An annual average of approximately 7.2 mgd of recycled water is beneficially reused at over 119 sites including landscape irrigation, re-pressurization of oil-bearing strata off the coast of Long Beach, groundwater replenishment, and toilet flushing. A portion of Long Beach WRP effluent is further processed at the Leo J. Vander Lans Advanced Water Treatment Facility where it is treated with microfiltration (MF), reverse osmosis (RO), and ultraviolet (UV) disinfection prior to injection into the Alamitos Seawater Barrier. The remaining effluent is discharged into the concrete-lined portion of Coyote Creek, about 2,200 feet upstream from its confluence with the San Gabriel River. The San Gabriel River is lined from Coyote Creek confluence to the river estuary.

1.1.3 Definitions

Although the permitted (NPDES) capacity of the LBWRP is determined based on the average dry weather flow (ADWF) of the plant, the plant must be able to handle daily and seasonal variations in influent flow and load. In fact, capacity limitations for various processes are often observed during the peak flow and load conditions, and not the ADWF. For reference, Table 1.2 summarizes the definitions of various flow and load conditions and their relevance to wastewater planning.

Table 1.2 Wastewater Flow and Load Definitions

Term	Definition	Purpose
ADWF	Average Dry Weather Flow The average flow occurring during the dry season, defined as the average flow occurring between the months of May and October.	Determining permitted capacity of the WRP and scheduling unit process downtime

Term	Definition	Purpose
ADWL	Average Dry Weather Load The average load occurring during the dry season, defined as the 90-day average load that coincides with the ADWF time period.	Scheduling unit process downtime and sizing of wastewater treatment facilities
AAF or AAL	Average Annual Flow or Load The average flow or load occurring over the course of the year.	Performing economic analysis of alternatives
ADMMF	Average Daily Maximum Month Flow The average daily flow occurring during the maximum flow month of the year. This is calculated as the maximum 30-day average for the year.	Sizing of secondary treatment and flow equalization facilities
ADMML	Average Daily Maximum Month Load The average daily organic or suspended solids load occurring during the maximum load month of the year. This is calculated as the maximum 30-day average for the year. The maximum monthly load does not necessarily occur during the same period as the maximum monthly flow.	Sizing of secondary treatment facilities
MDF	Maximum Day Flow The maximum single day flow or load recorded for the year.	Sizing of hydraulic facilities, secondary treatment facilities, process aeration, and biological nutrient removal facilities
PWWF	Peak Wet Weather Flow The peak wet weather inflow or load resulting from a rainfall event.	Sizing of hydraulic facilities, specifically the headworks

1.2 Population Projection

1.2.1 Historical Population

In 2008, CH2MHill and MWH performed 2000 to 2007 population counts as part of the Clearwater Program Master Facilities Plan. Population was distributed by City by the California Department of Finance (DoF) and by Census Tract by the Southern California Association of Governments (SCAG). Populations were allocated to individual parcels, based on each parcel's land use type as defined in the Los Angeles County Assessor's Roll. Further details can be found in Technical Memorandum 2-2 "Population Projections and Service Area Evaluations" (CH2MHill, MWH 2008).

The JOS population by tributary areas from 2010 to 2020 is derived from the DoF. The DoF provides population estimates for each city annually. The populations are allocated to individual residential parcels where each parcel receives an average population value, for example, 3.21 people per Single Family Residence (SFR). The parcel populations are then summarized by WRP tributary area.

In order to generate population projections for any portion of the Districts' JOS tributary area, population data must be distributed based on tributary area. This was done using the Simplified Node Model from the Clearwater Program Master Facilities Plan Technical Memorandum 2-3 "Wastewater Flows and Characteristics" (CH2MHill, MWH 2008), which takes into consideration

flow splits and plant bypasses. Figure 1.4 depicts the tributary areas for the individual WRPs. Six of ten JOS tributary areas can contribute flows to multiple plants. Tributary names are provided in Table 1.3.

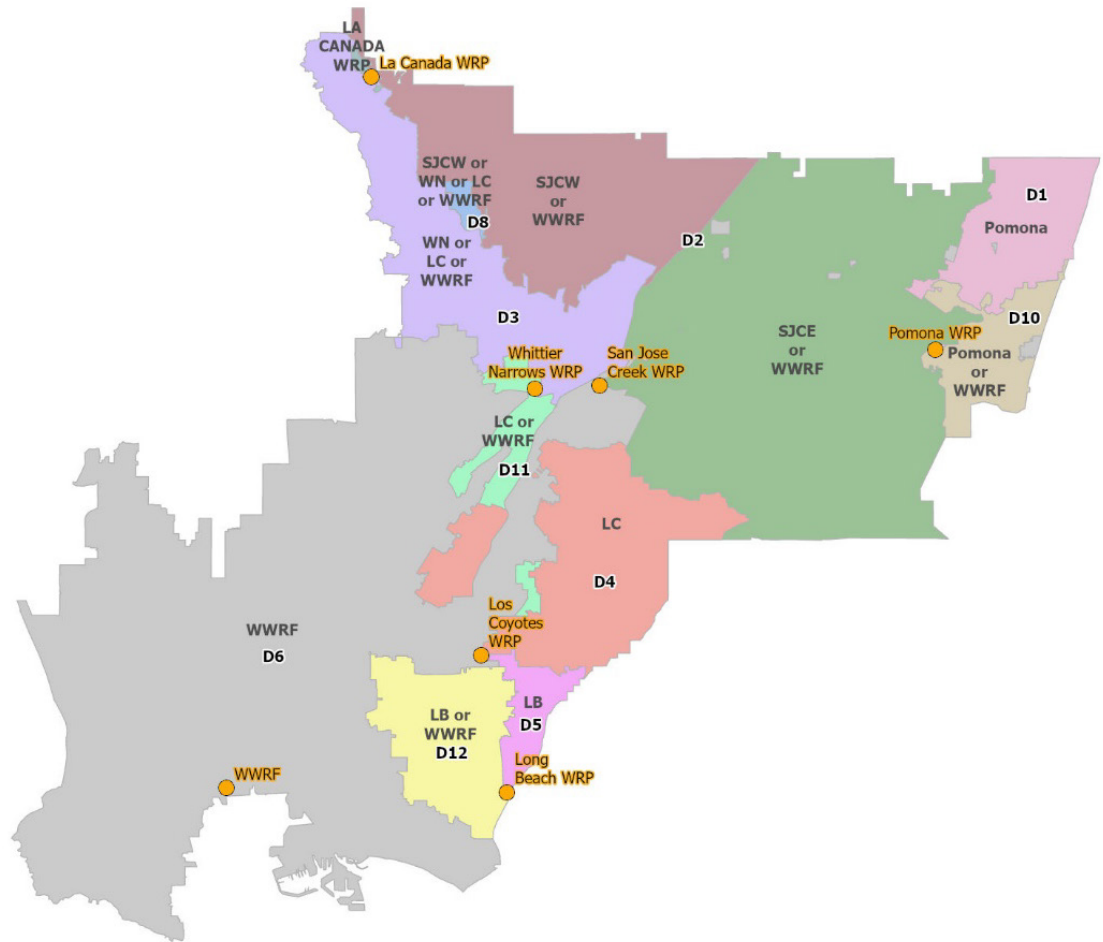


Figure 1.4 WRP Tributary Map

Table 1.3 Tributary Area Names

Node	Name
D1	Pomona
D2	SJC or Warren Facility
D3	WN or LC or Warren Facility
D4	LC
D5	LB
D6	Warren Facility
D8	SJC or WN or LC or Warren Facility
D10	Pomona or Warren Facility
D11	LC or Warren Facility
D12	LB or Warren Facility

Table 1.4 Tributary Area Relationships

Plant	Relationship to Tributary Areas
Pomona	D1 + 0.07 D10
SJC	0.99 D2 + 0.88 D8
WN	0.946 D3 + 0.12 D8
LC	D4 + 0.005 D3 + 0.38 D11 + 0.1 WN Bypass
LB	D5 + 0.91 D12
Warren Facility	D6 + 0.049 D3 + 0.01 D2 + 0.09 D12 + 0.62 D11 + 0.93 D10 + Upstream WRP Bypass (but only 90% WN bypass)

Table 1.4 shows the link between designated tributary area and WRP as of January 2008. The relationships for the LBWRP were re-evaluated as part of this effort. The relationships are used to allocate contributing populations to the correct WRPs. Mapping tributary areas to specific treatment plants is key to accurately estimating the populations and flows affecting each WRP.

The historical population per year for the LBWRP tributary area were calculated using the $D5 + 0.91 D12$ relationship, and are presented in Table 1.5 and Figure 1.5. There were refinements in the population estimation methodology starting in 2017 that led to a seven percent reduction in the estimated LBWRP tributary population.

Table 1.5 Historical Population for LBWRP Tributary Area

Year	Population
2000	218,127
2001	219,429
2002	222,630
2003	226,090
2004	228,258
2005	229,501
2006	230,050
2007	230,580
2010	239,258
2011	239,732
2012	240,063
2013	241,853
2014	242,057
2015	243,489
2016	241,063
2017	223,561
2018	225,253
2019	225,222
2020	228,639

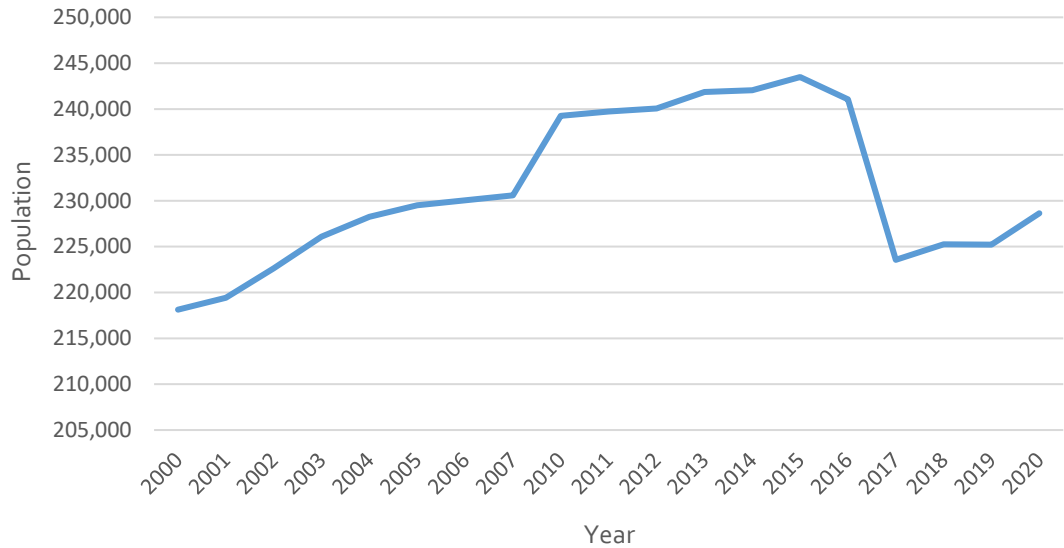


Figure 1.5 Historical Population for LBWRP Tributary Area

1.2.2 Population Projection

SCAG released the 2020-2045 Regional Transportation Plan/Sustainable Communities Strategy (RTP20) on September 3, 2020. It contains the latest population projections for Southern California. While the region's growth rate is lower than ever, between 2016 and 2045 the population in the SCAG region is expected to increase by 3.7 million people, with an annual growth rate of 0.61%. Table 1.6, excerpted from the SCAG RTP20 report, shows the annual average population growth rate from 1970 through 2045.

Table 1.6 Annual Population Growth Rate 1970 - 2045⁽¹⁾

	1970-2000	2000-2016	2016-2045
SCAG Region	1.65%	0.82%	0.61%
California	1.76%	0.93%	0.66%
United States	1.09%	0.86%	0.57%

Notes:

(1) Source: U.S. Census Bureau, CA DOF, SCAG.

The data were provided by SCAG at the Tier 2 Transportation Analysis Zone (TAZ) level. The tiered TAZ system was developed by SCAG and its member agencies and is generally based on Census Block data with some splits added according to major road, natural and artificial barriers, satellite photo, land use and local inputs. The Tier 2 level data are used by SCAG to help estimate current and future transportation conditions. SCAG data were used to determine projected populations by WRP tributary area in 2030, 2035, and 2045, presented in Table 1.7.

Table 1.7 Projected Population by WRP Tributary Area

WRP Tributary Area	2030	2035	2045
Warren Facility	2,709,250	2,743,121	2,803,692
La Cañada	1,299	1,305	1,315
LB	75,878	76,402	77,474

WRP Tributary Area	2030	2035	2045
LB or Warren Facility	187,319	188,199	189,816
LC	352,340	357,453	364,335
LC or Warren Facility	72,279	72,805	74,323
Pomona	98,021	99,146	101,398
Pomona or Warren Facility	114,304	120,176	131,786
SJC or Warren Facility	1,126,684	1,140,880	1,163,770
SJC or WN or LC or Warren Facility	11,833	12,356	13,374
WN or LC or Warren Facility	449,087	458,672	476,969
JOS Total	5,198,294	5,270,515	5,398,253

SCAG's 2020-2045 Regional Transportation Plan/Sustainable Communities Strategy only provides population data for the years 2030, 2035, and 2045. Therefore, populations for years in between the planning horizon of 2020 to 2050 (Table 1.8 and Figure 1.6) were calculated by either linear interpolation or extrapolated using the standard linear interpolation formula.

$$y = y_1 + \frac{(x - x_1)(y_2 - y_1)}{x_2 - x_1}$$

Where:

- x is the known value (year).
- y is the unknown value (population).
- x₁ and y₁ are the coordinates that are below the known x value.
- x₂ and y₂ are the coordinates that are above the x value.

The projected population per year for the LBWRP tributary area were calculated using the relationship presented in Table 1.4. The project population per year for the LBWRP tributary area are presented below in Table 1.8 and Figure 1.6.

Table 1.8 Projected Population for LBWRP Tributary Area

Year	Population
2021	226,811
2022	224,021
2023	226,811
2024	229,284
2025	231,482
2026	233,438
2027	235,183
2028	236,743
2029	238,139
2030	246,337
2031	246,601
2032	246,866
2033	247,131

Year	Population
2034	247,396
2035	247,661
2036	247,915
2037	248,169
2038	248,424
2039	248,678
2040	248,933
2041	249,187
2042	249,442
2043	249,696
2044	249,950
2045	250,205
2046	250,459
2047	250,714
2048	250,968
2049	251,223
2050	251,477

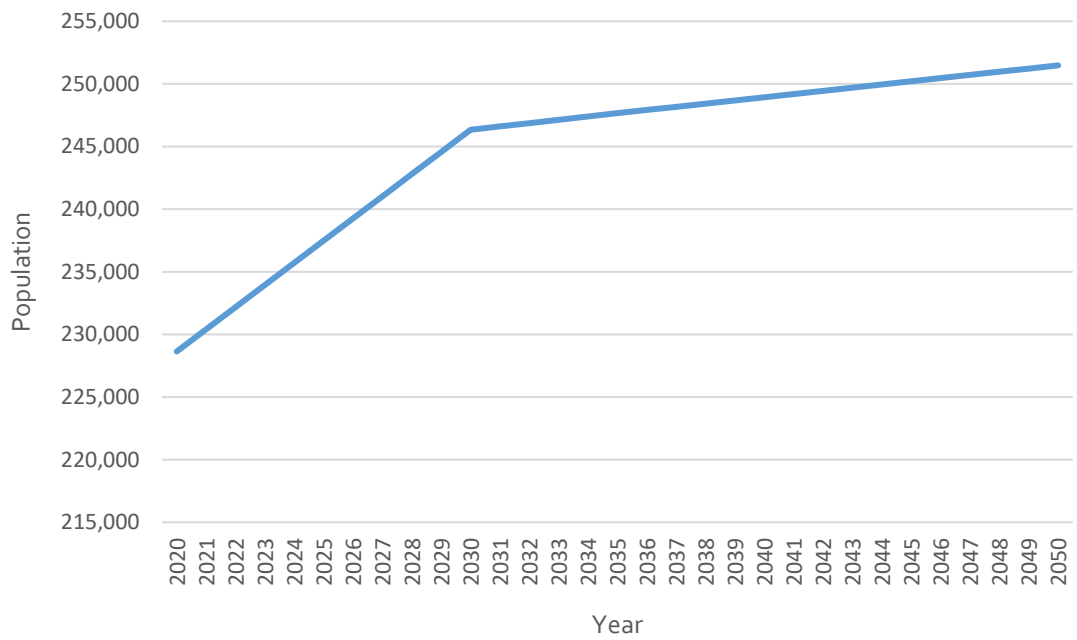


Figure 1.6 Projected Population for LBWRP Tributary Area

1.3 Historical Flow and Loading

To determine the capabilities of the LBWRP, the amount and composition of the influent wastewater flow must be quantified. The previous 8 years (2015 - 2022) of influent flow and wastewater quality data were examined to provide a representative sample of historical and recent conditions.

1.3.1 Influent Sources to LBWRP

Influent flows at LBWRP are comprised of mostly domestic (residential commercial) sewage with a small fraction of industrial sewage. Since 2015, the average amount of industrial flow at LBWRP has been approximately 3.75 percent (%) resulting in an average flow of 0.6 million gallons per day (mgd). Whereas commercial sewage comprises almost 1/3 of the total influent flow.

1.3.2 Historical Flow and Loading

The historical flows by water reclamation plant were developed using Table 1-1 in the industrial wastewater (IW) pretreatment annual reports, which identifies the percentage of the total flow that is comprised of the industrial wastewater flows. It was critical to know what percentage of the total flow was comprised of industrial flow to calculate Per Capita Generation Rates (PCGRs). The industrial wastewater flows were removed from the total flow since PCGR is traditionally based on Residential plus Commercial flows only. It should be noted that the annual reuse reports were not used as the influent flows do not differentiate what percentage of flow is from residential/commercial and industrial. Additionally, both the 2016 and 2018 technical reviews used the IW pretreatment annual reports to run their calculations. Furthermore, it should also be noted that the IW pretreatment annual reports list effluent flows, however, since these flows were used to calculate the total JOS PCGR, the entire influent flow within the JOS was captured in the calculations. To trend and calculate LBWRP-specific flows and loadings, influent flows for LBWRP were applied. The major influent parameters used in the assessment of current plant capabilities and establishing future needs are flow, biological oxygen demand (BOD), chemical oxygen demand (COD), and total suspended solids (TSS). Weekly composite samples of BOD, COD, and TSS for LBWRP raw influent were provided from the Reuse and Compliance section for the years of 2015 to 2022. The mass loadings in pounds per day (lb/d) were calculated by multiplying the concentration of each constituent by the flow rates in mgd and the density of water (8.34 lbs/gal). The data were first reviewed by the Reuse and Compliance section for extreme outliers. Data from the years 2017 through 2019 were excluded from the calculations due to the ongoing construction at the LBWRP. Two standard deviations were then applied to each sample result in the data set. Normally, the standard deviation is calculated using every sample result in the data set. The Average Annual Load was calculated by taking the average of the two standard deviations on the loads and the Average Daily Maximum Month Load was calculated by taking the 91.7-percentile of the data set, which represents 11 out of 12 months.

The Industrial Waste section provided the IW yearly surcharge data, including COD and TSS loads (BOD load is not measured for the industrial site dischargers). Some industrial sites have more than one permit (i.e. have more than one sewer connection) and the sample results of all the permits are then combined for each facility as a yearly total. This results in many yearly duplicates within the surcharge data, so duplicates were first removed, then each loading result was divided by 365 to calculate an estimated loading in pounds per day. This was then compared

to the total influent daily loads to LBWRP. The IW load was removed from the total load to determine the contribution from Residential and Commercial uses only.

The pounds per capita per day (ppcd) for each compound was then calculated by taking the mass loading (lb/d) and dividing by the population.

1.3.3 Historical Flows to LBWRP

The data demonstrate that the total flows to the LBWRP have been steady over the past 8 years, minus the years when the plant was under construction, which occurred August 2017-January 2018; June 15-November 5, 2018; and March 26-November 19, 2019. The influent flow was reduced approximately 25% during these periods. (Table 1.9, Figure 1.7). The data also show that the influent flows mostly remain under the daily average design flow of 34.0 mgd with a few exceptions for maximum daily flows. Additionally, the peak wet weather flows are under the Daily Maximum Peak Design Storm Flow of 60.0 mgd.

Table 1.9 Historical Flows to LBWRP Summary (Per Year) in mgd

Year	ADWF	AAF	ADMMF	MDF	PWWF
2015	16.0	14.7	17.2	18.6	29.1
2016	16.5	16.4	17.3	20.2	26.2
2017	11.8	14.0	18.2	27.0	53.4
2018	8.6	12.7	17.4	21.7	29.3
2019	9.5	11.9	17.0	23.7	33.4
2020	15.0	15.2	16.6	18.1	28.7
2021	15.5	15.6	16.4	22.9	33.7
2022	13.4	15.1	16.2	17.3	24.3
5-Year Average ⁽¹⁾	15.3	15.4	16.7	19.4	32.3

Notes:

(1) 5-year average is of 2015-2016, and 2020-2022 due to flow diversions to support LBWRP construction projects in 2017 through 2019, except for peak wet weather flows when LBWRP was in full operation.

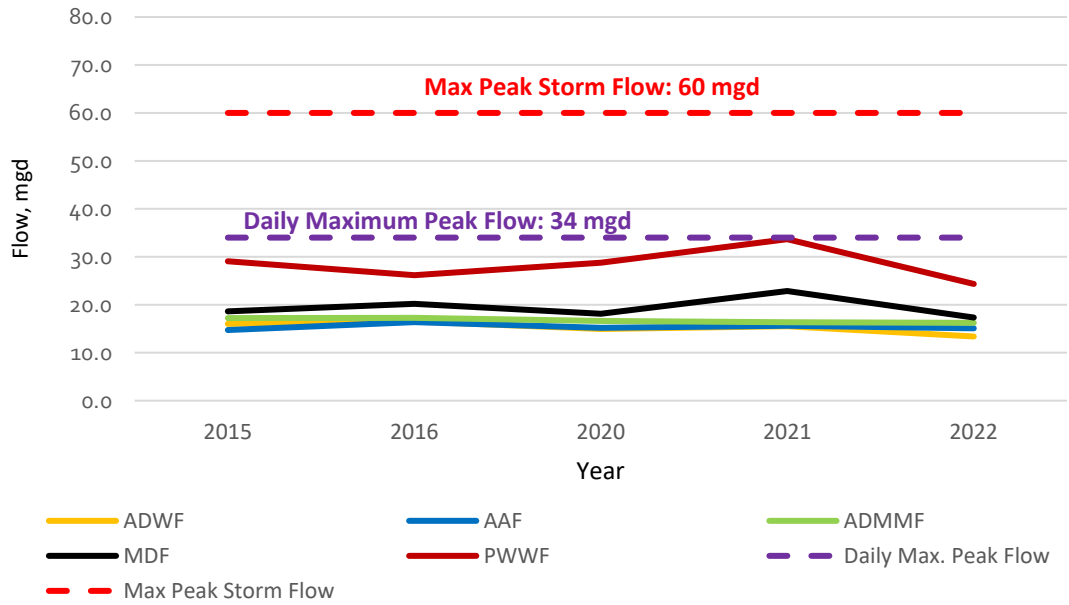


Figure 1.7 Historical Flow Trend at LBWRP

The Peaking Factor (PF) was calculated by dividing the peak flow (hourly, daily, monthly) by the average annual flow (Table 1.10). Typical PFs for plants more than 16 mgd are:

- Hourly PF: 2. This factor corresponds to the PWWF as maximum hourly flows are used to determine this factor.
- Daily PF: 1.5. This factor corresponds to the MDF as maximum single day flows are used to determine this factor.
- Monthly PF: 1.2. This factor corresponds to the ADMMF as the maximum 30-day average for the year is used to determine this factor.

Further, a 5-year average from 2015 to 2022 was calculated to project future flow conditions in Section 1.4.

Table 1.10 Historical Flow Peaking Factors at LBWRP Summary (Per Year)

Year	ADWF	AAF	ADMMF	MDF	PWWF
2015	1.1	1.0	1.2	1.3	2.0
2016	1.0	1.0	1.1	1.2	1.6
2017	0.8	1.0	1.3	1.9	3.8
2018	0.7	1.0	1.4	1.7	2.3
2019	0.8	1.0	1.4	2.0	2.8
2020	1.0	1.0	1.1	1.2	1.9
2021	1.0	1.0	1.0	1.5	2.2
2022	0.9	1.0	1.1	1.1	1.6
5-Year Average	1.0	1.0	1.1	1.3	2.3

1.3.3.1 Historical Loads to LBWRP

Mass loading data for the LBWRP indicate that overall loads over the past 8 years have remained consistent, with a slight decrease in 2020 which could be due, in part, to the pandemic (Tables 1.11 through 1.14). The IW load contribution was removed from the total load prior to calculating the peaking factors. Except for the BOD load, which is not measured for the industrial site dischargers. Peaking Factors (PF) were calculated by dividing Average Daily Maximum Month Load by the Annual Average Load (Table 1.11 through 1.14). The IW load contributions were assumed to remain constant throughout the planning horizon of 2050 and added back in with the Residential plus Commercial loading to get a complete projected total.

Table 1.11 Historical Influent LBWRP BOD5 Load Summary - Influent

Year	AA Loading (lb/d)	AA Concentration (mg/L)	MM Loading (lb/d)	MM Concentration (mg/L)	Peaking Factor
2015	46,585	379	59,018	481	1.3
2016	47,722	349	56,502	414	1.2
2017	43,111	368	58,939	504	1.4
2018	42,187	399	54,309	513	1.3
2019	35,270	356	49,716	502	1.4
2020	41,592	329	49,211	389	1.2
2021	44,121	338	49,101	377	1.1
2022	43,710	348	54,911	437	1.3
Average	44,580	347	54,208	422	1.2

Table 1.12 Historical Influent LBWRP COD Load Summary - Influent

Year	AA Loading (lb/d)	AA Concentration (mg/L)	MM Loading (lb/d)	MM Concentration (mg/L)	Peaking Factor
2015	95,680	779	121,861	992	1.3
2016	110,756	811	127,595	934	1.2
2017	85,705	732	124,004	1,059	1.4
2018	81,665	771	116,966	1,105	1.4
2019	67,697	683	100,864	1,018	1.5
2020	74,524	589	86,268	682	1.2
2021	84,572	649	99,267	762	1.2
2022	91,885	731	117,989	938	1.3
Average	91,255	711	119,095	927	1.3

Table 1.13 Historical Influent LBWRP TSS Load Summary - Influent

Year	AA Loading (lb/d)	AA Concentration (mg/L)	MM Loading (lb/d)	MM Concentration (mg/L)	Peaking Factor
2015	46,376	378	64,955	529	1.4
2016	50,604	371	60,616	444	1.2
2017	38,413	328	59,360	507	1.5

Year	AA Loading (lb/d)	AA Concentration (mg/L)	MM Loading (lb/d)	MM Concentration (mg/L)	Peaking Factor
2018	36,810	348	52,257	494	1.4
2019	31,506	318	43,586	440	1.4
2020	40,147	317	48,731	385	1.2
2021	41,182	316	48,017	368	1.2
2022	43,548	346	58,629	466	1.3
Average	44,236	344	57,774	450	1.3

Table 1.14 Historical IW Loads to LBWRP

Average	TSS (lb/d)	COD (lb/d)
2015-2016, 2020-2022	477	401

1.4 Flow and Load Projection

1.4.1 Approach

Projections of wastewater flow rates are used to determine the required capacity of treatment. Over the 2050 planning horizon, population in the LBWRP tributary area is projected to increase slightly while PCGRs are projected to decrease slightly. This will in turn increase the amount of wastewater flows to be treated despite PCGRs slightly decreasing. The projected flows were calculated by multiplying the projected populations by the projected per capita generation rate (PCGR). Further discussion on PCGRs can be found in the section below. The previous 8 years of influent wastewater quality data, from 2015 to 2022 (excluding the years during which the plant was under construction), were averaged on an Annual Average Load basis to provide a representative sample of recent conditions and used alongside the projected populations and flows to project future conditions. The industrial wastewater portion was assumed to remain constant and was therefore removed from the total loading. The remaining portion, comprised of Residential and Commercial, was used to forecast future Residential plus Commercial loads (i.e. equals the 5-year average Residential plus Commercial load divided by the 5-year average population). The constant IW load was then added back to attain complete projections.

1.4.2 Per Capita Generation Rate (PCGR)

The PCGR is the average amount of wastewater flow contributed to the system per person. The PCGR was calculated by dividing the wastewater flow (excluding industrial wastewater flow) by the corresponding tributary population, which provides the average amount of wastewater flow contributed per person with units of gallons per capita per day (gpcd).

1.4.2.1 Historical/Current PCGR

Two technical reviews of the Clearwater Program population and flow projections were performed, one in 2016 and one in 2018, and determined that the Districts' Joint Outfall System (JOS) PCGR (Residential plus Commercial) was 60 gpcd, was forecasted to drop to 55 gpcd within 10 years, and remain at 55 gpcd through 2050. Over the past several years, the total JOS flows have decreased slightly and the PCGR has remained at about 60 gpcd. The 5-year average from 2016 to 2020 was 60.6 gpcd. This coincides with the 2016 and 2018 Clearwater Program Technical Reviews determination of 60 gpcd.

Industrial wastewater flows are not included in the PCGR. Per the 2016 Technical Review, analysis of the industrial flows within the JOS demonstrated that they comprised 14 to 16 percent of the total flow from 2000 to 2016. Within the JOS, these flows varied between 55 to 58 mgd from 2008 to 2015 and were assumed to remain at the 2015 level of 58 mgd in the future per the 2016 Technical Review. The industrial flows have gone down to approximately 51 mgd over the past 5 years and even further down to 49 mgd in 2020; however, the pandemic may have contributed to this decrease. Since industrial flows can be unpredictable, for the purposes of this study, flows were kept at 58 mgd as they were in the 2016 and 2018 technical reviews as they remain reasonable for predicting future industrial wastewater flows.

A similar analysis for LBWRP was conducted to obtain the historical/current PCGR. The LBWRP PCGR differs from the JOS PCGR because it only includes the population tributary to LBWRP. Using the average total flow rates into LBWRP and subtracting IW flow, the 5-year average calculated PCGR is approximately 63 gpcd, about 5 percent higher than the Clearwater Program Technical Reviews determination. The average PCGR excluded the 2017 to 2019 years when there were active LBWRP construction projects.

1.4.3 Residential, Commercial, and Industrial Flows

Normally, the Districts' PCGR value includes both Residential and Commercial flows. However, for calculations involving the indoor residential water use standard, the Sewage Unit percentage for each category, 29.9% for Commercial and 70.1% for Residential as discussed below, were applied to the Districts' PCGR. This resulted in a Residential PCGR of 42.1 gpcd and Commercial PCGR of 17.9 gpcd.

The JOS Districts' SUs by user categories (residential versus commercial) are presented in the table below. The SUs were combined for the entire JOS to obtain the average percentage breakdown per Table 1.15 below. These percentages were used to determine the residential PCGR and the commercial PCGR for the JOS area as a whole.

Table 1.15 JOS Districts' Sewage Units

Use Type	Sewage Unit (Percent)
Commercial	29.9%
Residential	70.1%

1.4.4 DWR Indoor Residential Water Use Study (IRWUS) Goals

The current indoor residential water use standards set by the California State Legislature per Water Code Section 10609.4 are 55 gpcd (2020), 52.5 gpcd (2025), and 50 gpcd (2030) (Table 1.16). However, the Water Code allows for the Department of Water Resources (DWR), in coordination with the State Water Board, to conduct necessary studies in order to recommend a standard for indoor residential water use that more appropriately reflects best practices for indoor residential water use than the standard described in subdivision 10609.4. As California experiences more frequent and longer droughts due to climate change, DWR in coordination with the State Water Board conducted an Indoor Residential Water Use Study (IRWUS). The study produced the following DWR and State Water Board joint draft standards recommendations of 55 gpcd (2020), 47 gpcd (2025), and 42 gpcd (2030). See [Results of the Indoor Residential Water Use Study \(ca.gov\)](#), page 78.

It should be noted that these recommended standards are for indoor residential water use only and are only one part of many independent standards including indoor, outdoor, commercial, industrial, water losses, variances, and bonus incentives. Further, given that standards are not independently enforced, enforcement will be on the retail water supplier's total annual water use objective. It should also be noted that the minimum indoor water use is 35 gpcd per DWR's recent Working Group Meeting held April 22, 2021; therefore, once a supplier meets the 35 gpcd goal, there is no further requirement to decrease the gpcd.

Table 1.16 California State Standards and DWR Recommended Standards

Year	Current Statute (gpcd)	PCGR Recommended by DWR & State Water Board (gpcd)	Percent Decrease ⁽¹⁾
2020	55	55	N/A
2025	52.5	47	15%
2030	50	42	11%

Notes:

(1) Percent decrease is the calculated decrease between 2020-2025 and 2025-2030.

1.4.5 Projected PCGR

The Districts' new PCGR projection was based on DWR's recommended standards: 47 gpcd (2025) and 42 gpcd (2030 and onward). Only the Residential PCGR will decrease given that the Commercial PCGR will remain consistent and not be affected by any State PCGR standards (Table 1.17). The current total PCGR (Res + Comm) was calculated to be 63 gpcd and the residential portion is below current DWR's standards (44 gpcd) through 2025. Therefore, for this effort the total PCGR is kept at 63 gpcd through 2030, and adjusted down to 61 throughout the remaining planning period through 2050 to account for the DWR standard of 42 gpcd.

Table 1.17 Current and Projected PCGR

PCGR (gpcd)	Current	2025 - 2030	2030 - 2050
Res + Com	63	61	61
Res Only	44	42	42
Com Only	19	19	19

1.5 Flow and Load Projections

The flow and load projections for LBWRP are presented in Table 1.18. Flow and load projections were calculated as follows:

- The loading projections (years 2030, 2040, and 2050) were calculated by multiplying the projected population by the estimated pounds per capita per day (ppcd) for each constituent.
 - The ppcd was calculated by taking the combined Residential plus Commercial 5-year average loading (lb/d) and dividing by the 5-year average tributary population. Note that these loadings already subtract the IW contribution except for BOD as the IW section does not measure the BOD loads for industrial sites.
- Peaking factors were calculated by dividing Average Daily Maximum Month Load by the Annual Average Load (ADMML/AAL). The 5-year averages, using data from years 2016

to 2020, were calculated for each type of loading and then used to calculate future loadings (Table 1.18). To forecast future loadings, the equation below was applied.

- The mass loading in pounds per day (lbs/d) was calculated by multiplying the 5-year average ppcd for AAL by the projected population. The loading concentration in milligrams per liter (mg/L) was calculated by dividing the lbs/d by the projected flow (mgd) and a conversion factor of 8.34 pounds per gallon (lb/gal) to attain the concentration in milligrams per liter (mg/L).
- To attain the ADMML the AAL was multiplied by the peaking factor to attain mass loading in pounds per day (lb/d). This was then divided by projected flow and conversion factor of 8.34 pounds per gallon (lb/gal) to attain the concentration in milligrams per liter (mg/L). For example, to get the BOD in lb/d and mg/L for ADWL in the year 2030, the following calculations were performed.

$$BOD_{ADMML}(lb/d) = BOD_{AAL}(lb/d) * PF_{ADMML}$$

$$BOD_{ADMML}(mg/L) = BOD_{ADMML}(lb/d) / Flow_{2030}(mgd) / 8.34(lb/gal)$$

Table 1.18 Flow and Load Projection for LBWRP

Item	Recent 5-Year Average	Peaking Factor	2030 Projection	2040 Projection	2050 Projection
Service Area Population	231,227		246,338	248,935	251,479
AAL					
Flow, mgd	15.4		15.6	15.7	15.9
BOD, ppcd	0.2		0.2	0.2	0.2
BOD, lb/d	44,580		47,493	47,994	48,485
BOD, mg/L	347		365	365	366
COD, ppcd	0.4		0.4	0.4	0.4
COD, lb/d	91,255		97,193	98,213	99,213
COD, mg/L	711		748	748	748
TSS, ppcd	0.2		0.2	0.2	0.2
TSS, lb/d	44,236		47,104	47,596	48,079
TSS, mg/L	344		362	362	363
ADMML					
Flow, mgd	17.0	1.1	17.2	17.4	17.5
BOD, ppcd	0.23	1.2	0.23	0.23	0.23
BOD, lb/d	54,208		57,750	58,359	58,956
BOD, mg/L	422		444	444	445
COD, ppcd	0.51	1.3	0.51	0.51	0.51
COD, lb/d	119,095		126,844	128,176	129,480
COD, mg/L	927		976	976	976
TSS, ppcd	0.25	1.3	0.25	0.25	0.25
TSS, lb/d	57,774		61,518	62,161	62,792
TSS, mg/L	450		473	473	474

1.6 Summary

Population, flows, and loads are projected to increase through the 2050 planning horizon as shown in the figures below. Although population is expected to increase, the projected flow only increases slightly due to continued conservation measures and a predicted decrease to indoor residential water use standards. The mass loading is projected to only increase slightly as well (Figures 1.8 through 1.11). The projected flow decrease accounts for the DWR goal for reduced water usage starting in 2030, which will result in higher strength wastewater.

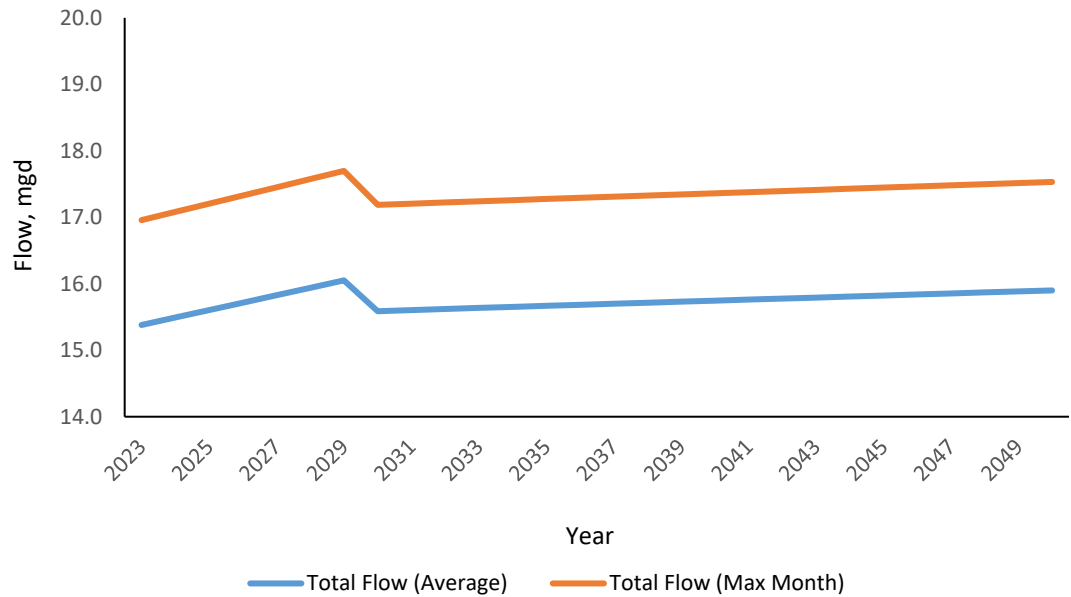


Figure 1.8 Flow Projections for LBWRP Tributary Area

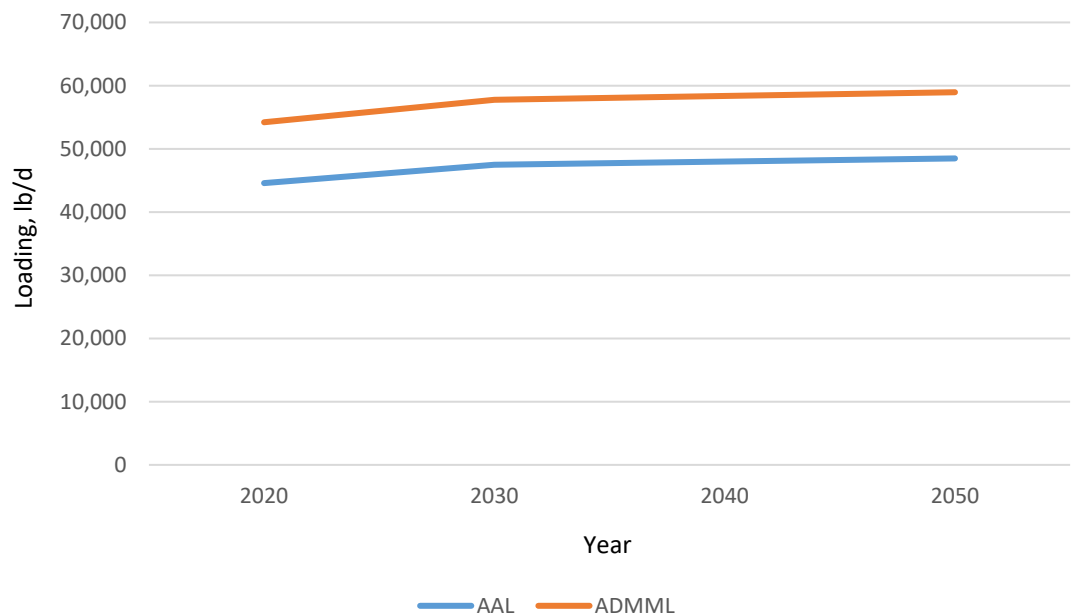


Figure 1.9 BOD₅ Loading Projections

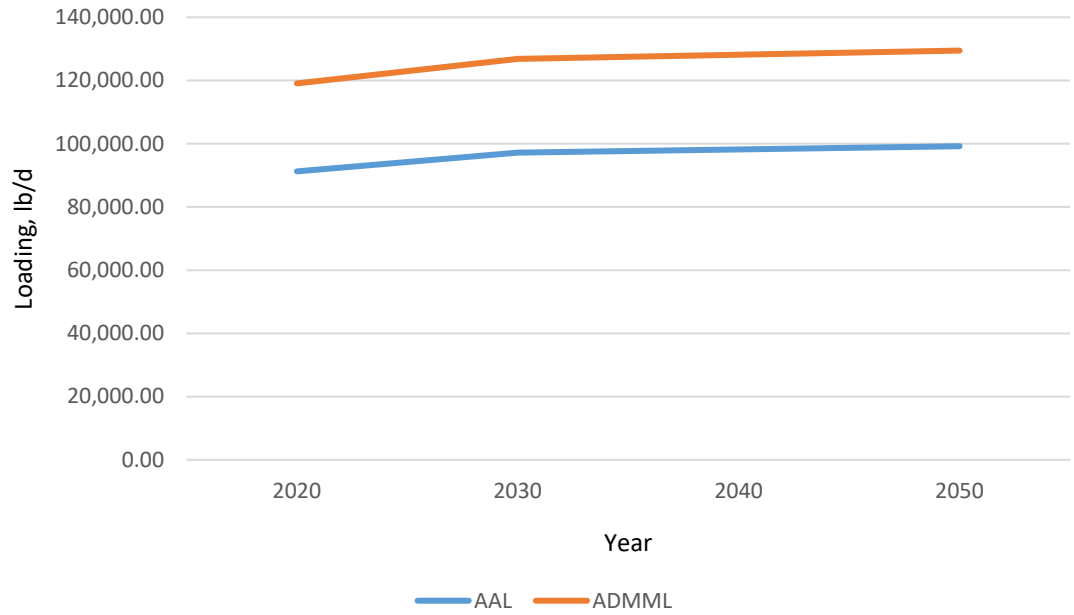


Figure 1.10 COD Loading Projections

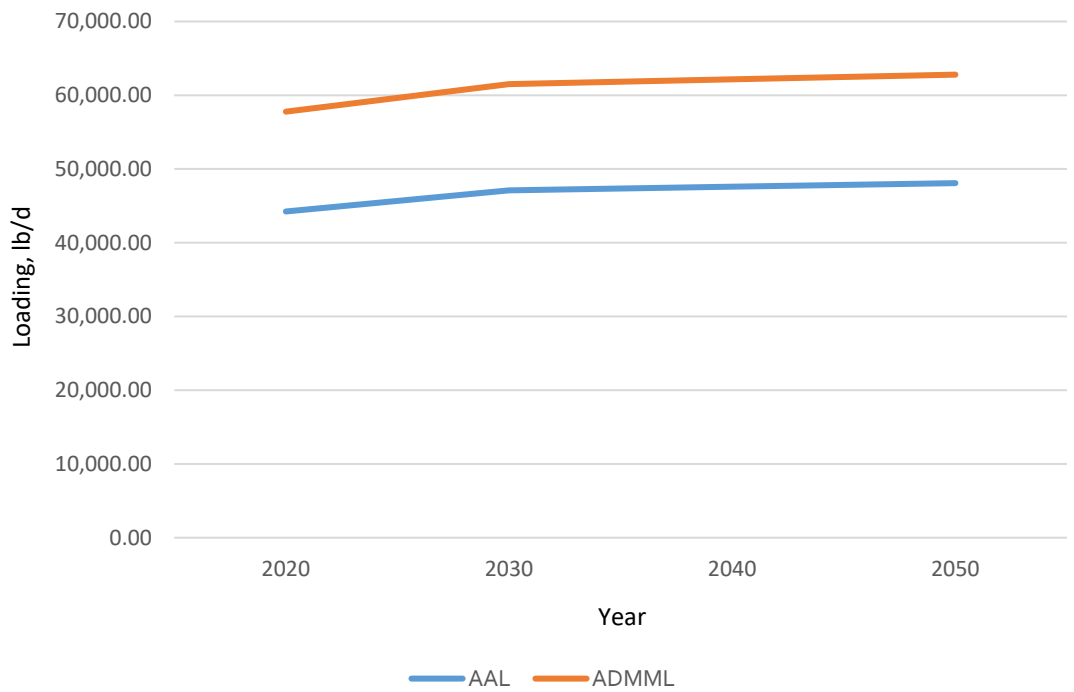


Figure 1.11 TSS Loading Projections

1.7 References

1. Clearwater Program Master Facilities Plan Technical Memorandum 2-2 "Population Projections and Service Area Evaluations" (DOC#[2542521](#)).
2. Clearwater Program Master Facilities Plan Technical Memorandum 2-3 "Wastewater Flows and Characteristics" (DOC#[2542524](#))
3. "2020-2045 Regional Transportation Plan/Sustainable Communities Strategy," Southern California Association of Governments, September 3, 2020
4. IW Pretreatment Annual Reports
5. "Design Guidelines for Wastewater Facilities," Maryland Department of the Environment, Engineering and Capital Projects Program, 2013 (typical PFs for plants more than 16mgd)
6. DWR Working Group Meeting, April 22, 2021 (DOC#[6184711](#))
7. "Public Review Draft Report to the Legislature on Results of the Indoor Residential Water Use Study," Department of Water Resources, 2021



Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 2
LONG BEACH WATER RECLAMATION
PLANT REGULATORY REVIEW

FINAL | January 2025





**LOS ANGELES COUNTY
SANITATION DISTRICTS**
Converting Waste Into Resources

Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 2
LONG BEACH WATER RECLAMATION PLANT
REGULATORY REVIEW

FINAL | January 2025

Digitally signed by Lydia A. Holmes
Contact Info: Carollo Engineers, Inc.
Date: 2025.01.15 15:34:39-08'00'



Contents

Technical Memorandum 2 - Long Beach Water Reclamation Plant Regulatory Review

2.1 Introduction	2-1
2.1.1 Purpose	2-1
2.1.2 Background and Regulatory Setting	2-2
2.2 Discharge to Surface Waters	2-5
2.2.1 Current Requirements	2-5
2.2.2 Future Requirements	2-6
2.2.3 Compliance Evaluation	2-14
2.2.4 Proposed Strategies for Compliance	2-21
2.3 Potable Recycled Water	2-21
2.3.1 Current Requirements	2-21
2.3.2 Future Requirements	2-21
2.3.3 Compliance Evaluation	2-23
2.3.4 Proposed Strategy for Compliance	2-27
2.4 Non-Potable Reuse	2-27
2.4.1 Current Requirements	2-27
2.4.2 Future Requirements	2-28
2.4.3 Compliance Evaluation	2-29
2.4.4 Proposed Strategy for Compliance	2-30
2.5 Summary	2-30

Tables

Table 2.1	Types of Operating Permits Issued to the Sanitation Districts WRPs	2-1
Table 2.2	LBWRP NPDES Permit Effluent Limitations (Order No. R4-2022-0032)	2-3
Table 2.3	Effluent Limits of Concern for Discharge to Surface Waters	2-5
Table 2.4	PFOA and PFOS EPA Recommended Aquatic Life Water Quality Criteria	2-12
Table 2.5	LBWRP Chronic Toxicity Results (2016-2022)	2-15
Table 2.6	LBWRP Chronic Toxicity TST "Fail" Occurrences (2016-2022)	2-15
Table 2.7	Current and Projected Effluent Limits for Ammonia (as N)	2-17
Table 2.8	California PFAS Drinking Water Advisory Values	2-22
Table 2.9	EPA PFAS MCLs and MCLGs	2-22

Table 2.10	Recycled Water Limits for NPR (Current and Future)	2-28
Table 2.11	Summary of Regulatory Concerns and Potential Strategies	2-31
Table 2.12	Projected Timelines for Addressing Regulatory Concerns	2-31

Figures

Figure 2.1	Schematic of LBWRP Discharge Location and End Uses	2-4
Figure 2.2	LBWRP Effluent Temperature 2016 to 2022	2-15
Figure 2.3	LBWRP Effluent Concentrations of Total Ammonia (as N) 2016 to 2022	2-16
Figure 2.4	LBWRP Effluent Concentration of Nitrite (as N) 2016 to 2022	2-18
Figure 2.5	LBWRP Effluent Concentrations of Nitrate Plus Nitrite (as N) 2016 to 2022	2-19
Figure 2.6	LBWRP Effluent Total Coliform 2016 to 2022	2-20
Figure 2.7	LBWRP Effluent Concentrations of Total Residual Chlorine 2016 to 2022	2-20
Figure 2.8	LBWRP Influent and Effluent Concentrations of PFOA 2016 to 2022	2-23
Figure 2.9	LBWRP Influent and Effluent Concentrations of PFOS 2016 to 2022	2-24
Figure 2.10	LBWRP Influent and Effluent Concentrations of PFBS 2016 to 2022	2-25
Figure 2.11	LBWRP Influent and Effluent Concentrations of PFHxS 2016 to 2022	2-26
Figure 2.12	LBWRP Influent and Effluent Hazard Index Results 2020 to 2022	2-27
Figure 2.13	LBWRP Effluent TDS 2016 to 2022	2-29
Figure 2.14	LBWRP CT Disinfection 2017 to 2022	2-30

Abbreviations

AMEL	average month effluent limit
ARB	antibiotic resistance bacteria
ARG	antibiotic resistant genes
B&C&B	biostimulations, cyanotoxins, and biological conditions
Basin Plan	Los Angeles Region Water Quality Control Plan
BMI	benthic macroinvertebrate
C	Celsius
<i>C. dubia</i>	<i>Ceriodaphnia dubia</i>
CEC	constituents of emerging concern
cfs	cubic feet per second
CFU/100 mL	colony forming unit per 100 milliliters
CIWQS	California Integrated Water Quality System
CT	contact time
CV	coefficient of variability
DDW	Division of Drinking Water
eDNA	environmental DNA
ELS	early life stage
EPA	U.S. Environmental Protection Agency
F	Fahrenheit
gpm/ft ²	gallons per minute per square foot
GWR	groundwater recharge
HA	health advisory
HFPO-DA or GenX	hexafluoropropylene oxide dimer acid
IPR	indirect potable reuse
JOS	Joint Outfall System
LACDPW	Los Angeles County Department of Public Works
lbs/day	pounds per day
LBWRP	Long Beach Water Reclamation Plant
LCWRP	Los Coyotes Water Reclamation Plant
MCL	maximum contaminant level
MCLG	maximum contaminant level goal
MDEL	maximum daily effluent limitation
MEC	measured environmental concentration
MFPs	Master Facilities Plans
mgd	million gallons per day
µg/L	micrograms per liter
mg/L	milligrams per liter

mg/L-min	milligrams per liter per minutes
mg-min/L CT	milligrams per minutes per liter contact time
mL/L	milliliters per liter
MMEL	median monthly effluent limitation
MPN	most probable number
MPN/100 mL	most probable number per 100 milliliters
MRP	monitoring and reporting program
MTL	monitoring trigger level
ng/L	nanogram per liter
NL	notification level
NOA	notice of applicability
NOEC	no observable effect concentration
NOI	notice of intent
NPR	non-potable reuse
NPDES	National Pollutant Discharge Elimination System
NTU	nephelometric turbidity unit
OCWD	Orange County Water District
OEHAA	Office of Environmental Health Hazard Assessment
PFAS	per- and polyfluoroalkyl substances
PFBS	perfluorobutanesulfonic acid
PFHxA	perfluorohexanoic acid
PFHxS	perfluorohexane sulfonate
PFNA	perfluorononanoic acid
PFOA	perfluorooctanoic acid
PFOS	perfluorooctane sulfonate
PHG	Public Health Goal
POTW	Publicly Owned Treatment Work
POWRP	Pomona Water Reclamation Plant
PQL	practical quantitation level
Regional Water Board	Regional Water Quality Control Board
RO	reverse osmosis
RL	response level
Sanitation Districts	Los Angeles County Sanitation Districts
SDWA	Safe Drinking Water Act
SJCWRP	San Jose Creek Water Reclamation Plant
SCCWRP	Southern California Coastal Water Research Project
SNMP	Salt and Nutrient Management Plan
State Water Board	State Water Resources Control Board
s.u.	standard units

TAC	technical advisory committee
TIE	toxicity identification evaluation
TDS	total dissolved solids
TM	technical memorandum
TMDL	total maximum daily load
TRE	toxicity reduction evaluation
TST	test of significant toxicity
Warren Facility	A.K. Warren Facility Water Resource Facility
WDR	waste discharge requirement
WNWRP	Whittier Narrows Water Reclamation Plant
WQBEL	Water Quality-based Effluent Limitations
WQO	water quality objective
WRD	Water Replenishment District of Southern California
WRP	water reclamation plant
WRR	Water Reclamation (or Recycling) Requirement
Vander Lans WTF	Leo J. Vander Lans Water Treatment Facility

Technical Memorandum 2

LONG BEACH WATER RECLAMATION PLANT REGULATORY REVIEW

2.1 Introduction

2.1.1 Purpose

The Los Angeles County Sanitation Districts (Sanitation Districts) operates the A.K. Warren Water Resource Facility (Warren Facility) and six water reclamation plants (WRPs) in the Joint Outfall System (JOS). The WRPs include the Long Beach Water Reclamation Plant (LBWRP), the Los Coyotes Water Reclamation Plant (LCWRP), the Pomona Water Reclamation Plant (POWRP), the San Jose Creek Water Reclamation Plant (SJCWRP), the Whittier Narrows Water Reclamation Plant (WNWRP), and La Cañada WRP. The WRPs operate under different types of permits issued by the Los Angeles Regional Water Quality Control Board (Regional Water Board) depending on the discharge location and use type. The type of permits currently in effect for each of the WRPs are listed in Table 2.1. National Pollutant Discharge Elimination System (NPDES) permits authorize discharge to surface waters and must be renewed every five years. Water Reclamation Requirements (WRRs) authorize use of recycled water for indirect potable reuse (IPR) or non-potable reuse (NPR). WRRs do not expire and may be amended or reissued when projects are modified or new regulations are adopted.

Table 2.1 Types of Operating Permits Issued to the Sanitation Districts WRPs

WRP	NPDES	WRRs	
	Discharge to Surface Waters	IPR by Groundwater Replenishment	NPR
SJCWRP	X	X	X
WNWRP	X	X	X
POWRP	X	X	X
LBWRP	X	(1)	X
LCWRP	X	--	X
La Cañada WRP	--	--	X

Notes:

(1) LBWRP recycled water is provided to the Water Replenishment District of Southern California (WRD) for advanced treatment at the Leo J. Vander Lans Water Treatment Facility (Vander Lans WTF) and groundwater injection into the Alamitos Barrier. This use is covered under a permit held by WRD and not the Sanitation Districts.

The upstream treatment plants are connected by more than 1,200 miles of interceptors and trunk sewers to the Warren Facility located in Carson. This system allows for the diversion of influent flows into or around each upstream plant. To assist in the planning for future uses and permit compliance, the Sanitation Districts is preparing Master Facilities Plans (MFPs) for (1) the SJCWRP, (2a) the LBWRP and LCWRP, and (2b) the POWRP and WNWRP. The SJCWRP MFP

was prepared during Phase 1. Preparation of the remaining MFPs are to be completed during Phase 2a and Phase 2b, continuing the process and information developed during Phase 1.

The purpose of this technical memorandum (TM) is to summarize the existing and future regulatory requirements and water quality constituents of concern associated with LBWRP operation. Potential strategies are recommended to address the identified compliance concerns. The information presented in this TM will be used to guide recommendations on potential regulatory solutions and alternatives for treatment plant improvements.

2.1.2 Background and Regulatory Setting

The Sanitation Districts owns and operates the LBWRP, a tertiary wastewater treatment plant that currently receives wastewater from the Cities of Artesia, Bellflower, Cerritos, Hawaiian Gardens, Lakewood, Long Beach, and Signal Hill. The wastewater is a mixture of domestic and pre-treated industrial wastewater. LBWRP has a design capacity of 25 million gallons per day (mgd), and serves an estimated population of 229,000 people. The LBWRP facility does not process biosolids. Solids are conveyed via a trunk sewer to the Warren Facility for treatment and disposal.

Treated effluent produced at LBWRP currently has three end uses: discharge to surface waters, IPR, and NPR. Each of the end uses is regulated separately. Surface water discharge from LBWRP to Coyote Creek (Discharge Point 001, tributary to San Gabriel River Estuary) is regulated under a NPDES permit (Order No. R4 2022-0032). The production and use of recycled water for IPR is regulated under waste discharge requirements (WDRs) and WRRs (Order No. R4-2014-0111) issued to the WRD and the Los Angeles County Department of Public Works (LACDPW). The requirements apply to operation of the Vander Lans WTF and the Alamitos Barrier Recycled Water Project and are not discussed in this TM. The production, distribution, and use of recycled water for NPR is regulated under two WRRs (Order No. 87-47 as amended by Order No. 97-072 and WQ Order 2016-0068-DDW). The Sanitation Districts is in the process of transitioning all non-potable uses to WQ 2016-0068-DDW (Statewide General Order WRRs).

The specific effluent limits applicable to the Discharge Point 001 are presented in Table 2.2. A schematic of LBWRP, its discharge point, and the associated end uses is shown as Figure 2.1.

Table 2.2 LBWRP NPDES Permit Effluent Limitations (Order No. R4-2022-0032)

Constituents	Effluent Limitations	Unit	Discharge Point 001
Biochemical Oxygen Demand	Average Monthly	mg/L lbs/day % Removal	20 4,200 ⁽¹⁾ ≥85
	Average Weekly	mg/L lbs/day	30 6,300 ⁽¹⁾
	Max Daily	mg/L lbs/day	45 9,400 ⁽¹⁾
Total Suspended Solids	Average Monthly	mg/L lbs/day % Removal	15 3,100 ⁽¹⁾ ≥85
	Average Weekly	mg/L lbs/day	40 8,300 ⁽¹⁾
	Max Daily	mg/L lbs/day	45 9,400 ⁽¹⁾
Temperature	Max Daily	degrees F	80 ⁽²⁾
Total Coliform	Average Monthly	MPN or CFU/100 mL	23 ⁽³⁾
	Average Weekly	MPN or CFU/100 mL	2.2 ⁽³⁾
	Max Daily	MPN or CFU/100 mL	240 ⁽³⁾
Oil and Grease	Average Monthly	mg/L lbs/day	10 2,100 ⁽¹⁾
	Max Daily	mg/L lbs/day	15 3,100 ⁽¹⁾
Settleable Solids	Average Monthly	mL/L	0.1
	Max Daily	mL/L	0.3
Chloride	Average Monthly	mg/L lbs/day	230 ⁽⁴⁾ 48,000 ⁽¹⁾
	Average Monthly	mg/L lbs/day	4.4 920 ⁽¹⁾
Ammonia Nitrogen	Max Daily	mg/L lbs/day	11 2,300 ⁽¹⁾
	Average Monthly	mg/L lbs/day	8 1,670 ⁽¹⁾
Nitrite (as N)	Average Monthly	mg/L lbs/day	1 210 ⁽¹⁾
Chronic Toxicity, <i>Ceriodaphnia dubia</i> , Survival and Reproduction Endpoints	Average Monthly	Pass or fail, % Effect (TST)	Pass ⁽⁵⁾⁽⁶⁾
	Max Daily	Pass or fail, % Effect (TST)	Pass or % Effect <50 (survival endpoint) ⁽⁵⁾⁽⁶⁾
Copper, Total Recoverable, Dry Weather	Average Monthly	µg/L lbs/day	2.8 ⁽⁷⁾ 0.59 ⁽¹⁾
	Max Daily	µg/L lbs/day	4.2 ⁽⁷⁾ 0.87 ⁽¹⁾
Copper, Total Recoverable, Wet Weather	Max Daily	µg/L lbs/day	14.7 ⁽⁸⁾ 3.1 ⁽¹⁾
Lead, Total Recoverable, Wet Weather	Max Daily	µg/L lbs/day	87 ⁽⁸⁾ 18 ⁽¹⁾
Zinc, Total Recoverable, Wet Weather	Max Daily	µg/L lbs/day	125 ⁽⁸⁾ 26 ⁽¹⁾
Selenium, Total Recoverable	Average Monthly	µg/L lbs/day	4.3 0.91 ⁽¹⁾
	Max Daily	µg/L lbs/day	7.4 1.5 ⁽¹⁾
Dieldrin	Average Monthly	µg/L lbs/day	0.00014 0.000029 ^(a)
	Max Daily	µg/L lbs/day	0.00028 0.000058 ^(a)
pH		s.u.	6.5 – 8.5
Turbidity		NTU	2 (average within 24-hours) 5 (no more than 5% of time within 24-hours) 10 (at any time)

Notes:

Abbreviations: CFU/100 mL - colony forming unit per 100 milliliters; F - Fahrenheit; lbs/day - pounds per day; µg/L - micrograms per liter; mg/L - milligrams per liter; mL/L - milliliters per liter; MPN/100 mL - most probable number per 100mL; NTU - nephelometric turbidity unit; s.u. - standard units; TST - test of significant toxicity.

- (1) The mass-based effluent limitations are based on the plant design flow rate of 25 mgd at Discharge Point 001. During wet-weather storm events in which the flow exceeds the design capacity, the mass discharge rate limitations do not apply and the concentration limitations are the only applicable effluent limitations.
- (2) NPDES Permit (Table 5) provides a 10-year compliance schedule and an Interim Effluent Limitation of 86 degrees F (daily maximum) which can only be exceeded as a result of ambient temperature.
- (3) Average Weekly Limit: the median number of total coliform bacteria in the disinfected effluent does not exceed a 7-day median of 2.2 MPN or CFU/100 mL utilizing the bacteriological results of the last 7 days for which an analysis has been completed. Average Monthly Limit: the number of total coliform bacteria does not exceed 23 MPN or CFU/100 mL in more than one sample within any 30-day period. Max Daily Limit: no sample exceeds 240 MPN or CFU/100 mL.
- (4) The chloride average monthly limit is based on a 4-day average recommended objective in the Basin Plan to protect aquatic life beneficial uses in Coyote Creek.
- (5) The average monthly result is compared to the median monthly effluent limitation (MMEL) and shall be reported as "Pass" or "Fail." The maximum daily result is compared to the maximum daily effluent limitation (MDEL) and shall be reported as "Pass" or "Fail" and "% Effect."
- (6) A numeric Water Quality-based Effluent Limitations (WQBEL) is established because some chronic toxicity test results showed there was reasonable potential for the effluent to cause or contribute to an exceedance of the chronic toxicity water quality objective (WQO).
- (7) The dry-weather effluent limitation applies when the maximum daily flow in Coyote Creek is less than 156 cubic feet per second (cfs) as measured at the LACDPW flow gauge station F354-R (RSW-007).
- (8) The wet-weather effluent limitations apply when the maximum daily flow in Coyote Creek is equal to or greater than 156 cfs as measured at the LACDPW flow gauge station F354-R (RSW-007).

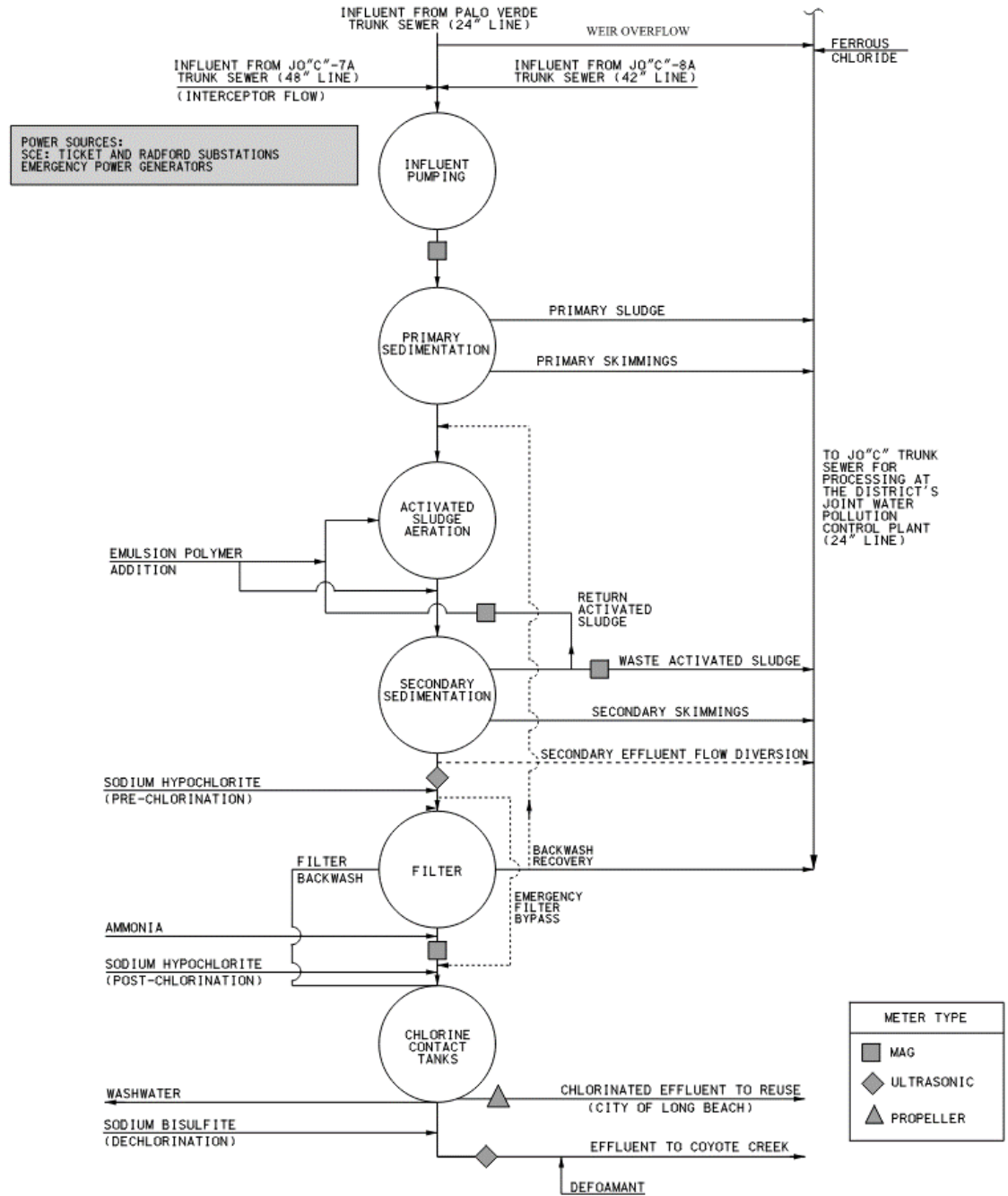


Figure 2.1 Schematic of LBWRP Discharge Location and End Uses

2.2 Discharge to Surface Waters

Discharge to surface waters is regulated under NPDES No. CA0054119, currently implemented as Order No. R4-2022-0032, adopted on February 10, 2022 and became effective on April 1, 2022. The effluent quality is compared to the receiving water quality at the discharge point to determine appropriate effluent limits for the reasonable protection of in-stream beneficial uses. Tertiary effluent from the LBWRP is discharged to Coyote Creek at Discharge Point 001 which is located approximately 2,200 feet upstream from the confluence with San Gabriel River, within the San Gabriel River Estuary in the San Gabriel River Watershed. As there are no additional discharge points for the WRP, this TM focuses on the requirements for this single point of discharge.

2.2.1 Current Requirements

The designated beneficial uses of Coyote Creek near Discharge Point 001 are rare, threatened or endangered species (RARE, existing) and noncontact water recreation (REC-2, intermittent). The potential beneficial uses are municipal and domestic water supply (MUN), industrial service supply (IND), industrial process supply (PROC), warm freshwater habitat (WARM), wildlife habitat (WILD), and water contact recreation (REC-1). The designated beneficial uses of the San Gabriel River Estuary are industrial service supply (IND), navigation (NAV), commercial and sport fishing (COMM), estuarine habitat (EST), marine habitat (MAR), wildlife habitat (WILD), rare, threatened migration of aquatic organism (MIGR), spawning, reproduction, and/or early development (SPWN), water contact recreation (REC-1), and noncontact recreation (REC-2). The potential beneficial uses are shellfish harvesting (SHELL).

The NPDES permit effluent limits that pose current or future compliance concerns are presented in Table 2.3. The LBWRP effluent is monitored at representative locations downstream of any in-plant return flows.

Table 2.3 Effluent Limits of Concern for Discharge to Surface Waters

Constituent	LBWRP (Discharge Point 001)		
	Average Weekly Limit	Average Monthly Limit	Max Daily Limit
Temperature	--	--	80 degrees F ⁽¹⁾
Chronic Toxicity (<i>Ceriodaphnia dubia</i> survival and reproduction endpoints)	--	Pass (reproduction endpoint)	Pass or % Effect <50 (survival endpoint)
Ammonia (as N)	--	4.4 mg/L (920 lbs/day)	11 mg/L (2,300 lbs/day)
Nitrite (as N)	--	1 mg/L (210 lbs/day)	--
Nitrate + Nitrite (as N)	--	8 mg/L (1,670 lbs/day)	--
Total Coliform	2.2 MPN/100 mL	23 MPN/100 mL	240 MPN/100 mL
Total Residual Chlorine	--	--	0.1 mg/L (21 lbs/day)

Notes:

(1) Interim limit of 86 degrees F is applicable 4/1/2022 through 3/31/2032.

2.2.2 Future Requirements

Temperature: Starting in 2021, the Regional Water Board issued NPDES permits to the Sanitation Districts WRPs, including the LBWRP, with a new interpretation of the Los Angeles Region Water Quality Control Plan ("Basin Plan") water quality objective (WQO) for temperature. Consequently, these permits now include a maximum temperature limitation of 80 degrees F for treated wastewater effluent and receiving waterbodies. Additionally, the receiving water cannot be altered more than 5 degrees F above the natural temperature. However, the origin of the temperature WQO to protect the warm water freshwater habitat beneficial use (WARM) is not provided in the Basin Plan or available historical documentation. Warm freshwater habitats vary widely across the region and the temperatures needed to protect aquatic life in those habitats may also vary. Because effluent discharged from the WRPs frequently exceeds the 80 degrees F limitation and the WRPs cannot immediately comply with the new limitations, the Regional Water Board has allowed the Sanitation Districts a 10-year compliance schedule which includes time to conduct studies to evaluate the temperature impacts in the watershed.

The Sanitation Districts has developed and is currently implementing the temperature study to better understand the temperature range that is protective of the WARM beneficial use for waterbodies in reaches that receive the WRP effluent. The goals of the study include:

- Determining the wholly or partially aquatic-dependent taxa in the waterbodies.
- Describing the relationship between waterbody temperatures and the probability (or likelihood) that different aquatic life stages are supported.
- Determining the relationships between waterbody temperature and the support of aquatic life based on the taxon's location in the watershed and seasonality.
- Describing critical exposure times, durations, and/or frequencies associated with the temperature relationships.

A technical workplan, which outlined the technical studies designed to fill data gaps identified as critical to addressing the study goals, was developed in consultation with a Technical Advisory Committee (TAC) and the Regional Water Board. The Sanitation Districts is in the process of conducting the water temperature and biological field studies identified in the workplan. Using existing data, supplemented by data collected through technical workplan implementation, occurrence of species and life stages; thermal preferences, tolerances, and limitations of each species or closely related relatives; habitat availability and suitability; and water temperature and flow, will be evaluated. The final report will include relational-analyses between waterbody temperature (exposure) and possible exceedance of taxa-specific upper temperature tolerance thresholds (effect) within reaches and under seasonal flow conditions. Comparison of water temperature versus the upper temperature tolerance of species at target locations within stream reaches will facilitate characterization of possible effects on those taxa from varying waterbody temperatures for the watershed and reaches. Such a comparison, in addition to advanced knowledge of habitat availability and suitability, along with seasonal water temperature and flow conditions within each stream reach, will be utilized to address questions identified in the workplan.

Task Status:

- Study Design/Special Study Development:
 - Completed: Technical Workplan.
- Biological Community Assessment:
 - Completed: Non-wadeable areas benthic macroinvertebrate (BMI) and algae collection, environmental DNA (eDNA) sampling for vertebrate taxa (Fall 2022).
 - Ongoing: Laboratory processing of BMI and algae samples, and eDNA samples.
 - Completed: eDNA and electrofishing sampling for vertebrate taxa (Spring 2023).
- Water Temperature Monitoring:
 - Completed: Summer monitoring (weekly cross-sectional temperature monitoring during Summer 2022).
 - Ongoing: Fall/Winter/Spring monitoring (monthly cross-sectional temperature monitoring during Fall 2022 through July 2023).
- Data Compilation and Evaluation:
 - Completed: Aggregation of existing biological data and Summer 2022 field data.
 - Ongoing: Aggregation of Fall/Winter/Spring field data.
 - Upcoming: Aggregation of BMI, algae, and eDNA data and review of temperature tolerance for reported taxa.
- Reporting:
 - Completed: First Progress Report (February 2023).
 - Upcoming: Final Report (Draft November 2024, Final February 2024).

***Ceriodaphnia Dubia* Toxicity:** The State Water Resource Control Board (State Water Board) adopted Toxicity Provisions¹ applicable to all discharges to inland surface waters and enclosed bays and estuaries. The provisions require new sensitive species screening, compliance with maximum daily and monthly median numeric toxicity effluent limitations, use of the TST for evaluating compliance, monthly monitoring², 24-hr notification to the Regional Water Board of violations, and requirements to perform a Toxicity Reduction Evaluation (TRE) after two consecutive months of effluent limit exceedance.

Throughout development of the Toxicity Provisions, stakeholders worked with the State Water Board, expressing concerns over requirements or implementation and how they might be resolved. A major issue was the *Ceriodaphnia dubia* (*C. dubia*) short term chronic toxicity tests with the reproduction endpoint (Cerio Test) and its suitability for use as a regulatory limit. For many wastewater treatment plants that discharge to freshwater (including the LBWRP), *C. dubia* is the most sensitive species and is required for use in compliance assessments under the Toxicity Provisions. The Toxicity Provisions require use of the TST method to evaluate test data, which is sensitive to the test's Coefficient of Variation (intra-lab variability). As a result, tests that passed under the No Observable Effect Concentration (NOEC) evaluation (for example) may fail and be deemed toxic under the TST. Analysis of historic Cerio Test results pointed to higher intra-lab variability than tests with other species, which would result in determinations of toxicity due to the statistical calculation and not from toxicity of the sample. Additionally, the inter-lab variability was found to be substantial with split samples sent to different laboratories which can

¹ The toxicity provisions were originally adopted by the State Water Board on December 1, 2020. Due to an administrative issue, the toxicity provisions were re-adopted on October 5, 2021. Formal approval by the EPA was issued on May 1, 2023.

² Monthly monitoring is required for facilities that are authorized to discharge ≥ 5.0 mgd and at least 15 days of discharge are expected during the calendar month.

result in differing determinations of toxicity. Three studies conducted by the U.S. Environmental Protection Agency (EPA), Southern California Stormwater Monitoring Coalition, and the Western Coalition of Arid States utilized blanks (non-toxic lab water) in the evaluation of laboratories. In these studies, blank samples were identified as toxic at a rate between 14 and 55 percent. The studies varied in size (number of labs and number of samples evaluated) and they were not specifically designed to evaluate the rate of incorrect determinations of toxicity. However, the data were used to demonstrate variability in the Cerio Test. While the *C. dubia* method is well established, it does allow a degree of latitude in procedures that may lead to intra- and inter-laboratory variability and poor comparability. Through the stakeholder process for the Toxicity Provisions, an agreement was formed to delay use of the Cerio Test for compliance in new permits until January 1, 2024 allowing time to conduct the "Quality Assurance Evaluation of the *Ceriodaphnia dubia* Reproduction Test" (Cerio Study).

The scope of the Cerio Study³, as defined by the contract between the State Water Board and the Southern California Coastal Water Research Project (SCCWRP), identifies both intra- and inter-laboratory variability as issues to investigate. Funding provided by the State Water Board for the Cerio Study allows for an evaluation of existing data to assess possible causes of intra-laboratory variability and follow-up testing to evaluate methods of controlling intra-laboratory variability. However, the State Water Board funding is insufficient to fully investigate the inter-laboratory variability. The inter-laboratory variability is the biggest concern for the regulated community, with the desired outcomes being methods and test requirements that will result in the same determinations of toxicity for samples split and analyzed by different labs. To that end, California Association of Sanitation Agencies members have committed additional funding and in-kind testing of approximately \$500,000 to augment the inter-variability analysis of the Cerio Study. The Cerio Study results will be considered by the State Water Board in late 2023, and appropriate action will be taken by the State Water Board to revise the evaluation or compliance procedures.

The Cerio Study uses a shared governance structure that includes a Stakeholder Advisory Committee comprised of regulatory agencies, regulated parties, and non-governmental organizations, and an Expert Panel comprised of national experts in toxicity testing. To date, the available control data from all California labs certified to perform the Cerio Test were obtained and analyzed by SCCWRP. The analysis was intended to identify factors that may contribute to intra-lab variability. However, after extensive review by the SAC and Expert Panel, no such indicators were evident, but each lab performed the test slightly differently highlighting the flexibility of the method. The second phase of the study included preparing various sample waters for the participating labs to analyze with specific instructions to gather and report data. Again, after review by the Expert Panel, no clear indicators could be determined for the cause of variability observed in the Cerio Test. In the split sampling, the leading cause for test acceptability failures were culture crashes. In the last phase of the study, labs will coordinate and discuss the standard of practice for maintaining food cultures and brood stocks, and another round of intercalibration will be performed. One goal of the second round of lab testing is to determine if labs will have the same level of variability observed in the first round. The second round of intercalibration is due to conclude in June 2023, and a draft study report was released on August 31, 2023. The final report with recommendations is due in late September/early October 2023 and a State Water Board hearing/workshop is scheduled for October 17, 2023.

³ <https://www.sccwrp.org/about/research-areas/additional-research-areas/ceriodaphnia-toxicity-testing-quality-assurance/>

Biostimulation, Cyanotoxins, and Biological (B&C&B) Condition Provisions (formerly Biointegrity and Biostimulation, B&B): The State Water Board is considering statewide WQOs for nutrients, other biostimulatory substances, and cyanotoxins, and a program of implementation under the B&C&B Condition Provisions.⁴ Currently under consideration are statewide numeric or narrative WQOs and regulatory control options for point and non-point sources in California's freshwater wadeable streams and rivers, non-wadeable streams and rivers, lakes, and reservoirs. While incident sunlight, temperature, flowrate, substrate, and other factors play a major role in B&C&B, the regulatory focus is generally limited to controlling nutrients. The B&C&B provisions will be established as a statewide policy for water quality control and will include a water quality control plan component. While the State Water Board holds the B&C&B provisions as priority⁵ for 2023, there has been little movement on developing the provisions.

Constituents of Emerging Concern (CECs): CECs encompass a vast number of chemical substances that are generally unregulated in the United States or have limited regulation in environmental media (e.g., air, water, sediment, and biota) around the world. CECs may include a wide variety of substances ranging from pharmaceuticals to flame retardants or per- and polyfluoroalkyl substances (PFAS) to newly registered contemporary use pesticides and newly developed commercial products such as nanomaterials. Advances in qualitative and quantitative analytical chemistry have allowed detection in various environmental media and have led to initiatives to estimate the potential hazard of CECs. Ongoing CEC evaluations, future requirements, and constituents of interest are discussed below:

1. **Aquatic Ecosystem Impacts.** Classes of CECs can be evaluated through bioassays. Bioassay monitoring requirements may be included in future permits to assess the impacts of effluent discharges on aquatic ecosystems. At the request of the State Water Board, a Science Advisory Panel developed and recommended a risk-based screening framework to identify CECs for monitoring in California's aquatic ecosystems in 2012. The 2012 Science Advisory Panel applied the framework using existing information to three representative receiving water scenarios to identify a list of appropriate CECs for initial monitoring, developed an adaptive phased monitoring approach, and suggested development of bioanalytical screening and predictive modeling tools to improve assessment of the presence of CECs and their potential risk to the environment. The State Water Board in conjunction with the Ocean Protection Council and a group of stakeholder advisors re-convened a group of leading scientists in October 2020 to address the issues associated with CECs in the State's aquatic systems. The 2022 Science Advisory Panel was comprised of seven experts in chemistry, biochemistry, toxicology, chemical and risk assessment, engineering, and coastal and marine environmental health science. The Science Advisory Panel worked with State Water Board Division of Water Quality to develop an updated risk screening approach and an initial prioritization of CEC compounds of interest based on a complex statewide CEC dataset, as well as compound-specific toxicological data. The 2022 Science Advisory

⁴ https://www.waterboards.ca.gov/water_issues/programs/biostimulatory_substances_biointegrity/index.html

⁵ The State Water Board appears to be more focused on Ocean Acidification and Hypoxia than the B&C&B Provisions.

Panel released a final report in December 2022⁶ which resulted in the recommendations for four products that will help the state develop a "monitoring process for CECs based on sound, up-to-date scientific principles."

Guidance to structure, quality assurance and visualization of CECs covered by the existing State Water Board CEC dataset.

Following the data collection recommendations of the 2012 Science Advisory Panel, over 280,000 records were available for review from freshwater systems. Only 12 percent of those records were detected, but the detections do not necessarily imply risk or hazard, nor do non-detects necessarily imply lack of risk or hazard. The Science Advisory Panel reviewed the status and quality of the dataset provided and established a quality assurance workflow for CEC monitoring data. Since the data came from many different sources with various data quality standards,⁷ a substantial effort was required to reconcile the differences and highlighted the need for statewide guidance on data quality objectives for CEC monitoring. The Science Advisory Panel recommended the State Water Board categorize the quality of data and potential uses of the data from each of the databases such that expected shortcomings of the data quality and acceptance criteria are documented as part of applying the proposed prioritization framework.

Guidance to use other sources to inform a CEC monitoring program.

The Science Advisory Panel also used additional multimedia-occurrence data, such as other state, national, and international sources reporting on CECs, that are not currently included in California's ongoing previous or existing monitoring programs. In total 133 compounds were included in a "new CECs" list. A subset of these compounds was assessed by the Science Advisory Panel. Sixteen compounds were selected for prioritization including 6PPD-quinone, a compound highly toxic to coho salmon (*Oncorhynchus kisutch*) and salmonid species.

An updated risk-based approach to assess and identify CECs for monitoring in California receiving waters.

Because the data required substantial time to clean and process into a usable form, the Science Advisory Panel only provided a "proof of concept" update and an expanded risk and occurrence based on a monitoring prioritization framework. The expanded framework includes the following four steps:

- a. Toxicity Assessment: develop monitoring trigger levels (MTLs) for CECs with the greatest risk to aquatic systems.
- b. Preliminary Monitoring Prioritization: rate CECs based on the measured environmental concentrations (MECs) and trends for the CECs where MTLs could be estimated.
- c. Refined Monitoring Prioritization: prioritize CECs based on sample size, trends, and monitoring trigger quotient (MTQ = MEC/MTL).
- d. Recommended Monitoring Program: specify the nature of the local, regional, and statewide monitoring efforts.

⁶ https://www.waterboards.ca.gov/water_issues/programs/cec/index.html

⁷ For example, the lack of the latitude/longitude information for 27% of surface water results severely limited the utility of the data.

Establish sound foundation for State-wide and regional CEC monitoring in California.

The Science Advisory Panel provided a model and a roadmap to be used by the Water Boards for implementation of the statewide CEC monitoring program. The Science Advisory Panel provided recommendations to complement the continued risk-based monitoring approach with temporal and spatial evaluations, improve the quality of data reported to the State Water Board, update MTLs as they are available, develop a pilot biomonitoring program focused on identifying early effects in ambient waters, and update the State's approach to CEC monitoring and management. The overarching concern from the Science Advisory Panel was the challenge of using the existing databases and systems to collect and compile occurrence data. Focused statewide monitoring may be the only solution to gather meaningful data if the existing databases and systems cannot be unified.

2. **PFAS.** The State Water Board Division of Drinking Water (DDW) has issued Notification Levels (NLs) and Response Levels (RLs) for perfluorobutanesulfonic acid (PFBS), perfluorooctanoic acid (PFOA), perfluorooctane sulfonate (PFOS), and perfluorohexane sulfonate (PFHxS). The Office of Environmental Health Hazard Assessment (OEHHA) recommended an NL for perfluorohexanoic acid (PFHxA) to DDW on September 8, 2024.

OEHHA adopted Public Health Goals (PHGs) for PFOA and PFOS on April 5, 2024. Establishing a PHG is the first step towards adopting drinking water maximum contaminant levels (MCLs). The PFOS and PFOA MCL rulemaking process is underway with plans for adoption of final MCLs in 2025. The MCLs would apply to discharges to receiving waters with the MUN or groundwater recharge (GWR) beneficial use.

The EPA's "PFAS Strategic Roadmap"⁸ was released on October 18, 2021. The document describes timelines and actions that will be taken to "safeguard public health, protect the environment, and hold polluters accountable." In particular, the EPA is developing national drinking water regulations, monitoring guidance for NPDES permits, ambient water quality criteria for PFAS, and human health criteria for PFAS. A second-year progress report was issued by the EPA in December 2023 and a third-year progress report was issued in December 2024.

On April 10, 2024, the EPA announced the final PFAS National Primary Drinking Water Regulations.⁹ The regulations establish legally enforceable MCLs and non-enforceable health-based maximum contaminant level goals (MCLGs) for six PFAS compounds (PFOA, PFOS, perfluorononanoic acid [PFNA], PFBS, hexafluoropropylene oxide dimer acid (HFPO-DA or GenX), and PFHxS) as well as monitoring, public notification, and treatment requirements. The final rule requires public water systems to conduct three years of initial monitoring (by 2027) followed by ongoing compliance monitoring, notify the public of PFAS levels in the water supply (by 2027), and implement treatment or other controls to reduce PFAS levels below the MCLs within five years (by 2029).

⁸ <https://www.epa.gov/pfas/pfas-strategic-roadmap-epas-commitments-action-2021-2024>

⁹ <https://www.epa.gov/sdwa/and-polyfluoroalkyl-substances-pfas>

In September 2024, the EPA released final recommended aquatic life water quality criteria for PFOA and PFOS¹⁰, presented in Table 2.4. The final criteria represent a scientific assessment of the ecological effects of PFOA and PFOS on aquatic organisms. The criteria are national recommended aquatic life criteria that can be adopted by states and tribes as water quality standards for specific waterbodies and beneficial uses. Since the values are non-regulatory, states and tribes have the option to modify the criteria based on site-specific conditions or develop other numeric criteria that are scientifically based and protective of designated beneficial uses. The September 2024 update also included the EPA’s recommended acute saltwater aquatic life benchmarks for PFOA and PFOS and the recommended acute freshwater aquatic life benchmarks for eight additional PFAS compounds. In addition to the water column criteria, the EPA recommended final tissue-based criteria for fish (whole body, muscle) and invertebrates (whole body).

Table 2.4 PFOA and PFOS EPA Recommended Aquatic Life Water Quality Criteria

Compound	Freshwater Acute Water Column	Freshwater Chronic Water Column	Estuarine/Marine Acute Water Column Benchmark
PFOA	3.1 mg/L	0.1 mg/L	7.0 mg/L
PFOS	0.071 mg/L	0.00025 mg/L	0.55 mg/L

In December 2024, the EPA released draft national recommended human health ambient water quality criteria for PFOA, PFOS, and PFBS¹¹. The criteria are recommended to prevent exposure when drinking water or eating fish and shellfish from polluted water bodies. There are two human health criteria assigned for each of the PFAS compounds, one criteria based on consuming water and organisms and one criteria based on consuming organisms only. The EPA also provided an illustrative example for states or tribes to develop a water quality standard for a mixture of two or more PFAS compounds using the Hazard Index.

On December 5, 2022, the EPA released supplemental guidance¹² to States on how to use the NPDES permit program to address PFAS pollution in wastewater. The purpose of the guidance is to provide information to permit writers on how to use existing authorities (such as industrial pretreatment program provisions and monitoring requirements) in NPDES permits as the EPA continues to finalize its Effluent Limitation Guidelines for PFAS. The guidance suggests that publicly owned treatment works (POTWs) identify and locate all possible industrial users that may be subject to a PFAS pretreatment program, along with the volume of pollutants contributed to the POTWs by the industrial users. With regards to biosolids, the guidance recommends (where appropriate) that states work with POTWs to reduce PFAS in biosolids through the following steps: (1) Use EPA method 1633 to analyze presence of PFAS in biosolids, (2) indicate the presence of PFAS in biosolids from industrial sources, and (3) validate PFAS reductions with regular monitoring of biosolids.

¹⁰ <https://www.epa.gov/system/files/documents/2024-09/pfoa-pfos-pfas-final-factsheet-2024.pdf>

¹¹ <https://www.epa.gov/system/files/documents/2024-12/draft-hhc-pfas-tech-fact-sheet.pdf>

¹² <https://www.epa.gov/newsreleases/epa-issues-guidance-states-reduce-harmful-pfas-pollution>

EPA Method 1633 was published on January 31, 2024, and is now recommended for NPDES permit compliance monitoring.¹³ The method specifies testing procedures for 40 PFAS compounds in wastewater, surface water, groundwater, soil, biosolids, sediment, landfill leachate, and fish tissue. Use of EPA Method 1633 is not required for NPDES programs until the EPA completes its formal rulemaking process under the Clean Water Act which is expected to occur in 2025.

3. **Microplastics.** Microplastics are being studied by the State Water Board to assess the environmental impact of the plastics on aquatic organisms. Specifically, the two methods of microplastics impacts are food dilution (e.g., starvation due to filling the gut with microplastics instead of food) or translocation of microplastic particles into tissues/organs.¹⁴

Microplastics are largely removed via surface skimming during primary treatment and are then sent to the landfill. Any remaining microplastic particles captured during later treatment phases would end up in the biosolids. Food waste slurry serves as a major source of microplastics to the wastewater stream and was identified by the EPA as a major source of microplastics to biosolids¹⁵. The current concern is regarding loadings of microplastic particles from effluent discharged to surface waters and from biosolids which are land applied. A statewide POTW microplastics treatment efficiency study was funded by the Ocean Protection Council in 2019¹⁶ and will serve to inform the State Water Board and legislators about the amount of microplastics being discharged to receiving waters via wastewater effluent and biosolids.

A definition of microplastics in drinking water was adopted by the State Water Board in July 2020¹⁷ and this definition may be applied to other matrices in the future. The State Water Board is currently developing draft standard methods for analyzing microplastics in drinking water, guidelines for accrediting qualified laboratories, a 4-year plan for testing and reporting in drinking water, and quantitative guidelines for interpreting the results.¹⁸ The "Draft Microplastics in Drinking Water Policy Handbook"¹⁹ was released for public comment on November 10, 2021.

Related guidelines for wastewater may result in future monitoring requirements after standardized methods are developed), limitations for discharges to receiving waters with the MUN and GWR designated beneficial uses, and water bodies being listed as impaired due to exceeding microplastics loading thresholds. There is little evidence of groundwater contamination by microplastics and this is not a likely pathway of future

¹³ <https://www.epa.gov/cwa-methods/cwa-analytical-methods-and-polyfluorinated-alkyl-substances-pfas#background>

¹⁴ Information provided by Shelly Walther (Sanitation Districts), November 2021.

¹⁵ Emerging Issues in Food Waste Management – Plastic Contamination, EPA Office of Research and Development, EPA/600/R-21/116, August 2021, <https://www.epa.gov/system/files/documents/2021-08/emerging-issues-in-food-waste-management-plastic-contamination.pdf>

¹⁶ https://opc.ca.gov/webmaster/ftp/pdf/agenda_items/20200619/Item9_MicroplasticsProjects_FINAL.pdf

¹⁷ State Water Board Resolution No. 2020-0021.

¹⁸ https://www.waterboards.ca.gov/drinking_water/certlic/drinkingwater/microplastics.html

¹⁹ https://www.waterboards.ca.gov/drinking_water/certlic/drinkingwater/documents/microplastics/mcrplsts_plcy_drft.pdf

concern since tertiary treated wastewater contains negligible amounts of microplastics.²⁰

4. **Antibiotic Resistant Genes (ARG) and Antibiotic Resistance Bacteria (ARB).** ARG/ARB were identified as potential recycled water CECs in 2017. SCCWRP is investigating the presence of ARG/ARB in treated wastewater effluent and whether it is discharged in a viable form after treatment.²¹ SCCWRP is currently identifying antibiotic resistant isolates during culture sampling and testing the bacteria for multi-drug resistance. Results have not yet been released for technical review and it is unknown if ARG/ARB could impact groundwater supplies through indirect potable reuse.

Basin Plan Amendments and Total Maximum Daily Loads (TMDLs): The Basin Plan is periodically amended to address changes from the triennial review and to incorporate TMDL requirements. Basin Plan Amendments could result in new effluent limitations and permit requirements if WQOs are added/revised or if TMDLs are developed for the San Gabriel River Watershed. TMDLs are implemented to address constituents that exceed WQOs in the receiving water and can result in additional requirements for discharges to the affected surface waters. The current NPDES permit implements requirements from the San Gabriel River Metals and Selenium TMDL (established by the EPA in 2007), the San Gabriel River, Estuary, and Tributaries Indicator Bacteria TMDL (established by the Regional Water Board in 2015), and the Dominguez Channel and Greater Los Angeles and Long Beach Harbor Toxics TMDL (established by the Regional Water Board in 2012). Additional TMDLs may be developed in the future to address water quality impairments in Coyote Creek and the San Gabriel River Estuary. Coyote Creek is currently included on the Clean Water Act Section 303(d) list of impaired water bodies for malathion, iron, copper, toxicity, pH, and indicator bacteria. The San Gabriel River Estuary is on the 303(d) list for nickel, dissolved oxygen, copper, dioxin, and indicator bacteria. The Clean Water Act Section 303(d) list is undergoing a review and update through the 2024 California Integrated Report, where updated listing and delisting recommendations will be provided.

2.2.3 Compliance Evaluation

The specific constituents identified as NPDES permit compliance concerns are discussed below.

2.2.3.1 Temperature

A maximum daily effluent limit of 80 degrees F with no exceptions for ambient temperature is included in the 2022 NPDES permit. As described previously, the Sanitation Districts have been granted an interim limit and a 10-year in-permit compliance schedule to achieve compliance with the new, more stringent interpretation of the narrative WQO in the Basin Plan. The interim maximum daily limit is 86 degrees F based on the limit in the 2015 NPDES permit and allows excursions greater than 86 degrees F if they are a result of external ambient temperature. Recent effluent temperature measurements are compared to the interim and final effluent limitations in Figure 2.2. The 10-year compliance strategy includes studies during the first 5-years to evaluate pretreatment, source control, and other temperature management options to reduce effluent temperature. The preferred approach must then be selected and implemented during the second 5-year period.

²⁰ Information provided by Shelly Walther (Sanitation Districts), November 2021.

²¹ <https://www.sccwrp.org/news/bacterial-species-genes-being-identified-for-study-examining-antibiotic-resistance-in-wastewater-effluent/>

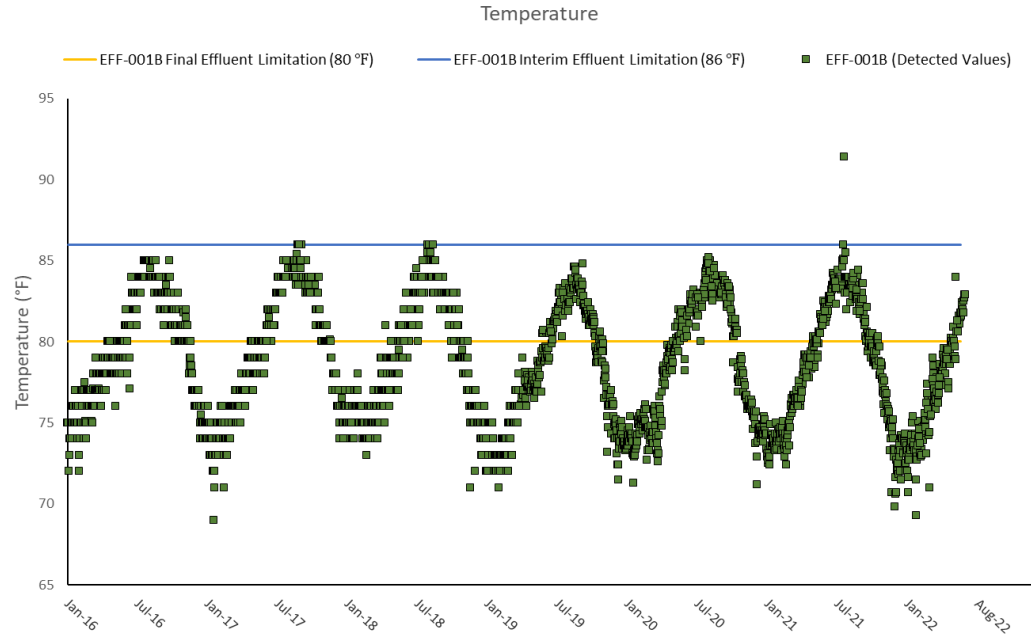


Figure 2.2 LBWRP Effluent Temperature 2016 to 2022

2.2.3.2 Chronic Toxicity

Several of the JOS facilities have experienced chronic toxicity exceedances. Toxicity Identification Evaluations (TIE) efforts are ongoing to determine/address possible causes and the Sanitation Districts is currently reviewing a report on the study results. Recent chronic toxicity test results for LBWRP effluent are presented in Table 2.5 and Table 2.6. Effluent monitoring is conducted monthly to evaluate compliance with the average monthly and maximum daily limits. Samples are collected at Monitoring Location EFF-001A, after disinfection but prior to dechlorination.

Table 2.5 LBWRP Chronic Toxicity Results (2016-2022)

Monitoring Location	Toxicity Test (Species Tested)	Result			% Fail
		N/A ⁽¹⁾	Fail	Pass	
EFF-001A	Chronic Toxicity (<i>C. dubia</i>) - Reproduction	-	5	80	6
EFF-001A	Chronic Toxicity (<i>C. dubia</i>) - Survival	85	-	-	-
Total		85	5	80	6

Notes:

(1) The TST statistical procedure is not amenable to the *Ceriodaphnia* survival endpoint.

Table 2.6 LBWRP Chronic Toxicity TST "Fail" Occurrences (2016-2022)

Monitoring Location	Date	Toxicity Test (Species Tested)	TST Result	% Effect
EFF-001A	Jan 2016	Chronic Toxicity (<i>C. dubia</i>) - Reproduction	Fail	26.6
EFF-001A	Jul 2016	Chronic Toxicity (<i>C. dubia</i>) - Reproduction	Fail	72.7
EFF-001A	Jan 2017	Chronic Toxicity (<i>C. dubia</i>) - Reproduction	Fail	-15.2
EFF-001A	Oct 2017	Chronic Toxicity (<i>C. dubia</i>) - Reproduction	Fail	24.5
EFF-001A	Oct 2018	Chronic Toxicity (<i>C. dubia</i>) - Reproduction	Fail	23.9

Species sensitivity screening must be conducted within the first 18 months after the effective date (April 1, 2022) of the current permit (R4-2022-0032) using *Ceriodaphnia Dubia* (water flea), *Pimephelas Promelas* (Fathead Minnow), and *Selenastrum* (green algae). Routine monitoring for permit compliance is currently conducted using *Ceriodaphnia Dubia*, the most sensitive species identified during the previous screening process. *Ceriodaphnia Dubia* is anticipated to be the LBWRP compliance species when new screening is conducted. However, the State Water Board has convened a panel²² that will make recommendations to improve quality assurance for *Ceriodaphnia Dubia* testing and eliminate the intra-laboratory variability associated with this species. When implemented, the panel's recommendations may improve compliance results for LBWRP.

2.2.3.3 Ammonia

Recent effluent ammonia concentrations are compared to the current NPDES permit limits in Figure 2.3. Effluent monitoring is conducted monthly to evaluate compliance with the average monthly and maximum daily limits. While six results were measured above the average monthly effluent limit, additional monthly sampling was conducted and the monthly average calculation produced just one effluent limit exceedance. This exceedance is believed to have been caused by ongoing construction within the WRP.

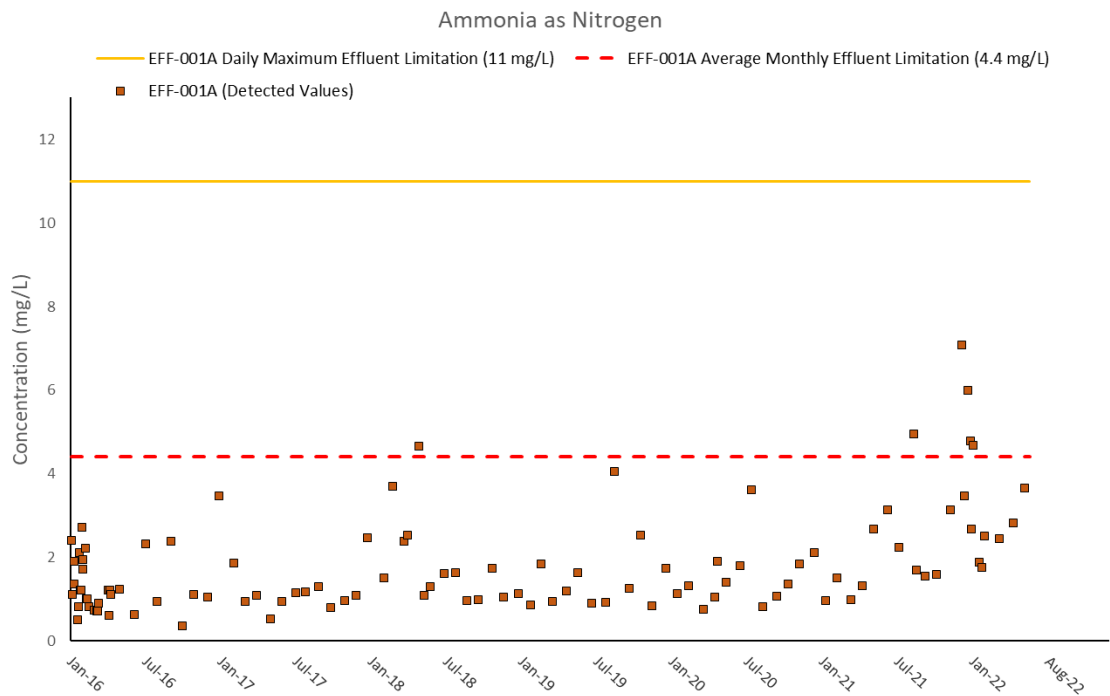


Figure 2.3 LBWRP Effluent Concentrations of Total Ammonia (as N) 2016 to 2022

The ammonia effluent limits are expected to decrease when the Regional Water Board amends the Basin Plan to include the EPA's 2013 freshwater ammonia criteria.²³ The Regional Water Board started the Basin Plan Amendment process in April 2023 with release of the proposed

²² <https://www.sccwrp.org/about/research-areas/additional-research-areas/ceriodaphnia-toxicity-testing-quality-assurance/>

²³ <https://www.sccwrp.org/news/bacterial-species-genes-being-identified-for-study-examining-antibiotic-resistance-in-wastewater-effluent/>

Basin Plan Amendment and Staff Report for public review. Based on comments received on the draft documents, it was determined additional time was needed to prepare responses and the adoption hearing was postponed. The new hearing date has not yet been scheduled. After adoption by the Regional Water Board, the Basin Plan Amendment will also have to be adopted by the State Water Board, and then approved by the EPA before it can be implemented.

The lowest WQOs are based on the presence of freshwater mussels. However, the Regional Water Board recognizes that freshwater mussels have been extirpated from the Los Angeles Coastal Watershed Region and salmonids are not present in the San Gabriel River watershed. As a result, the new ammonia WQOs will be based on the presence/absence of fish early life stages (ELS). The proposed Basin Plan Amendment will supersede the current site-specific ammonia WQOs and establish seasonal limits based on ELS absent from April 1 to September 30 and ELS present from October 1 to March 31. POTW comments on the Basin Plan Amendment are anticipated to support retention of the existing site-specific WQOs and ELS absent condition.

Current and projected ammonia effluent limits based on existing site-specific objectives and the 2013 EPA criteria are shown in Table 2.7. The *Proposed Amendment to the Los Angeles Water Quality Control Plan (Basin Plan) – To incorporate 2013 United States Environmental Protection Agency "Aquatic Life Ambient Water Quality Criteria for Ammonia – Freshwater" in the Los Angeles Region* (Staff Report) and receiving water and effluent data reported between 2015 and 2021 were used to calculate the projected effluent limits. It is anticipated the Regional Water Board will continue its current practice of using the 50th percentile pH and temperature receiving water measurements to represent the chronic condition and 90th percentile pH and temperature receiving water measurements to represent the acute condition.

Table 2.7 Current and Projected Effluent Limits for Ammonia (as N)

1.5	Mussels absent, salmonids absent, ELS absent ⁽³⁾⁽⁴⁾ Chronic Conditions: pH = 7.5 at 50th percentile, T = 24.5 degrees C Acute Conditions: pH = 7.7 at 90th percentile, T = 27.956 degrees C		Mussels absent, salmonids absent, ELS present ⁽³⁾⁽⁴⁾ Chronic Conditions: pH = 7.9 at 50th percentile, T = 24.9°C Acute Conditions: pH = 8.6 at 90th percentile, T = 28.6 degrees C	
	AMEL (mg/L)	MDEL (mg/L)	AMEL (mg/L)	MDEL (mg/L)
Current Site Specific Objectives	4.4	11	--	--
Proposed EPA 2013 Criteria ⁽¹⁾⁽²⁾	3.7 Chronic conditions drive effluent limits	9.3 Chronic conditions drive effluent limits	1.1 Acute conditions drive effluent limits	1.6 Acute conditions drive effluent limits

Notes:

Abbreviations: AMEL - average month effluent limit; C - Celsius.

- (1) Proposed Amendment to the Basin Plan - To incorporate 2013 EPA "Aquatic Life Ambient Water Quality Criteria for Ammonia – Freshwater" in the Los Angeles Region (Staff Report).
- (2) The 2022 NPDES Permit coefficient of variability (CV) values for ELS absent were utilized in the calculations. The CV for ELS present was calculated using effluent ammonia data reported in California Integrated Water Quality System (CIWQS) (2015 to 2021).
- (3) The 50th and 90th percentile pH values (chronic and acute) and 50th percentile temperature (chronic) for ELS absent were obtained from the 2022 NPDES Permit. The 90th percentile temperature for ELS absent and ELS present, the 50th percentile temperature for ELS present, and the 50th and 90th percentile pH values for ELS present were calculated using data from the respective receiving water reported in CIWQS (2015 to 2021).
- (4) ELS present occurs from October 1 - March 31 and ELS absent occurs from April 1 – September 30.

In addition, activities to comply with future B&C&B provisions may include reductions in effluent total nitrogen or total inorganic nitrogen concentrations which will involve ammonia removal. Additional studies are needed to determine applicable treatment processes and achievable effluent concentrations.

2.2.3.4 Nitrite

Recent effluent nitrite concentrations are compared to the NPDES permit limit in Figure 2.4. Effluent monitoring is conducted monthly to evaluate compliance with the average monthly limit in the NPDES permit. While four results were measured above the average monthly effluent limit, additional monthly sampling was conducted and the monthly average calculation produced no effluent limit exceedances.

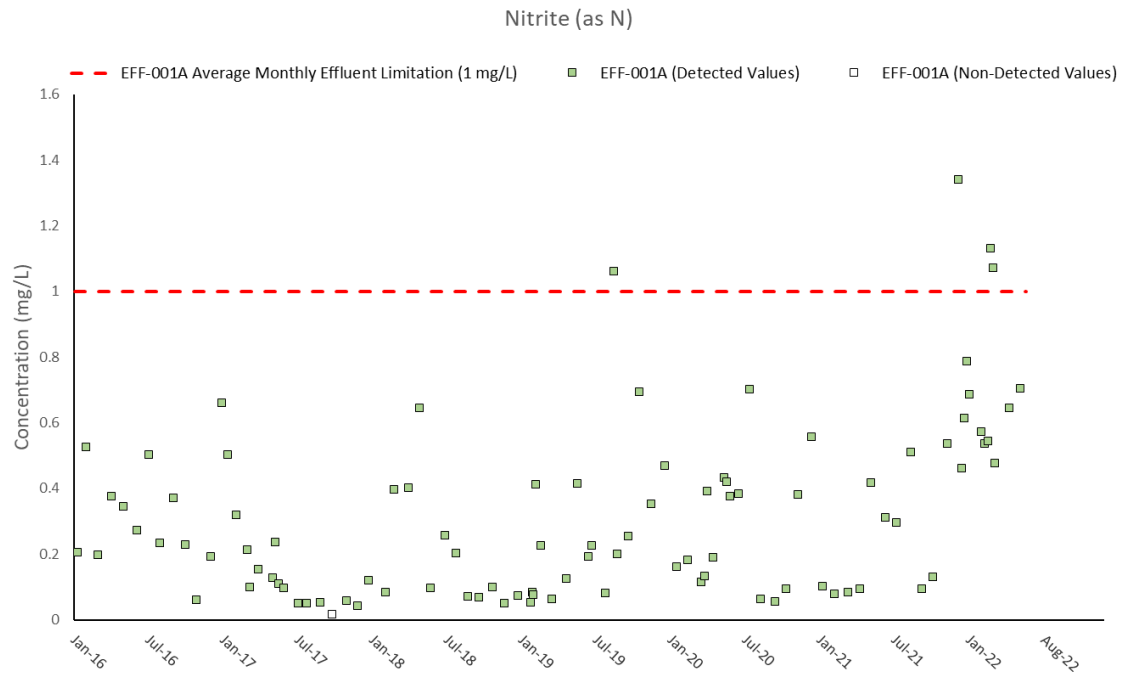


Figure 2.4 LBWRP Effluent Concentration of Nitrite (as N) 2016 to 2022

Similar to ammonia, activities to comply with future B&C&B provisions may include reductions in effluent total nitrogen or total inorganic nitrogen concentrations which involve nitrite removal. Additional studies are needed to determine applicable treatment processes and achievable effluent concentrations.

2.2.3.5 Nitrate Plus Nitrite

Recent effluent nitrate plus nitrite concentrations are compared to the NPDES permit limit in Figure 2.5. Effluent monitoring is conducted monthly to evaluate compliance with the average monthly limit in the NPDES permit. There were a few exceedances of the monthly average limit in recent years, which are believed to have occurred during a period of ongoing construction at the LBWRP. In addition, water conservation activities may have resulted in increased concentrations. Activities to comply with future B&C&B provisions may include reductions in effluent total nitrogen or total inorganic nitrogen concentrations which will involve nitrate/nitrite

removal. Additional studies are needed to determine applicable treatment processes and achievable effluent concentrations.

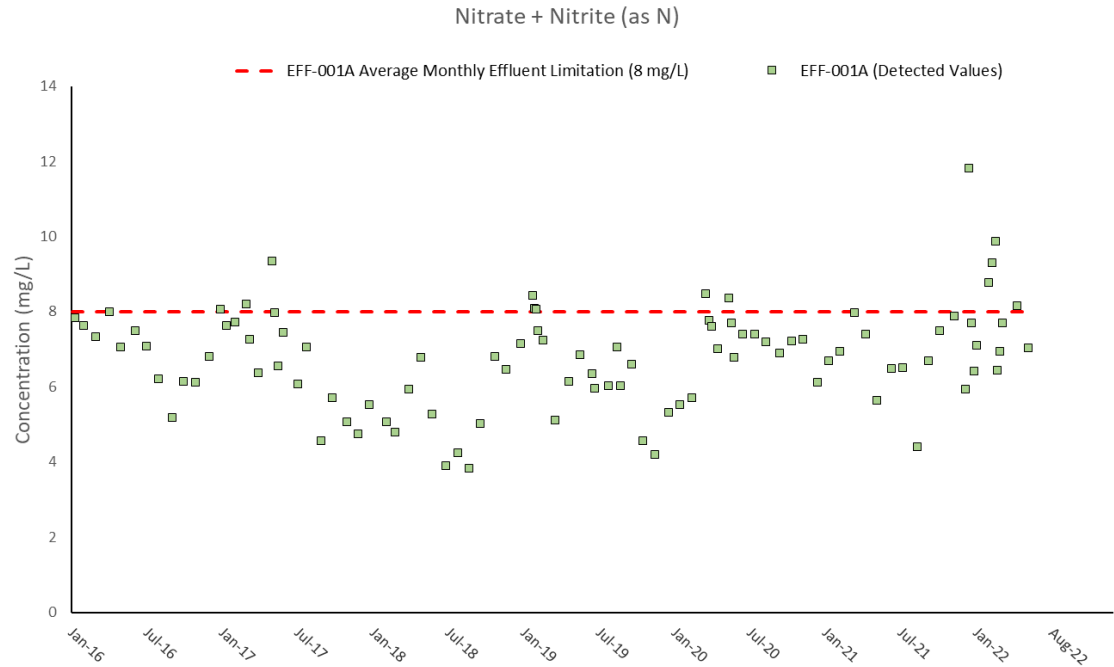


Figure 2.5 LBWRP Effluent Concentrations of Nitrate Plus Nitrite (as N) 2016 to 2022

2.2.3.6 Total Coliform

Total coliform results obtained in recent years are compared to the NPDES permit limits in Figure 2.6. Total coliform is monitored daily to evaluate compliance with the average monthly, average daily, and maximum daily limits in the NPDES permit. The effluent limitations for total coliform in the NPDES permit are based on the requirements and averaging period specified in Title 22 (7-day rolling median, no more than one sample >23 most probable number (MPN) in any 30-day period, no sample >240 MPN). Based on these requirements, no effluent limit exceedances occurred from 2016 to 2022.

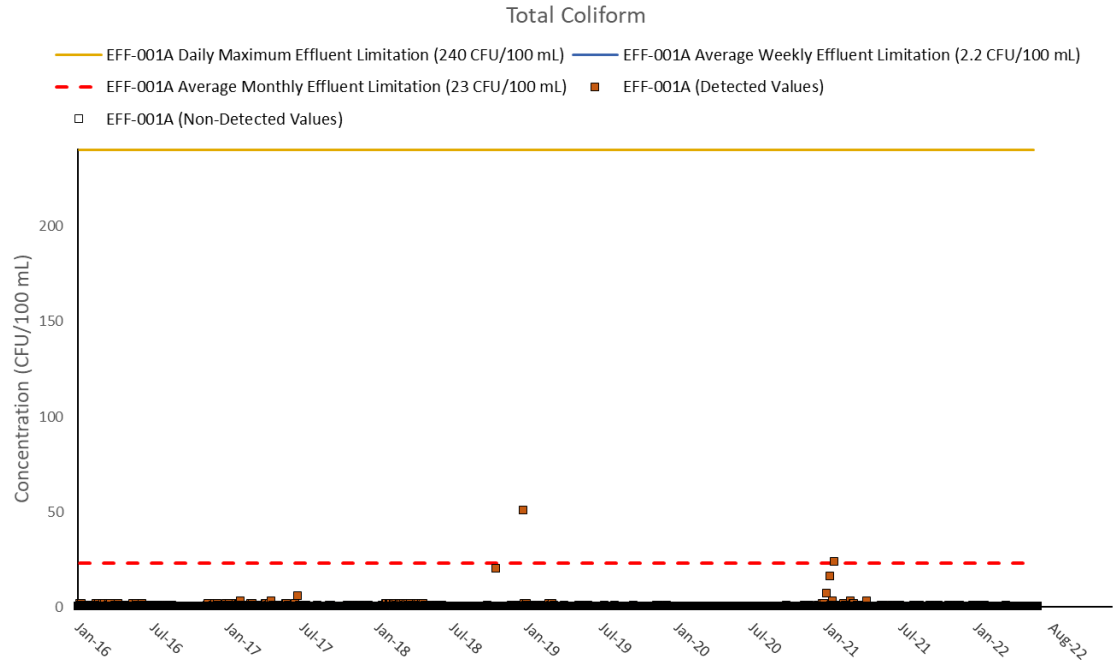


Figure 2.6 LBWRP Effluent Total Coliform 2016 to 2022

2.2.3.7 Total Residual Chlorine

Recent total residual chlorine effluent concentrations are compared to the NPDES permit limit in Figure 2.7. There were eight exceedances of the NPDES permit limit from 2016 to 2022. The Sanitation Districts have determined that operation of recycled water pumps and increases in recycled water demand affect residual chlorine within the system.

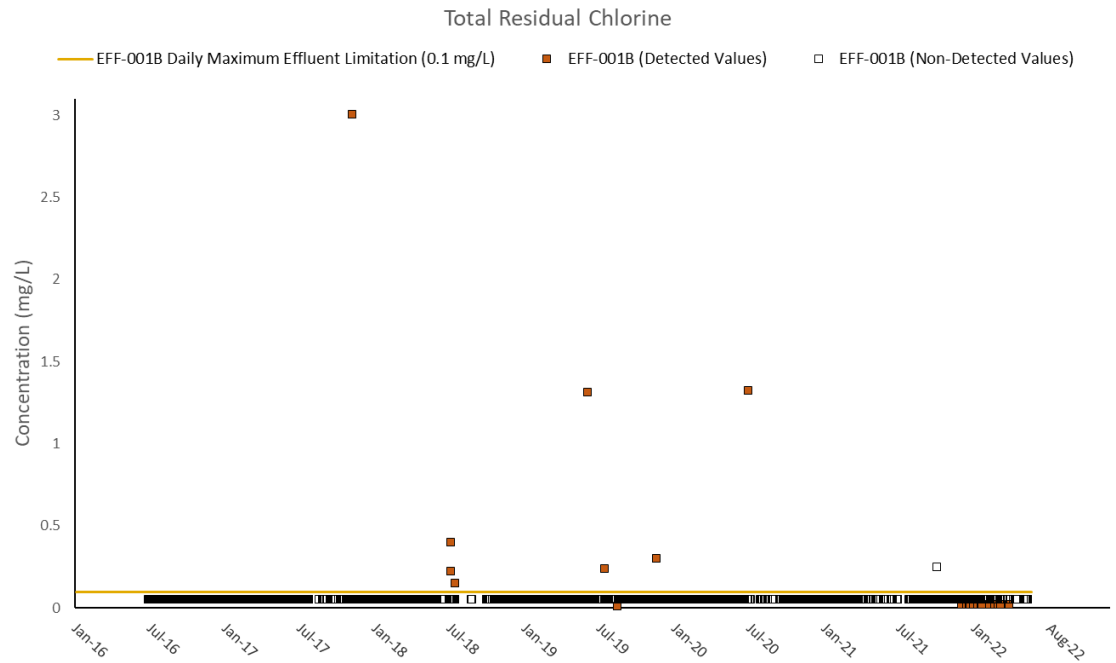


Figure 2.7 LBWRP Effluent Concentrations of Total Residual Chlorine 2016 to 2022

2.2.3.8 Per- and Polyfluoroalkyl Substances

As discussed previously, the EPA has proposed freshwater aquatic life criteria for PFOA and PFOS that could be implemented in NPDES permits to protect the WILD, WARM, MIGR, and SPWN beneficial uses. The proposed criteria are shown in Table 2.4. All effluent concentrations measured from 2016 to 2022 were much lower than the proposed criteria. The maximum effluent concentration of PFOA was 15 nanograms per liter (ng/L), compared to the chronic criteria of 94,000 ng/L. The maximum effluent concentration of PFOS was 8.5 ng/L, compared to the chronic criteria of 8,400 ng/L.

2.2.4 Proposed Strategies for Compliance

An overall strategy for compliance is provided in the summary section (Section 2.5).

2.3 Potable Recycled Water

Disinfected tertiary recycled water produced at the LBWRP is the source water for the Vander Lans WTF and is applied at the Alamitos Seawater Intrusion Barrier. The Vander Lans WTF provides further treatment of the LBWRP recycled water using microfiltration, reverse osmosis (RO), advanced oxidation (ultraviolet light), decarbonation, pH adjustment, and corrosivity stabilization prior to blending with potable water. The advanced treated recycled water is then mixed with potable water and injected into the Orange County Groundwater Basin to prevent seawater intrusion. The Vander Lans WTF is owned by WRD and operated and maintained by the City of Long Beach. The Alamitos Barrier facilities are co-owned and operated by LACDPW and the Orange County Water District (OCWD).

2.3.1 Current Requirements

The WDRs/WRRs for the Alamitos Barrier Recycled Water Project (Order No. R4-2014-0111) include the following requirement that applies to LBWRP effluent.

Provision I. The influent to the Vander Lans WTF shall be tertiary treated effluent as described in the approved 2013 Title 22 Engineering Report and shall at all times be adequately oxidized.

2.3.2 Future Requirements

If additional requirements are applied or new potable reuse projects are planned, an Engineering Report will be prepared to demonstrate Title 22 compliance for the type of potable reuse. The Sanitation Districts would then receive WDRs/WRRs for LBWRP operation and the recycled water limits would be based on groundwater WQOs (for groundwater recharge projects), MCLs, and NLs. In addition, CEC monitoring will be required. CEC monitoring requirements and MTLs are periodically evaluated and modified by the State Water Board's Science Advisory Panel on CECs in Recycled Water. The most recent review was conducted by the Science Advisory Panel in 2018²⁴ and the Panel's recommendations were incorporated into the 2018 Recycled Water Policy. The next Science Advisory Panel may prescribe different CECs and different MTLs.

New WQOs or additional constituent requirements may be prescribed through Basin Plan Amendments.

²⁴ Monitoring Strategies for CECs in Recycled Water-Recommendations of a Science Advisory Panel, SCCWRP Technical Report 1032, April 2018.

As stated in Section 2.2, the State Water Board is in the process of developing MCLs for PFAS constituents. PFAS concentrations are currently the main concern for future potable reuse projects using LBWRP effluent. DDW has issued NLs and RLs for PFOA, PFOS, PFBS, and PFHxS and OEHHA has adopted PHGs for PFOS and PFOA. In addition, OEHHA has recommended a NL for PFHxA. DDW has asked OEHHA to consider grouping of PFAS for regulatory purposes. MCLs may result in future limitations for potable reuse projects and discharges to waters with the MUN and GWR beneficial uses. The current California drinking water advisory values are presented in Table 2.8.

Table 2.8 California PFAS Drinking Water Advisory Values

Compound	NL (Issued)	RL (Issued)	PHG
PFOA	5.1 ng/L	10.0 ng/L	0.007 ng/L (Cancer Health Protection)
PFOS	6.5 ng/L	40 ng/L	1.0 ng/L (Cancer Health Protection)
PFBS	500 ng/L	5,000 ng/L	--
PFHxS	3 ng/L	20 ng/L	--

The Safe Drinking Water Act (SDWA) authorizes the EPA to issue Health Advisories (HAs) for contaminants that are not subject to National Primary Drinking Water Regulations. HAs primarily serve as information to drinking water systems and officials responsible for protecting public health when emergency spills or other contamination situations occur. The EPA has issued interim updated drinking water HAs for PFOA and PFOS and final HAs for PFBS and HFPO-DA (GenX). In chemical and product manufacturing, GenX chemicals are considered a replacement for PFOA and PFBS is considered a replacement for PFOS.

On April 10, 2024, the EPA announced the final PFAS National Primary Drinking Water Regulations.²⁵ The regulations establish legally enforceable MCLs and non-enforceable health-based maximum contaminant level goals (MCLGs) for six PFAS compounds (PFOA, PFOS, perfluorononanoic acid [PFNA], PFBS, HFPO-DA [GenX], and PFHxS) as well as monitoring, public notification, and treatment requirements. The regulatory requirements are implemented under the authority of the SDWA and applied to operation of public drinking water systems nationwide. It is anticipated that DDW and the Regional Water Board will incorporate these requirements for operation of facilities that produce recycled water for potable purposes. The final MCLs and MCLGs are shown in Table 2.9. The individual MCLs are based on concentrations that can be reliably measured using EPA Method 1633. Compliance with the Hazard Index is determined by dividing the result of each PFAS compound by its assigned Health Based Factor and calculating the sum of the resulting fractions.

Table 2.9 EPA PFAS MCLs and MCLGs

Compound	Proposed MCLG	Proposed MCL
PFOA	0.0 ng/L	4.0 ng/L
PFOS	0.0 ng/L	4.0 ng/L
PFNA (health based factor = 10 ng/L)	1.0 (unitless) Hazard Index	
PFHxS (health based factor = 9 ng/L)		
PFBS (health based factor = 2,000 ng/L)		
HFPO-DA/GenX (health based factor = 10 ng/L)		

²⁵ <https://www.epa.gov/sdwa/and-polyfluoroalkyl-substances-pfas>

2.3.3 Compliance Evaluation

The specific constituents identified as LBWRP compliance concerns for potable reuse are discussed in the following sections.

2.3.3.1 Perfluorooctanoic Acid and Perfluorooctane Sulfonate

The possible future recycled water limits for PFOA and PFOS are based on the current NLs of 5.1 ng/L and 6.5 ng/L (respectively) and the EPA MCLs of 4.0 ng/L. LBWRP influent and effluent concentrations of PFOA are presented in Figure 2.8. LBWRP influent and effluent concentrations of PFOS are presented in Figure 2.9. Note that PFOS and PFOA data contained in this report are preliminary and were obtained using a method not approved by the EPA. While these data are valuable for research purposes, they are not intended for use related to the implementation of, or compliance with, any statutory or regulatory scheme, including but not limited to the federal Clean Water Act, the SDWA, and/or the Porter-Cologne Water Quality Control Act. There are currently no finalized EPA-approved analytical test methods for PFOS or PFOA in wastewater. The Sanitation Districts reserve the right to modify the data or contest any use of the data the agency deems to be inappropriate.

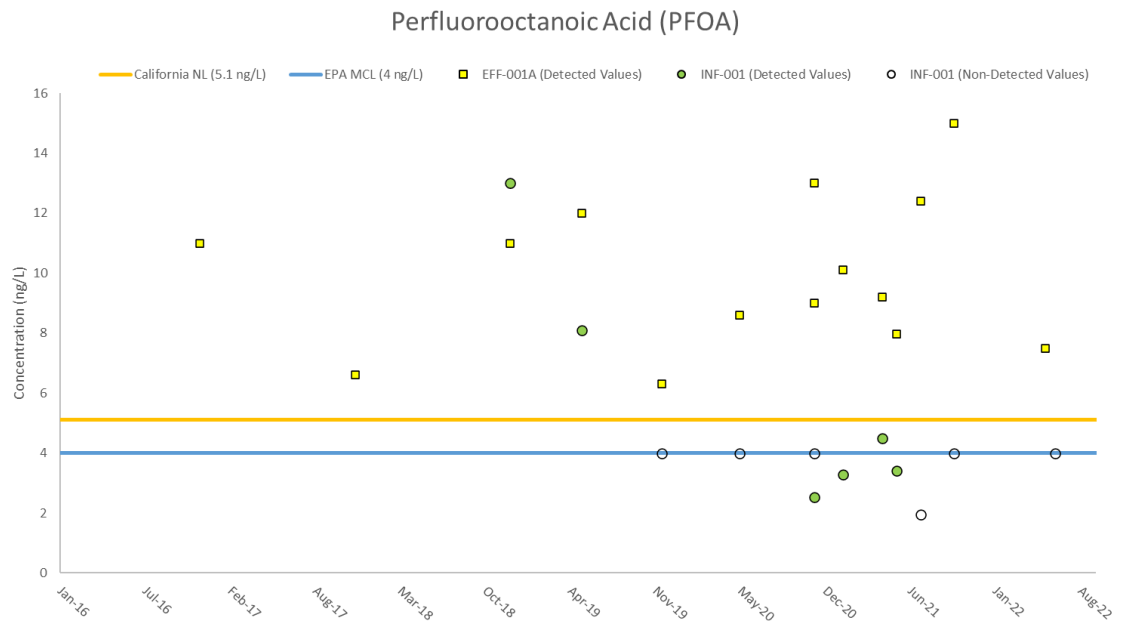


Figure 2.8 LBWRP Influent and Effluent Concentrations of PFOA 2016 to 2022

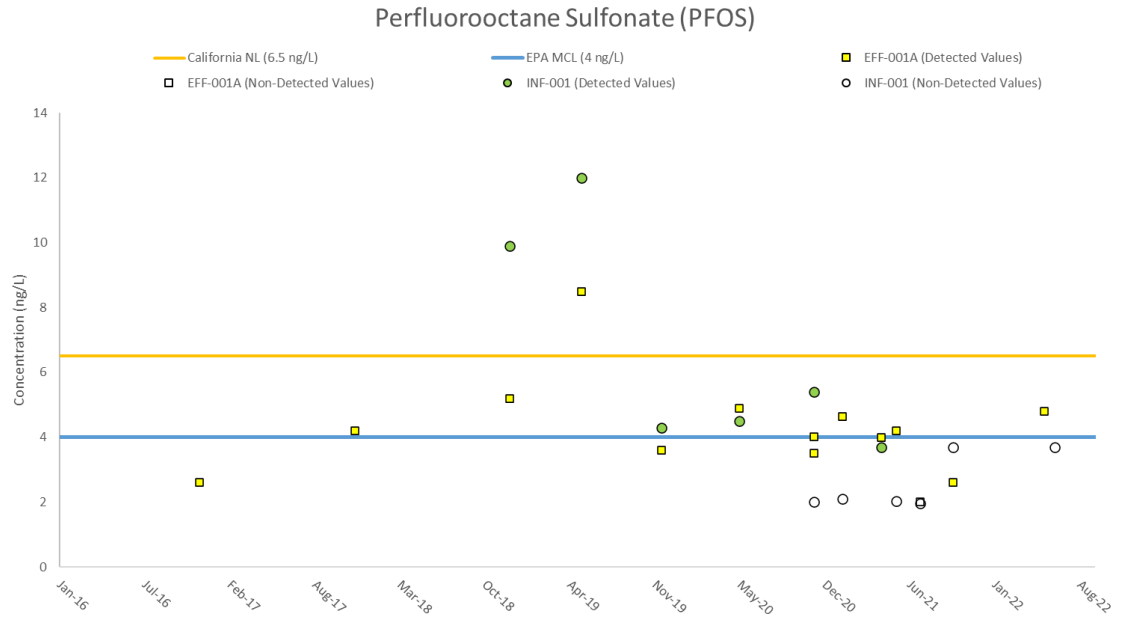


Figure 2.9 LBWRP Influent and Effluent Concentrations of PFOS 2016 to 2022

Effluent concentrations of PFOA regularly exceed the NL and the EPA MCL. As for PFOS, a number of effluent results exceed the MCL, but only a few results are above the NL. Influent concentrations of PFOA and PFOS have also periodically exceeded the NLs and MCLs. Treatment may be needed to meet the PFOA and potentially the PFOS limits. Since the effluent from LBWRP is sent to an advanced treatment facility before potable reuse, removal of PFAS is expected through that treatment and additional treatment may not be required for the LBWRP effluent.

2.3.3.2 Perfluorobutanesulfonic Acid

The possible future recycled water limit for PFBS is based on the current NL of 500 ng/L. LBWRP influent and effluent concentrations of PFBS are presented in Figure 2.10. Note that PFBS data contained in this report are preliminary and were obtained using a method not approved by the EPA. While these data are valuable for research purposes, they are not intended for use related to the implementation of, or compliance with, any statutory or regulatory scheme, including but not limited to the federal Clean Water Act, the SDWA, and/or the Porter-Cologne Water Quality Control Act. There are no finalized EPA-approved analytical test methods for PFBS in wastewater. The Sanitation Districts reserve the right to modify the data or contest any use of the data the agency deems to be inappropriate. Effluent concentrations are below the NL and compliance appears to be feasible.

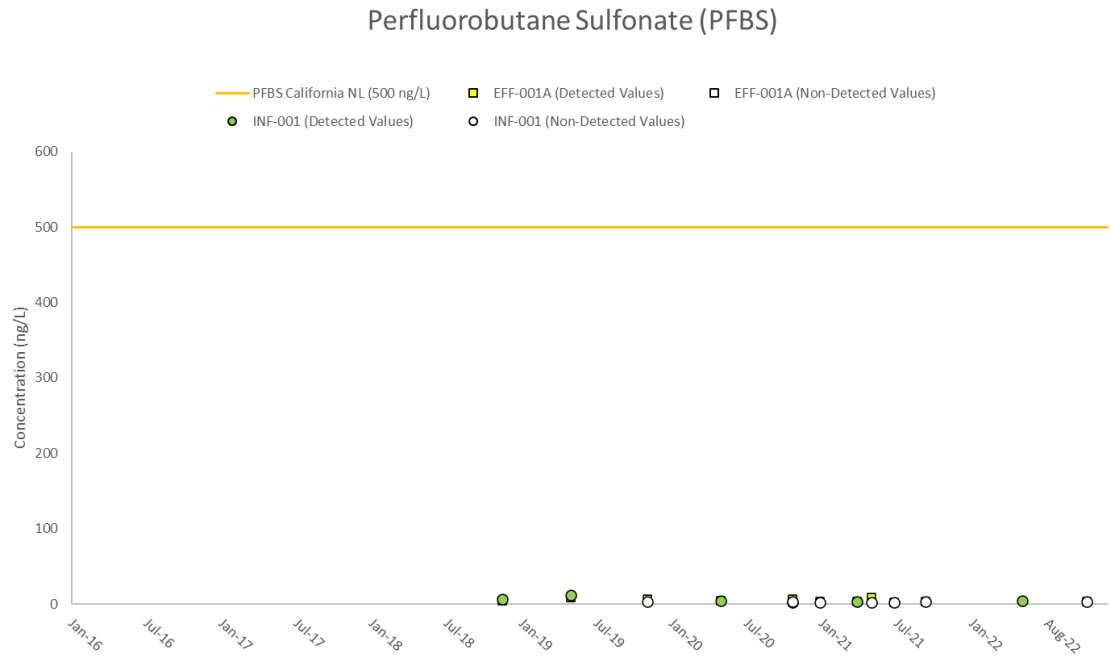


Figure 2.10 LBWRP Influent and Effluent Concentrations of PFBS 2016 to 2022

2.3.3.3 Perfluorohexanesulfonic Acid

The possible future recycled water limit for PFHxS is based on the current NL of 3 ng/L. Influent and effluent concentrations of PFHxS are presented in Figure 2.11. Note that PFHxS data contained in this report are preliminary and were obtained using a method not approved by the EPA. While these data are valuable for research purposes, they are not intended for use related to the implementation of, or compliance with, any statutory or regulatory scheme, including but not limited to the federal Clean Water Act, the SDWA, and/or the Porter-Cologne Water Quality Control Act. There are no finalized EPA-approved analytical test methods for PFHxS in wastewater. The Sanitation Districts reserve the right to modify the data or contest any use of the data the agency deems to be inappropriate. Several results have been measured above the NL.

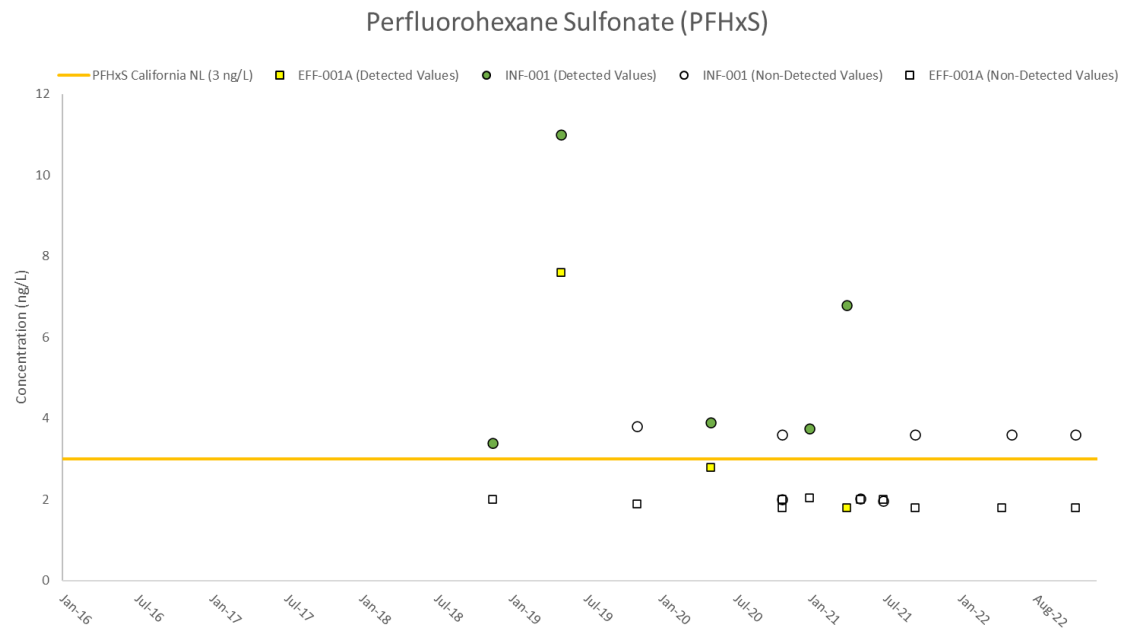


Figure 2.11 LBWRP Influent and Effluent Concentrations of PFHxS 2016 to 2022

2.3.3.4 Per- and Polyfluoroalkyl Substances Mixture

The possible future recycled water limit for PFAS mixtures is based on the EPA's Hazard Index MCL for PFBS, PFHxS, HFPO-DA, and PFNA. To determine compliance with the Hazard Index of 1.0, the concentration of each compound is compared to its health-based water concentration and the resulting ratio is summed. PFAS concentrations of influent and effluent at LBWRP were utilized for the Hazard Index calculations and the results are presented in Figure 2.12. Note that data contained in this report are preliminary and were obtained using a method not approved by the EPA. While these data are valuable for research purposes, they are not intended for use related to the implementation of, or compliance with, any statutory or regulatory scheme, including but not limited to the federal Clean Water Act, the SDWA, and/or the Porter-Cologne Water Quality Control Act. There are no finalized EPA-approved analytical test methods for PFBS, PFHxS, HFPO-DA, and PFNA in wastewater. The Sanitation Districts reserve the right to modify the data or contest any use of the data the agency deems to be inappropriate. All calculated values were below the proposed EPA Hazard Index. Concentrations that were less

than the practical quantitation level (PQL) were replaced with zero in the calculations according to the EPA regulations.²⁶

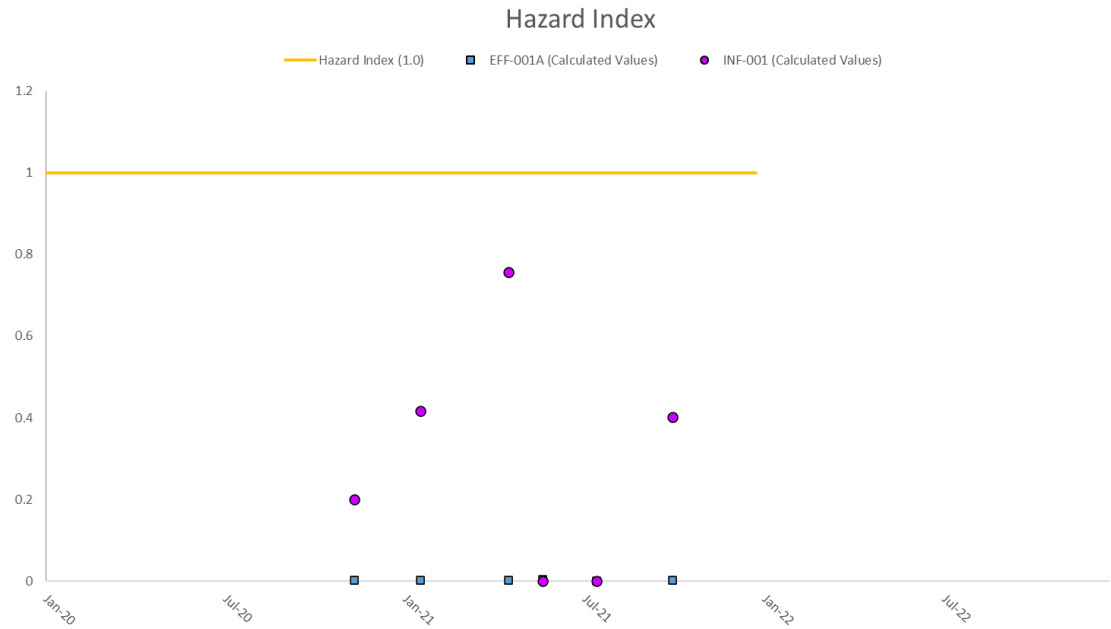


Figure 2.12 LBWRP Influent and Effluent Hazard Index Results 2020 to 2022

2.3.4 Proposed Strategy for Compliance

An overall strategy for compliance is provided in the summary section (Section 2.5).

2.4 Non-Potable Reuse

LBWRP produces disinfected tertiary recycled water²⁷ for NPR. Current uses include sanitary sewer cleaning, urban landscape irrigation, landscape impoundments, industrial processes, and toilet flushing. The City of Long Beach owns and operates the recycled water system which consists of approximately 34 miles of recycled water distribution pipeline, two pump stations, and three storage tanks at its Alamitos Hills tank farm. Currently, all recycled water production, distribution, and use (except sanitary sewer cleaning) is regulated by WRRs Order No. 87-47 (amended by Order No. 97-072). Sanitary sewer cleaning is regulated by the Statewide General Order WRRs. The Sanitation Districts is in the process of transitioning all non-potable recycled uses to the Statewide General Order WRRs.

2.4.1 Current Requirements

WRRs Order No. 87-47 includes recycled water limitations for pH, TDS, chloride, sulfate, and boron. The 1987 Order also specifies recycled water not contain concentrations of constituents in excess of California drinking water standards and not cause exceedance of Basin Plan nitrogen WQOs in groundwater. The treatment process requirements include oxidation, filtration, and disinfection to meet the most stringent coliform limitations associated with irrigation of parks,

²⁶ <https://www.federalregister.gov/documents/2024/04/26/2024-07773/pfas-national-primary-drinking-water-regulation>

²⁷As defined in California Code of Regulations Title 22 Section 60301.230.

schools, and playgrounds with unrestricted access. The current and future recycled water limitations are presented in Table 2.10.

Table 2.10 Recycled Water Limits for NPR (Current⁽¹⁾ and Future⁽²⁾)

Constituents	Recycled Water Limits		
	At All Times	Maximum Daily	Single Sample Maximum
TDS (Current)	--	--	1,000 mg/L
Chloride (Current)	--	--	250 mg/L
Sulfate (Current)	--	--	250 mg/L
Boron (Current)	--	--	1.5 mg/L
pH (Current)	6.0 to 9.0	--	--
Total Coliform (Current for most stringent Title 22 use)	2.2 MPN/100mL (median for last 7 days) ^(1,2)	23 MPN/100 mL (in any sample) ⁽¹⁾	240 MPN/100 mL ⁽²⁾
Total Coliform (Future for all Title 22 uses that require disinfected tertiary recycled water)			
Filter Loading Rate (Future)	5 gpm/ft ² (maximum)	--	--
Filter Effluent Turbidity (Current and Future)	2 NTU (within a 24-hour period) (Current)	5 NTU (no more than 5% of the time during a 24-hour period) (Current)	10 NTU (at any time) (Future)
Chlorine Disinfection (Future)	90 minute (modal contact time) 450 mg-min/L CT	--	--

Notes:

Abbreviations: gpm/ft² - gallons per minute per square foot; mg-min/L CT - milligrams per minutes per liter contact time.

(1) Current limitations based on WRRs Order No. 87-47.

(2) Future requirements based on the Statewide General Order WRRs, Title 22 Sections 60301.230 and 60301.320.

2.4.2 Future Requirements

The Sanitation Districts is currently in the process of transitioning all non-potable uses to the Statewide General Order WRRs. The Statewide General Order WRRs require recycled water production, distribution, and use in accordance with Title 22, Sections 6001-60355, consistent with any applicable Salt and Nutrient Management Plan (SNMP) adopted by the Regional Water Board, and in compliance with the Notice of Applicability (NOA) issued by the Regional Water Board. The LBWRP Title 22 Engineering Report has been revised to be consistent with current operations of the plant and regulations. The revised Engineering Report and Notice of Intent (NOI) for the Statewide General Order WRRs were submitted to DDW and the Regional Water Board. When the Engineering Report is accepted by DDW, the Regional Water Board will enroll the LBWRP recycled water program under the Statewide General Order WRRs and a Notice of Applicability (NOA) will be issued to the Sanitation Districts. Based on NOAs previously issued to the LCWRP (May 1, 2020) and WNWRP (July 7, 2017), recycled water limits will not be applied but a site-specific Monitoring and Reporting Program (MRP) that includes salts and nutrients will

be developed by the Regional Water Board. Recycled water production requirements will be included in the NOA because the General Order only addresses requirements for recycled water distribution and use. As a result, the transition to the Statewide General Order WRRs will trigger compliance with filter loading rates, filter effluent turbidity limits, minimum modal contact time and CT for disinfection, total coliform limits, and (possibly) demonstration of agronomic application rates for irrigation users. The additional disinfection, filter loading, and filter effluent turbidity requirements are shown in Table 2.10.

2.4.3 Compliance Evaluation

2.4.3.1 Total Dissolved Solids, Boron, Sulfate, Chloride

The maximum daily recycled water limit for TDS in WRRs Order No. 87-47 is 1,000 mg/L. Recent effluent data compared to this limit are shown in Figure 2.13. There were no TDS concentrations measured above the limit from 2016 to 2022. However, future potential limits for TDS, such as implementation of the recommended secondary MCL of 500 mg/L (12-month running average), could be of concern. However, in general non-potable reuse applications deal with TDS through implementation of SNMPs. There were no values measured above the recycled water limits for boron, sulfate, or chloride from 2016 to 2022.

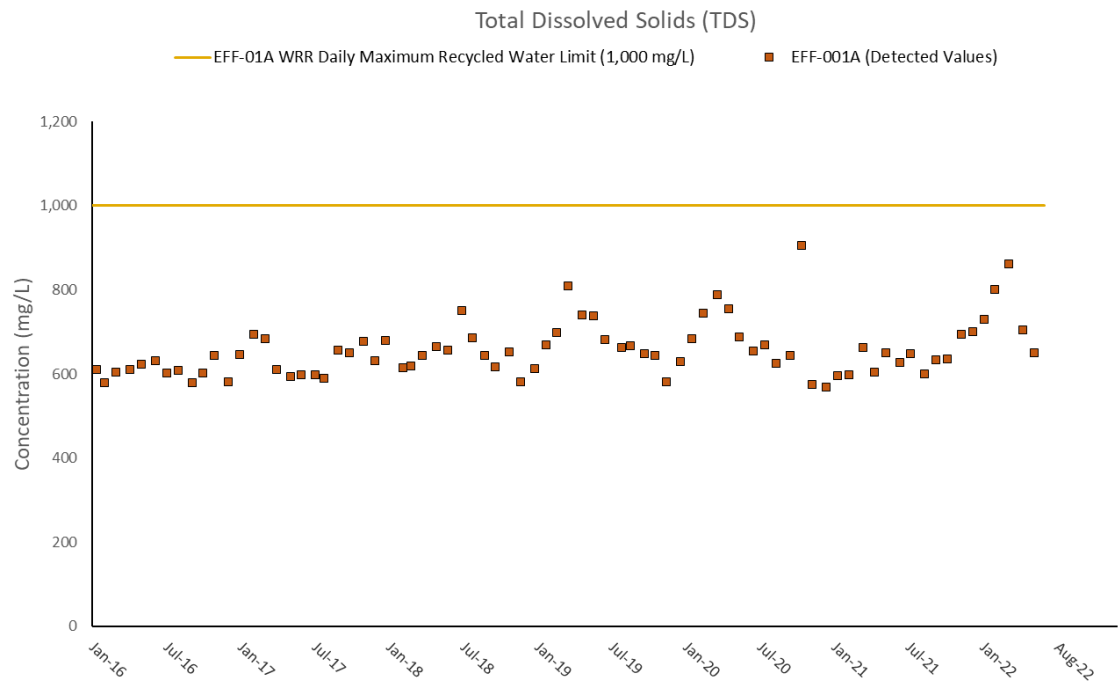


Figure 2.13 LBWRP Effluent TDS 2016 to 2022

2.4.3.2 Contact Time Disinfection

The Statewide General Order WRRs incorporate Title 22 standards for disinfected tertiary recycled water, including a minimum CT of 450 mg/L-min and minimum modal contact time of 90 minutes. The Sanitation Districts Operations and Reuse/Compliance staff are currently voluntarily tracking CT compliance using the new modal contact time equation (based on filter effluent flow). There have been some complaints from WRD about the high chlorine residual damaging their membranes. Results obtained between 2017 and 2022 are presented in

Figure 2.14. A 30-day average is shown as a trendline in the figure. There were several occasions when the CT was less than the 450 milligrams per liter per minutes (mg/L-min) requirement. The data shown in Figure 2.14 is calculated as Modal Contact Time x Chlorine Residual and not used for compliance purposes. Residual chlorine concentration was not provided from July 2019 through June 2021, therefore is not included in the figure.

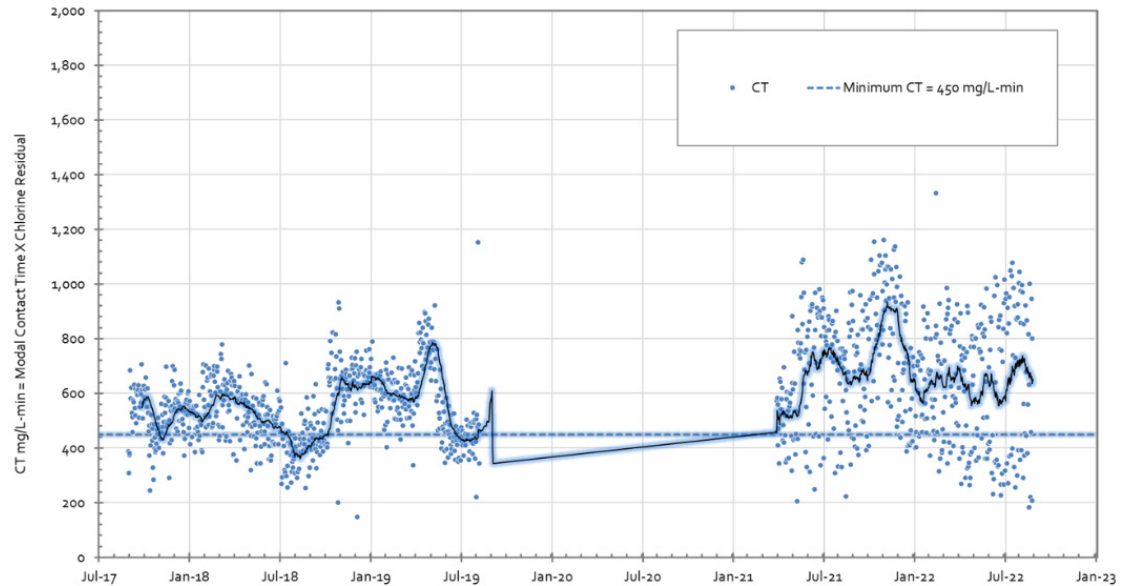


Figure 2.14 LBWRP CT Disinfection 2017 to 2022

2.4.4 Proposed Strategy for Compliance

An overall strategy for compliance is presented in the summary section (Section 2.5).

2.5 Summary

There is a range of strategies that may be employed for addressing regulatory concerns. Source control, scientific evidence, state and national regulatory advocacy, permit changes, and treatment optimization/upgrades are all options available for preparing and responding to regulatory compliance challenges. Participation in policy development and pilot studies may provide regulatory relief for specific constituents based on scientific evidence and development of information for improved decision-making by regulators. The possible response strategies are described within the following categories:

- **Source Control** – Monitoring collection systems, identifying and eliminating pollutant sources, establishing local limits, revising sewer use ordinances, working with manufacturers on product reformulation, and supporting/enacting product bans.
- **Regulatory Advocacy** – Development of policies and regulations to provide attainable regulatory solutions that are also protective of the environment and public health.
- **Permit Changes** – Conducting scientific studies to evaluate impacts of discharges on the environment, developing site specific objectives, conducting mixing zone/dilution

studies, modeling groundwater travel times and flow directions, and conducting antidegradation analyses to allow use of available assimilative capacity.

- **Treatment** – Optimization of existing facilities, modification of existing facilities, or additional treatment to reduce or remove constituents to comply with effluent limitations.

The compliance concerns and recommended strategies to address are presented in Table 2.11. The projected timeline to address the regulatory concerns is presented in Table 2.12.

Table 2.11 Summary of Regulatory Concerns and Potential Strategies

End Use	Regulatory Concerns	Potential Strategies to Address Concerns
Surface Water Discharge	Temperature	Source Control, Permit Changes, Treatment
	Chronic Toxicity	Source Control, Permit Changes, Treatment
	Ammonia	Permit Changes, Treatment
	Total Residual Chlorine	Treatment
	B&C&B Provisions	Regulatory Advocacy, Permit Changes
	CECs (Aquatic Ecosystems)	Regulatory Advocacy, Permit Changes, Treatment
Potable Reuse	PFAS	Source Control, Regulatory Advocacy, Treatment by WRD
NPR	TDS ((if a requirement is specified in the future)	Source Control, Regulatory Advocacy, Treatment
	CT Requirements	Treatment

Table 2.12 Projected Timelines for Addressing Regulatory Concerns

Operating Permit	Next 5 Years	Next 5-10 Years	10-20 Years Out
NPDES Permit (Discharge to Surface Waters)	<ul style="list-style-type: none"> • Toxicity Provisions. • Ammonia Limits. • Microplastics (Investigations). • Total Residual Chlorine. • Temperature (Develop Compliance Strategy). 	<ul style="list-style-type: none"> • Temperature (Implement Compliance Strategy). • Revised CEC Monitoring Requirements (Aquatic Ecosystems). • Microplastics (Monitoring Requirements). 	<ul style="list-style-type: none"> • B&C&B Provisions for Nutrients. • Microplastics (Effluent Limits).
Potable WDRs/WRRs (Discharge to Vander Lans WTF)	<ul style="list-style-type: none"> • PFAS Limits (met through treatment by WRD). 		
Non-Potable WRRs (Discharge for Irrigation, Industrial, Commercial Uses)	<ul style="list-style-type: none"> • Engineering Report Revisions. • CT Compliance. • Statewide General Order WRRs. 	<ul style="list-style-type: none"> • Agronomic Rate Evaluations (irrigation) • TDS (Revised Effluent Limits). 	



Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 3
LONG BEACH WATER RECLAMATION
PLANT EXISTING FACILITIES,
PERFORMANCE, AND
CAPACITY ASSESSMENT

FINAL | January 2025





**LOS ANGELES COUNTY
SANITATION DISTRICTS**
Converting Waste Into Resources

Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 3
LONG BEACH WATER RECLAMATION PLANT
EXISTING FACILITIES, PERFORMANCE, AND
CAPACITY ASSESSMENT

FINAL | January 2025

Digitally signed by Andre Gharagozian
Contact Info: Carollo Engineers, Inc.
Date: 2025.01.15 14:56:09 -0800



Contents

Technical Memorandum 3 - Long Beach Water Reclamation Plant Existing Facilities, Performance, and Capacity Assessment

3.1 Summary of Findings	3-1
3.2 Existing Facilities Description	3-4
3.3 Flow and Load Projections	3-4
3.3.1 Average Annual and Maximum Month Flow and Load Projections	3-4
3.3.2 Peak Dry Weather Flow Projections	3-7
3.3.3 Peak Wet Weather Flow Projections - Filters	3-7
3.3.4 Summary of Projected Flows	3-8
3.4 Hydraulic Evaluation	3-8
3.5 Performance Evaluation	3-10
3.5.1 Compliance With National Pollutant Discharge Elimination System Limits	3-10
3.5.2 Unit Process Evaluation	3-11
3.6 Capacity Evaluation	3-21
3.6.1 Approach to Capacity	3-21
3.6.2 Capacity Summary	3-22
3.7 Recent Wet Weather Events and Operational Strategies	3-27

Appendices

Appendix 3A	Process Description and Detailed Design Criteria
Appendix 3B	One-Line Diagram
Appendix 3C	Filter Design Criteria
Appendix 3D	Hydraulic Capacity Summary
Appendix 3E	Performance Assessment

Tables

Table 3.1	Unit Process Capacity Ratings	3-3
Table 3.2	Flow and Load Projections Summary From TM 1	3-4
Table 3.3	Peaking Factors From TM 1	3-7
Table 3.4	PDWF Projections for Disinfection Facilities	3-7
Table 3.5	PWWF Projections for Effluent Filters	3-8

Table 3.6	Summary of Projected Flows	3-8
Table 3.7	Unit Process Hydraulic Evaluation	3-9
Table 3.8	Unit Process Performance and Capacity Criteria	3-12
Table 3.9	PE Concentrations	3-16
Table 3.10	Laboratory Data for MLSS Concentration	3-17
Table 3.11	Reliable AADF Analysis Criteria	3-22
Table 3.12	Unit Process Capacity Ratings	3-23
Table 3.13	Peaking Factors for Wet Weather Analysis	3-28

Figures

Figure 3.1	Existing Site Layout	3-5
Figure 3.2	Process Flow Diagram	3-6
Figure 3.3	PSTs Percent TSS, COD, and BOD ₅ Removal	3-14
Figure 3.4	PE Water Quality	3-15
Figure 3.5	MLSS (Pass 4, Units 1 and 2)	3-18
Figure 3.6	Total SRT and aSRT	3-19
Figure 3.7	SVI (Units 1 and 2)	3-20
Figure 3.8	Unit Process Capacity	3-23
Figure 3.9	Secondary Treatment System Capacity With All Aeration Tanks and Clarifiers in Service	3-25
Figure 3.10	Secondary Treatment System Reliable Capacity With 1 of 2 Aeration Tanks in Service	3-26
Figure 3.11	Secondary Treatment System Reliable Capacity With 12 of 13 FSTs in Service	3-26
Figure 3.12	Projected Peak Day Process Air Demands	3-27
Figure 3.13	Recurrence Frequency for February 2024 Storm at Long Beach	3-28

Abbreviations

AADF	average annual daily flow
aSRT	aerobic solids retention time
BOD ₅	five-day biochemical oxygen demand test
COD	chemical oxygen demand
CCT	chlorine contact tank
CT	contact time
FST	final sedimentation tank
gpd/sf	gallons per day per square foot
gpm/sf	gallons per minute per square foot
IPS	influent pumping station
klb/d	thousand pounds per day
LBWRP	Long Beach Water Reclamation Plant
mgd	million gallons per day
mg/L	milligrams per liter
mg-min/L	milligrams-minute per liter
min	minute
mL/g	milliliters per gram
MLR	mixed liquor recirculation
MLSS	mixed liquor suspended solids
NdN	nitrification/denitrification
NH ₃	ammonia
NO ₂	nitrite
NO ₃	nitrate
NPDES	National Pollutant Discharge Elimination System
NTU	nephelometric turbidity unit
O&M	operations and maintenance
OOS	out of service
PAC	process air consumption
PDWF	peak dry weather flow
PE	primary effluent
ppd	pounds per day
ppd/sf	pounds per day per square foot
PST	primary sedimentation tank
PWWF	peak wet weather flow
RAS	return activated sludge

Sanitation Districts	Los Angeles County Sanitation Districts
scfm	standard cubic feet per minute
SJC	San Jose Creek
SLR	solids loading rate
SOR	surface overflow rate
SRT	solids retention time
SVI	sludge volume index
TM	technical memorandum
TSS	total suspended solids
WRP	water reclamation plant
WRR	water recycling requirement
WSEL	water surface elevation
WW	wastewater

Technical Memorandum 3

LONG BEACH WATER RECLAMATION PLANT EXISTING FACILITIES, PERFORMANCE, AND CAPACITY ASSESSMENT

3.1 Summary of Findings

This technical memorandum (TM) provides a description of the existing facilities, an evaluation of the process performance, and a process and hydraulic capacity assessment of the Los Angeles County Sanitation Districts' (Sanitation Districts) Long Beach Water Reclamation Plant (LBWRP).

Daily influent, effluent, and operations data were reviewed from January 2015 through December 2016 and January 2020 through December 2022 for the performance assessment. The data from January 2017 through December 2019 were excluded due to the construction during this period. The key findings of the performance assessment are summarized below:

- With very few exceptions, the LBWRP was in compliance with all of the conventional National Pollutant Discharge Elimination System (NPDES) discharge requirements (5-day biochemical oxygen demand test [BOD₅], total suspended solids [TSS], turbidity, ammonia [NH₃], nitrite [NO₂], nitrate [NO₃] + NO₂, pH, and oil and grease). Although the plant is generally in compliance with NPDES requirements, some unit processes are operating under stressed conditions, at or near their capacity limits, and at higher loading rates than their original design.
- Rags and grit accumulate in and around mechanical equipment and the primary feed channels and sedimentation tanks because there is no preliminary treatment (i.e., screening and grit removal). The accumulation of debris and grit increases maintenance efforts and labor for these facilities. The Sanitation Districts intends to install chopper recirculation pumps to help reduce this accumulation.
- Primary effluent (PE) concentrations for chemical oxygen demand (COD), BOD₅, TSS, and NH₃ have increased by 50 - 80 percent from the original design basis, which is likely due to the trend of increasing wastewater (WW) strength due to drought and water conservation in the Western United States. Primary removal rates have a significant impact on the estimated capacity of the secondary process. The wastewater discharged in the sewer from LBWRP recirculates back into the LBWRP during high flows resulting in high primary effluent concentrations.

- Key findings and challenges related to the step feed nitrifying and denitrifying secondary performance are described below:
 - Although the secondary process at LBWRP generally produces effluent that meets effluent nitrogen targets ($\text{NO}_3 + \text{NO}_2 < 8$ milligrams per liter [mg/L]), there are a few instances between 2020 and 2022 when this target was exceeded.
 - The PE distribution between the step feed passes in the aeration tanks is not consistent, which is largely due to physical limitations with flow splitting and insufficient automation and controls. Improving the consistency and accuracy of the flow split and upgrading the aeration controls along with on-line NH_3 and NO_3 monitoring may improve performance.
 - An additional challenge relating to the flow split is that most of the filter backwash is returned to Stage 1 and therefore, to equalize loads between the two stages, the flow split between Stage 1 and Stage 2 is 60/40, respectively. The Sanitation Districts has investigated the feasibility of improving the PE flow split and determined that the potential for performance improvements is minimal.
 - Although NH_3 removal has been good, there is a noticeable increase in secondary effluent NH_3 concentration for 2020 - 2022. Additionally, tertiary NH_3 concentrations did exceed the monthly average limit in 2022. However, the Sanitation Districts mentioned that additional samples were taken for the same period and monthly average compliance limits were met.
 - The solids loading rate (SLR) for the final sedimentation tanks (FSTs) is high. The average peak SLR for the review period was 55 pounds per day per square foot (ppd/sf), which is higher than the typical peak SLR of 35 to 40 ppd/sf. The FSTs perform well most of the time with the addition of the polymer, but are operating near their capacity limits and are sometimes overloaded during wet weather events.
 - Secondary effluent TSS concentration is less than 5 mg/L for both the stages, however, TSS concentration for Stage 1 is consistently higher than Stage 2. This imbalance could be due to differences in flow distribution because Stage 1 receives 60 percent of the flow and Stage 2 receives 40 percent flow.
- Effluent filter performance has been good with respect to TSS removal and meeting effluent turbidity limits for the review period. The tertiary effluent turbidity is well within the 2 nephelometric turbidity unit (NTU) permit limit, typically averaging about 0.7 NTU even during periods of high secondary turbidity.
- The disinfection process has performed well with no compliance issues meeting effluent coliform limits or contact time (CT) requirements. However, there have been eight (8) exceedances of the chlorine residual limit which is believed to be due to the configuration of the reuse system. During the site visit it was noted that the reuse water draws its suction from chlorine contact tanks (CCTs), which causes fluctuation in the CCT water level, hydraulics, CT, and chlorine dose. This configuration makes the process control difficult. Maintaining 450 CT (contact time x chlorine residual) with varying water levels will be difficult when the new water recycling requirements (WRRs) are issued.

Recommended capacity rating criteria were developed for each unit process as part of the performance assessment. The recommended capacity criteria were used along with steady state process modeling and state point analysis to develop average annual daily flow (AADF), peak wet weather flow (PWWF), and reliable AADF capacity for each unit process. Peak dry weather flow (PDWF) capacity was developed only for CCTs based on Title 22 Requirements of

450 milligrams-minute per liter (mg-min/L) CT. Reliable AADF capacity reflects the capacity when process tanks are taken out of service (OOS) for planned maintenance.

LBWRP has an AADF and PWWF permitted capacity of 25 and 60 million gallons per day (mgd), respectively. Capacity limitations were identified when unit processes had less capacity than the 2050 flow and load projections, which establishes the end of the Master Facilities Plan planning period. Capacity, the 2050 projections, and the capacity limitations at LBWRP are summarized in Table 3.1.

Table 3.1 Unit Process Capacity Ratings

Process	Controlling Condition	Capacity, mgd	Projected 2050 Flow, mgd	Capacity Deficit, mgd
Influent Pumping Station	PWWF	84 ⁽¹⁾ - 114 ⁽²⁾	38.2	0
PSTs	PWWF	60	38.2	0
	AADF	25	15.9	0
	Reliable AADF	19 ⁽³⁾	--	--
Aeration Tanks	PWWF	35.5 ⁽⁴⁾	38.2	2.6
	AADF	14.8 ⁽⁴⁾	15.9	1.1
	Reliable AADF	9.4 ⁽⁵⁾	--	--
FSTs	PWWF	35.5 ⁽⁴⁾	38.2	2.6
	AADF	14.8 ⁽⁴⁾	15.9	1.1
	Reliable AADF	14.2 ⁽⁶⁾	--	--
Aeration Blowers	AADF	27.8	15.9	0
	Reliable AADF	27.8	--	--
Filters	PWWF	33.2	33.5 ⁽⁸⁾	0.3
	AADF	25.1	15.9	0
	Reliable AADF	20.1 ⁽⁷⁾	--	--
Effluent Filter Pump Station	PWWF	34.1 ⁽¹⁾ - 46.5 ⁽²⁾	38.2	0 ⁽²⁾ - 4.1 ⁽¹⁾
Disinfection	PDWF	31.0 ⁽⁹⁾	27.5 ⁽¹⁰⁾	See Section 3.6.2.2 ⁽¹¹⁾
	AADF	17.9 ⁽¹⁰⁾	15.9	0

Notes:

Abbreviations: PST - primary sedimentation tank.

- (1) Based on firm capacity (one pump OOS).
- (2) Based on total capacity (all pumps in service).
- (3) Based on one PST OOS.
- (4) Based on maximum month load conditions, which correspond to PE concentration of: COD = 580, BOD₅ = 296, TSS = 186, and NH₃ = 47 mg/L. Other assumptions include an aerobic solids retention time (aSRT) of five days and resulting mixed liquor suspended solids (MLSS) concentration of 2,800 mg/L, sludge volume index (SVI) of 150 milliliters per gram (mL/g), and a PWWF/AADF peaking factor of 2.4.
- (5) Based on one aeration tank unit OOS.
- (6) Based on one FST OOS.
- (7) Based on two filters OOS.
- (8) Based on the Sanitation Districts' approach for projecting PWWF for effluent filters described in Appendix 3C.
- (9) Based on CCTs meeting Title 22 Requirements of 450 mg-min/L CT.
- (10) Calculated using a peaking factor (PDWF/AADF) of 1.73.
- (11) While the CCT volume is sufficient for future flows, the operation of the reuse system impacts water levels and prevents full use of the CCT volume. Modifications to recycled water permit will require full compliance with 450 CT in the future. Therefore, either additional volume must be added, or the reuse operations will need to be modified for compliance with future 450 CT requirements for PDWF.

3.2 Existing Facilities Description

The LBWRP is located approximately 1 mile north of the Interstate 405/Interstate 605 interchange near the eastern border of Los Angeles County. The facility address is 7400 East Willow Street, Long Beach, CA 90815. The AADF and PWWF design ratings are 25 and 60 mgd, respectively. The existing facilities description was primarily generated from the following sources:

- LBWRP Operations Manual (2015).
- Site Visit on 1/24/23 and discussions with the Sanitation Districts' staff.
- The Sanitation Districts' Design Memoranda.

The existing site layout is shown on Figure 3.1. The process flow diagram is shown on Figure 3.2 and consists of primary sedimentation, aeration, final sedimentation, filtration, and disinfection. A detailed description of the existing facilities and design criteria for each unit process are provided in Appendix 3A. One-line diagrams for the power distribution system at LBWRP are provided in Appendix 3B.

3.3 Flow and Load Projections

3.3.1 Average Annual and Maximum Month Flow and Load Projections

The Sanitation Districts prepared TM 1, which projects future influent flows and loads to the LBWRP. A summary of the projected future flows and loads is shown in Table 3.2 for average annual and average daily maximum month. Average day WW flow is projected to increase by 0.5 mgd by 2050 from the current five-year average assuming there is no additional infiltration and inflow flow. The current five-year average includes flows from 2015 - 2016 and 2020 - 2022. Average flows from 2017 - 2019 were excluded from calculations due to several ongoing construction projects during the period. During construction periods, the Sanitation Districts reduced influent flows by diverting flow to the A.K. Warren Water Resource Facility (Warren Facility). Therefore, reported flows and loads during these construction periods are not representative of actual conditions in the service area.

Table 3.2 Flow and Load Projections Summary From TM 1

	Current Five Year Average (2015-2016, 2020-2022)	2030	2040	2050
Average Annual				
Flow, mgd	15.4	15.6	15.7	15.9
BOD ₅ , ppd	44,580	47,493	47,994	48,485
COD, ppd	91,255	97,193	98,213	99,213
TSS, ppd	44,236	47,104	47,596	48,079
Maximum Month				
Flow, mgd	17.0	17.2	17.4	17.5
BOD ₅ , ppd	54,208	57,750	58,359	58,956
COD, ppd	119,095	126,844	128,176	129,480
TSS, ppd	57,774	61,518	62,161	62,792

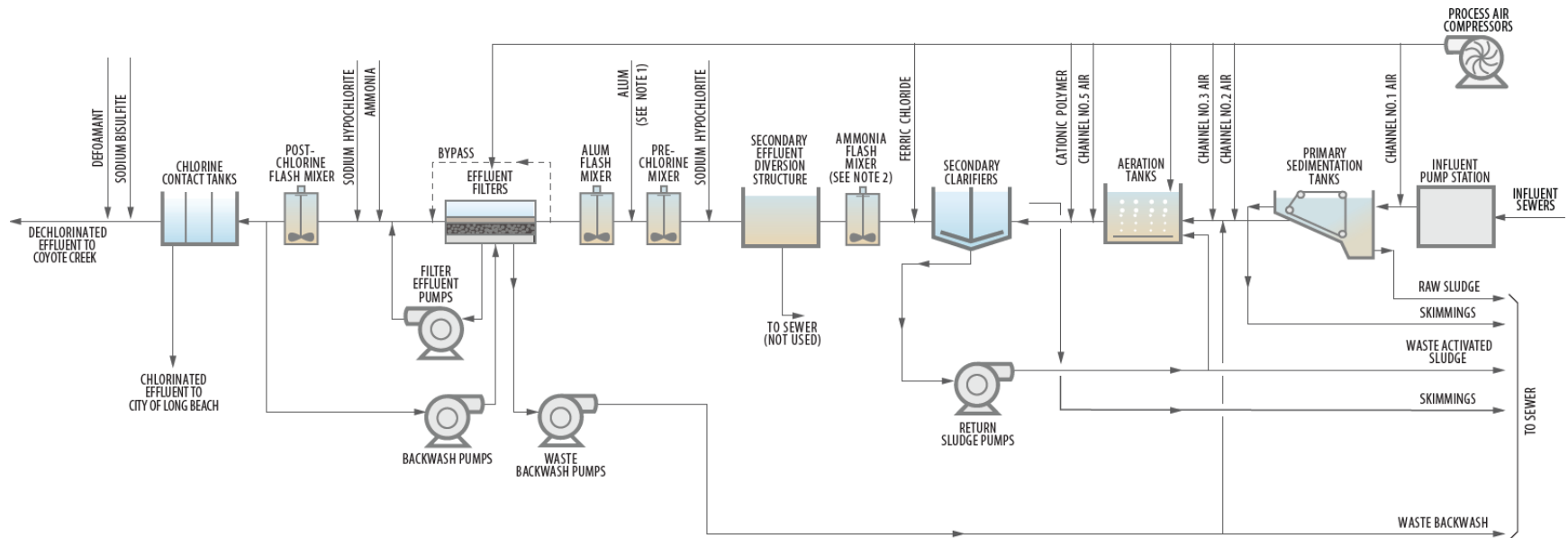
Notes:
Abbreviations: ppd - pounds per day.



- LEGEND**
1. Potable and Non-Potable Water Station
 2. Stormwater Pump Station No. 2
 3. Lab/Control Building
 4. Influent Pump Station
 5. Channel No. 1
 6. Primary Sedimentation Tanks
 7. Channel No. 2
 8. Standby Emergency Generator
 9. Process Air Compressor Building
 10. SC Edison Power Substation
 11. Channel No. 3
 12. Aeration Tanks
 13. Channel No. 5
 14. Secondary Clarifiers
 15. Gallery No. 6
 16. Stage I Return Sludge Pump Station
 17. Channel and Gallery No. 7
 18. Ferric Chloride
 19. Effluent Division Structure
 20. Stage II Return Sludge Pump Station
 21. Filter Influent Pumps
 22. Filter Backwash
 23. Effluent Filters
 24. Filter Effluent Pumps
 25. Waste Backwash Recovery Pump
 26. Secondary Polymer Station
 27. Ammonia Station
 28. Sodium Bisulfite Facility
 29. Sodium Hypochlorite Facility
 30. Chlorine Contact Tanks
 31. Secondary Sodium Bisulfite Station
 32. Defoamant
 33. Washwater Pumps
 34. Fire Pump
 35. Effluent Forebay
 36. City of Long Beach Reuse Pump Station
 37. Stormwater Pump Station No. 2

LONG BEACH
 WATER RECLAMATION PLANT
 OPERATIONS MANUAL
SITE PLAN

Figure 3.1 Existing Site Layout



NOTES:

1. Alum is not currently injected into Filter Influent.
2. Ammonia Mixer in Channel No.7 is currently not in service.

Figure 3.2 Process Flow Diagram

A summary of peaking factors for the current five-year average period is shown in Table 3.3. The calculated five-year average peaking factor is 1.9, which is lower than the original design peaking factor of 2.4, as shown in Table 3.3. Though the average peaking factor is 1.9, the original design peaking factor of 2.4 was decided to be used for flow projections and capacity analysis after review of TM 1 and discussion with the Sanitation Districts’ team.

Table 3.3 Peaking Factors From TM 1

Year	PWWF, mgd	AADF, mgd	Peaking Factor
2015	29.1	14.7	2.0
2016	26.2	16.4	1.6
2020	28.7	15.2	1.9
2021	33.7	15.6	2.2
2022	24.3	15.1	1.6
Average	28.4	15.4	1.9
Original Design Criteria	60.0⁽¹⁾	25.0⁽¹⁾	2.4⁽¹⁾
2050 Projections	38.2	15.9	2.4

Notes:

(1) From LBWRP Operations Manual (2015).

3.3.2 Peak Dry Weather Flow Projections

The 2050 design PDWF is calculated using a peaking factor of 1.73, based on the current five-year average period. For each individual year from 2015 - 2016 and 2020 - 2022, the PDWF is calculated as the peak hourly flow for the dry weather period (July - September). PDWF is used for evaluating disinfection facility capacity. As shown in Table 3.4, the 2050 projected PDWF is 27.5 mgd (15.9 x 1.73 mgd).

Table 3.4 PDWF Projections for Disinfection Facilities

Year	PDWF ⁽¹⁾ , mgd	Influent AADF, mgd	Peaking Factor
2015	25.1	14.7	1.71
2016	25.7	16.4	1.57
2020	29.4	15.2	1.93
2021	28.7	15.6	1.84
2022	23.3	14.5	1.61
Average	26.4	15.4	1.73
2050 PDWF Projections	27.5	15.9	--

Notes:

(1) Calculated as peak hourly flow for dry weather period (July through September).

3.3.3 Peak Wet Weather Flow Projections - Filters

The PWWF for filters is calculated using an approach established by the Sanitation Districts to develop filter design criteria aimed to size filters to permit reuse during most, but not all, wet weather events over a 30-year planning horizon. The 2050 projected PWWF for filters is summarized in Table 3.5. The projected flow is 33.5 mgd with a peaking factor of 2.1. Filter capacity assessment is based on a typical wet weather event that occurs annually (i.e., one-year

recurrence frequency) instead of a two- to five-year storm event. The same approach was used for evaluating capacity of filters at San Jose Creek (SJC) and is summarized in Appendix 3C.

Table 3.5 PWWF Projections for Effluent Filters

Year	Filter Effluent PWWF, mgd	Influent AADF, mgd	Peaking Factor
2015	34.9	14.7	2.4
2016	30.6	16.4	1.9
2020	32.8	15.2	2.2
2021	35.0	15.6	2.2
2022	25.3	14.5	1.7
Average	31.7	15.4	2.1
50th Percentile	32.8	--	--
Incremental Flow to Buildout	0.50	0.50	--
2050 Projected Flows	33.3	15.9	--
Rounded-Up Projection	33.5	15.9	2.1

3.3.4 Summary of Projected Flows

The 2050 projected flow and peaking factors as compared to original design basis were summarized in Table 3.6. The 2050 design AADF projection for LBWRP is 15.9 mgd. The PWWF projection for secondary process is calculated using the 2.4 peaking factor as established in Section 3.3.1. The PDWF projection for disinfection facilities is calculated using a peaking factor of 1.73 as discussed in Section 3.3.2 and the PWWF projection for filters is 33.5 mgd based on the calculation described in Section 3.3.3.

Table 3.6 Summary of Projected Flows

Flow Condition	Projected Flow, mgd	Peaking Factor (2015 - 2016, 2020 - 2022)	Original Flow Design Basis
AADF	15.9	--	25.0
PDWF	27.5	1.73	34.0
PWWF - Secondary	38.2	2.40	60.0
PWWF - Filters	33.5	2.10	--

3.4 Hydraulic Evaluation

Spreadsheet hydraulic models developed by the Sanitation Districts were used to assess the hydraulic capacity at LBWRP. The model has physical dimensions of the processes and hydraulic control structures, pipe sizes, weir elevations, top of wall elevations and calculations to determine hydraulic losses through the processes. The model calculates the water surface elevation (WSEL) for each process by adding the calculated hydraulic loss between processes to the WSEL in the downstream process (or downstream control point such as a weir, trough, or wet well). The main purpose of the hydraulic assessment is to identify hydraulic limitations. The model was reviewed by the planning team, although the accuracy was not verified or calibrated to historical data. In addition, no modifications were made to the formulas or physical

dimensions in the models. It is recommended that quality checks be performed prior to using it for final design if the results have not yet been verified with field measurements.

The models were simulated with the following criteria and assumptions:

- 2050 PWWF Projections.
- Return activated sludge (RAS) is set at the lower 90 percent influent flow or firm pump capacity (total pump station capacity with largest pump OOS) of RAS pump station.
- Mixed liquor recirculation (MLR) is set at 0 percent as there is no MLR at LBWRP.
- All process units are in service.
- Flow is equally split between process units.
- All gates are 100 percent open, which is a reasonable assumption to accommodate peak wet weather events.
- If the water elevation is above the top of wall elevation of the structure, the top of wall elevation is used as the water elevation to evaluate the upstream process. The maximum water elevation in the structure will be at the top of wall elevation.

Hydraulic limitations were noted if any of the following conditions were observed:

- **Weir Submergence:** Weir submergence occurs when the WSEL downstream of the weir is higher than the weir crest elevation. The unit process may short circuit when effluent weirs are submerged. Accuracy of the flow split will be reduced when flow splitting weirs are submerged, which may degrade process performance.
- **Six Inches of Freeboard:** Freeboard is the distance between the WSEL and the top of concrete for a water holding structure. Operating with 6 inches (or more) of freeboard is desirable during peak flow conditions to minimize the risk of overtopping the structure. This criterion was also built into the Sanitation Districts’ hydraulic models.
- **No Freeboard (Overflow):** Although not a desirable operating scenario, the no freeboard condition was considered to identify areas of overflow risk. Flow rates corresponding to minimum freeboard were estimated for the processes that show tank overflow.

Hydraulic evaluation of each process for LBWRP at 2050 PWWF projection of 38.16 mgd and an RAS firm capacity of 34.34 mgd is summarized in Table 3.7. The LBWRP hydraulic profile is in Appendix 3D.

Table 3.7 Unit Process Hydraulic Evaluation

Location		Elevation at Water Surface (ft)	Elevation at Top of Wall (ft)	Meet 6-Inch Freeboard Requirement
Outlet Structure	Upstream of Weir	26.00	28.50	YES
CCT	CCT	26.00	28.50	YES
Effluent Distribution Channel	Effluent Distribution Channel	24.17	30.00	YES
Filter Effluent Channel	Filter Effluent Channel	24.32	30.00	YES
Filter Effluent Pump Station	Filter Effluent Pump Station	11.00	16.50	YES

Location		Elevation at Water Surface (ft)	Elevation at Top of Wall (ft)	Meet 6-Inch Freeboard Requirement
Filter	Exit from Filter	11.89	30.00	YES
	Filter	19.39		
Filter Influent Channel	Downstream of Outlet Weir	23.54	30.00	YES
	Upstream of Outlet Weir	27.08		
Effluent Diversion Structure	Effluent Diversion Structure	27.22	30.00	YES
Channel No. 7		27.36	30.00	YES
FST	Launder	26.95	30.00	YES
	FST	28.51		
Channel No. 5	Channel No. 5	28.66	31.00	YES
Aeration Tanks	Exit	23.00	31.00	YES
	Step Feed Channel	23.08		
	Entrance	23.65		
Channel No. 3	Channel No. 3	23.65	31.00	YES
Channel No. 2	Channel No. 2	23.99	31.00	YES
PSTs	Launder	27.91	31.00	YES
	PST	30.61		
Channel No. 1	Channel No. 1	30.70	31.00	NO

There are no hydraulic limitations observed from PSTs through a CCT, however, the freeboard in Channel No. 1 upstream of the PST is less than 6 inches. The difference between the PST effluent weir elevation and top of the wall elevation of the PST is 7 inches resulting in less than 6 inches of freeboard in the PST. There are some flow splitting challenges that exist with PE distribution into the aeration tank and the approximate 1-foot elevation difference between primary and FST effluent weirs. However, Sanitation Districts' staff mentioned that no hydraulic issues were observed during the 2023 storm events in a 5/10/2023 meeting.

3.5 Performance Evaluation

Historical treatment performance of the LBWRP is summarized in this section. Daily operating data from January 2015 through December 2016 and January 2020 through December 2022 were reviewed for the evaluation. In addition to reviewing historical data, discussions were held with operations staff, and the Sanitation Districts' Design Memoranda were reviewed to better understand the water reclamation plant (WRP) performance and limitations.

3.5.1 Compliance With National Pollutant Discharge Elimination System Limits

The overall treatment performance of the LBWRP with respect to meeting NPDES discharge limits was evaluated in detail in TM 2 - LBWRP Regulatory Review. Effluent concentrations of metals and priority pollutants are largely driven by the influent concentrations (and not treatment performance). The effluent concentration for conventional pollutants, turbidity, and nitrogen species is a reasonable gauge for treatment performance. The WRP's compliance

history for those parameters during the review period is summarized below. Additional information and time series plots for these parameters are in TM 2.

- One hundred percent compliance for BOD₅ and TSS.
- One exceedance in 2022 of NH₃ monthly average limit of 4.1 mg/L.
- One hundred percent compliance for NO₂.
- A few exceedances in spring of 2020 and 2022 for NO₃ + NO₂ monthly limit of 8 mg/L.
- One hundred percent compliance for effluent turbidity.
- One hundred percent compliance for total effluent coliforms.
- Eight exceedances for total residual chlorine monthly limit of 0.1 mg/L.

The exceedances of the chlorine residual requirement are believed to be due to the configuration and operation of the recycled water pumps. The suction for the recycled water pumps is located in the CCT. The level in the basin drops during operation, which affects CT, hydraulics, and chlorine dose control. It is recommended that the Sanitation Districts consider modifications to simplify the operation.

Time series plots for effluent NH₃, NO₃, and NO₃ + NO₂ concentrations are shown on Figure 2.3 through Figure 2.5 in TM 2. Based on these figures, there are a few exceedances for the effluent NH₃ and NO₃ + NO₂ concentrations in the springs of 2020 and 2022. The cause for the exceedances is believed to be due to construction activities. It is recommended that the Sanitation Districts consider modifications to improve process control to minimize the risk or frequency of future exceedances.

3.5.2 Unit Process Evaluation

An understanding of the WRP's unit process performance is critical to determining the treatment capacity. Based on historical loading and performance, recommended criteria for assessing capacity were developed for each major treatment process to serve as the basis for the process capacity evaluation. The historical load and performance of each unit process was compared to the original design criteria and typical anticipated performance. The performance of each unit process provides a benchmark for the planning of new facilities and assessing capacity. In some cases, historical performance confirms that original design criteria are appropriate for assessing unit process capacity. In others, above or below average performance warrants adjusting original design criteria for assessing capacity. Recommended design criteria are identified for use in the capacity assessment for each unit process.

The results of the performance evaluation for LBWRP are summarized in Table 3.8. The rest of this section provides a focused discussion of the operating conditions for the secondary and disinfection processes as there are many factors which impact performance and capacity. Other unit processes are not discussed in this section as their performance has been satisfactory and no capacity limitations are expected. Time series plots for key operating criteria and for unit processes not discussed in this section (i.e., PSTs, effluent filters, and CCTs) are provided for reference in Appendix 3E.

Table 3.8 Unit Process Performance and Capacity Criteria

Process	Design Parameter	Units	Original Design Capacity or Rating	Source	Average Performance from 2015 - 2016, 2020 - 2022	Average No. of Units in Service from 2015 - 2016, 2020 - 2022	MOP 8 ⁽¹⁾ (or Typical Values)	Criteria Used for Capacity Analysis
IPS	Flow Rate	mgd	2 at 30 each 2 at 27 each 84 firm 114 installed	O&M Manual	15.3 Average 30.9 Average PWWF 34.1 Maximum PWWF	Not Evaluated	Sufficient firm capacity (i.e., one-unit OOS) to pump PWWF	Sufficient installed capacity (i.e., all units in service) to pump PWWF.
	Overflow Rate at AADF	gpd/sf	1,042	O&M Manual	1,151		800-1,200	1,042
	Overflow Rate at PWWF	gpd/sf	2,500	Calculated based on 2.4 peaking factor	2,485 Average 2,845 Peak		2,000-3,000	2,500
PSTs	Percent BOD ₅ Removal at AADF	percent	37	O&M Manual	31	2 of 4 (Weekdays) 3 of 4 (Weekends)	25 - 40	N/A
	Percent TSS Removal at AADF	percent	67	O&M Manual	62		50 - 70	N/A
	PE Concentration at Maximum Month Load during AADF	mg/L	COD = 356 - 370 BOD ₅ = 170 TSS = 102.5 - 115 NH ₃ = 25.9	Pre-Design Report and O&M	COD = 580 ⁽²⁾ BOD ₅ = 296 ⁽²⁾ TSS = 186 ⁽²⁾ NH ₃ = 47 ⁽²⁾		Variable depending on WW strength and primary treatment performance	COD = 580 ⁽²⁾ BOD ₅ = 296 ⁽²⁾ TSS = 186 ⁽²⁾ NH ₃ = 47 ⁽²⁾
Aeration Tanks	Minimum Month MLSS Temperature	degrees Celsius	-	-	23	1.9 of 2	Variable depending on climate	23
	aSRT	Days	-	-	8		Variable depending on treatment objectives and desired safety factor	5
	SVI in Pass 4	mL/g	-	-	137 Average 188 - 90th Percentile		150	150
	MLSS in Pass 4	mg/L	-	-	3,939		1,500 - 3,500 for step feed	2,800 mg/L is based on average SVI of 150 mL/g. Degraded settleability will lower allowable MLSS and capacity.
	Process Aeration	scfm	2 at 22,500 2 at 10,000 42,500 firm 65,000 installed	O&M Manual	20,100 Average 24,500 Maximum Month 28,000 Maximum Peak Day		Variable	Sufficient firm capacity (i.e., one-unit OOS) at peak day load.

Process	Design Parameter	Units	Original Design Capacity or Rating	Source	Average Performance from 2015 - 2016, 2020 - 2022	Average No. of Units in Service from 2015 - 2016, 2020 - 2022	MOP 8 ⁽¹⁾ (or Typical Values)	Criteria Used for Capacity Analysis
FSTs	SOR at AADF	gpd/sf	694	O&M Manual	441	11.6 of 13	400 - 800g	380 is recommended overflow rate based on SVI of 150 mL/g and recommended MLSS of 2,800 mg/L. Degraded settleability, or higher MLSS concentration will reduce recommended overflow rate.
	SOR at PWWF	gpd/sf	1,666	Calculated based on 2.4 peaking factor	1,019 Average Peak 1,167 Maximum Peak		1,000 - 1,200	910 is recommended overflow rate-based SVI of 150 mL/g and recommended MLSS of 2,800 mg/L. Degraded settleability, or higher MLSS concentration will reduce recommended overflow rate.
	SLR at AADF	ppd/sf	-	-	27		20 - 30	18 ⁽³⁾ is recommended SLR based on SVI of 150 mL/g and recommended MLSS of 2,800 mg/L. Degraded settleability, or higher MLSS concentration will reduce recommended SLR.
	SLR at PWWF	ppd/sf	-	-	55 Average Peak 67 Maximum Peak		35 - 40	43 ⁽³⁾ is recommended SLR based on SVI of 150 mL/g and recommended MLSS of 2,800 mg/L. Degraded settleability, or higher MLSS concentration will reduce recommended SLR.
RAS Pump Station	Flow Rate	mgd	3 at 7.2 2 at 8.5 30.1 firm 38.6 installed	O&M Manual	15.5 Average 30.8 Peak Day	Not Evaluated	Sufficient firm capacity (i.e., one-unit OOS) to pump 100 percent of AADF or minimum required by state point analysis.	Sufficient installed capacity (i.e., all units in service) to pump 100 percent of AADF or minimum required by state point analysis.
Effluent Filters	Hydraulic Loading Rate at AADF	gpm/sf	3.4	O&M Manual	2.4	7.8 of 10	2 - 6	3.4
	Hydraulic Loading Rate at PWWF	gpm/sf	5.0 (1-unit OOS)	O&M Manual	5.8 Average Peak 7.4 Maximum Peak		2 - 6	5.0
Filter Effluent Pump Station	Flow Rate	mgd	2 at 10.8 2 at 12.45 34.1 firm 46.5 installed	O&M Manual	13.9 Average 20.7 Peak Day	Not Evaluated	Sufficient firm capacity (i.e., one-unit OOS) to pump PWWF	Sufficient installed capacity (i.e., all units in service) to pump PWWF.
CCT	Theoretical CT at AADF	min	150	O&M Manual	278 Average 232 Minimum Month	3 of 3	450 CT for Title 22 Requirements at PWWF. 450 CT and 90 mins modal contact time for Title 22 requirements at PDWF	450 CT for Title 22 requirements at PWWF. 450 CT and 90 mins modal contact time for Title 22 requirements at PDWF.
	Modal CT at AADF	min	-	-	208 Average ⁽⁴⁾ 174 Minimum Month ⁽⁴⁾	-	75 percent hydraulic efficiency.	

Notes:

Abbreviations: gpm/sf - gallons per minute per square foot; gpd/sf - gallons per day per square foot; IPS - influent pumping station; min - minute; O&M - operations and maintenance; scfm - standard cubic feet per minute; SOR - surface overflow rate.

(1) Water Environment Federation Design of Water Resource Recovery Facilities, Manual of Practice 8.

(2) PE concentrations based on data average data from September 2017 through August 2022.

(3) SLR calculated using average recycle rate from 2015 - 2016 and 2020 - 2022 (100 percent).

(4) Modal time calculated with an assumed hydraulic efficiency of 75 percent.

Primary Sedimentation Tanks

PE samples for TSS are collected daily and samples for BOD₅ are collected weekly at LBWRP. Percent removal for selected evaluation period (2015 - 2016, 2020 - 2022) along with the target design values are shown on Figure 3.3. The figure presents the 30-day running average of the percent removals (shown as solid color lines) compared to the target design values.

The average TSS and BOD₅ removal achieved is 62 and 31 percent, respectively. The TSS and BOD₅ removals are slightly lower than the target design values but falls within the typical range for primary clarifier removal rates. The typical primary clarifier removal rates for TSS and BOD₅ range from 60 to 65 and 30 to 35 percent, respectively.

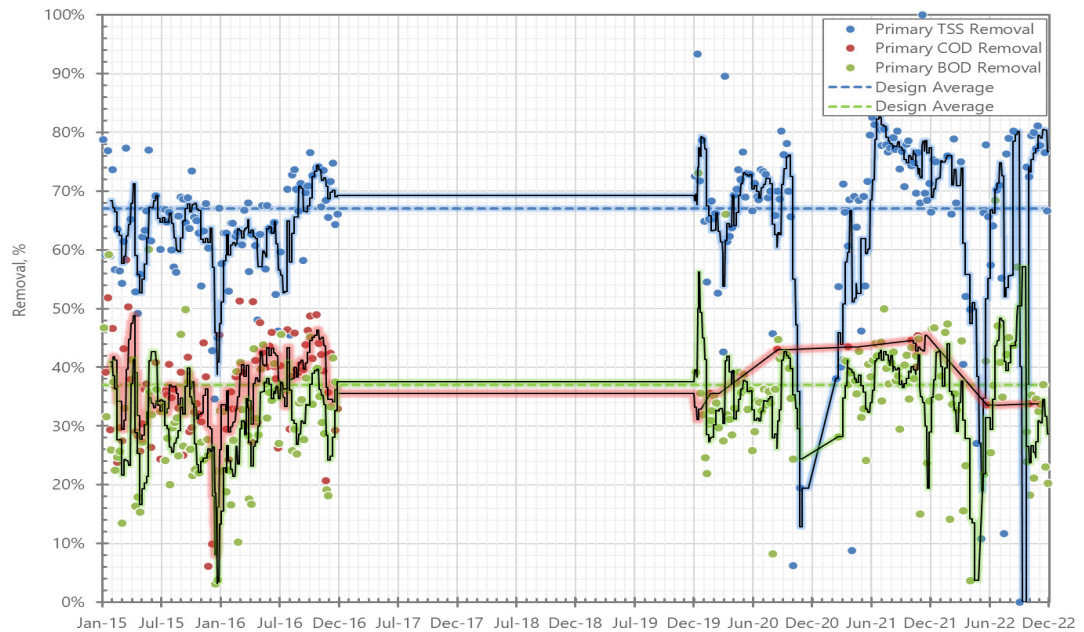


Figure 3.3 PSTs Percent TSS, COD, and BOD₅ Removal

PE concentrations for COD, BOD₅, TSS, and NH₃ for the last five years (September 2017 - August 2022) is shown on Figure 3.4. The PE concentrations for COD, BOD₅, TSS, and NH₃ are higher than the original basis of design, which reflect higher influent concentrations at LBWRP compared to the original design criteria. The 30-day running average of the daily data (shown as solid color lines) compared to the original design averages is presented on Figure 3.4. PE concentrations were significantly higher in early 2021 and much of 2022, most likely due to construction activities that required taking PSTs OOS, and due to problems with draining waste discharge to the downstream sewer (JOA-1A Long Beach WRP Sludge Gravity Sewer). PE concentration trends for TSS and BOD roughly correlate to water depth trends in the downstream sewer at manhole A 0421. Waste from LBWRP recirculates back into the LBWRP when the waste overtops a stoplog at one LBWRP influent sewer (Palo Verde Trunk), manhole 19 0010.

It is recommended that the historical maximum month concentrations be used for assessing the capacity of the secondary process. Maximum month concentrations are typically used for design and capacity assessment. Note that the data period (2017 - 2022) used for evaluating PE

concentrations is different than the data used for performance assessment (2015 - 2016, 2020 - 2022). This decision was made as the 2017 - 2022 concentration data is believed to be more representative of the current conditions. Most water reclamation plants in California are seeing a trend of increasing WW strength due to drought and water conservation. Moreover, since the capacity of the secondary process is largely dependent on loads (and several other factors), it is imperative to use representative concentrations for capacity analysis.

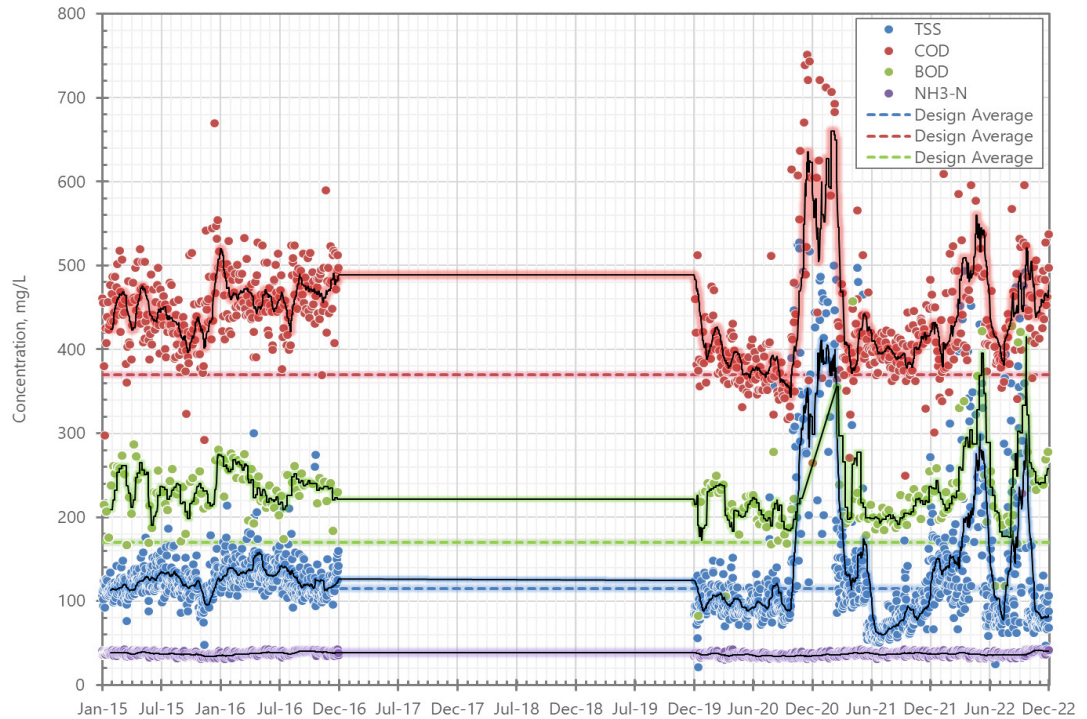


Figure 3.4 PE Water Quality

Historical PE concentrations from 2017 through 2022 are compared to the original design basis in Table 3.9. Loads were calculated from the flow and PE concentration for various parameters on days PE concentrations were measured for COD, BOD₅, TSS, and NH₃. Maximum month loads were estimated as the 91.7 percentile value from the load data set, and peaking factors were calculated as the maximum month load divided by the average load. The maximum month concentrations were calculated using the maximum month load and the average flow from the review period. Maximum month PE concentrations compared to their original design basis are in the range of 50 - 80 percent as indicated in Table 3.9. Observed high PE concentrations will ultimately reduce the calculated flow capacity because the secondary process largely depends on loads. The impact of high PE concentrations will be evaluated in the capacity section of this TM.

Table 3.9 PE Concentrations

Parameter	Average Concentration From 2017 - 2022 (mg/L)	Average Load From 2017 - 2022 (klb/d)	Maximum Month Load From 2017 - 2022 (klb/d)	Maximum Month Load Peaking Factor	Maximum Month Concentration From 2017 - 2022 (mg/L) ⁽¹⁾	Original Design Basis (mg/L) ⁽²⁾	Percent Increase in Maximum Month Concentration from Original Design
COD	412	47.0	65.8	1.40	580	356 - 370	57% - 63%
BOD ₅	213	25.0	33.6	1.36	296	170	74%
TSS	118	13.0	21.2	1.65	186	102.5 - 115	62% - 82%
NH ₃	36	3.9	5.3	1.37	47	25.9	82%

Notes:

Abbreviations: klb/d - thousand pounds per day.

(1) Concentrations calculated from maximum month load occurring during AADF (13.6 mgd for this data set).

(2) From O&M manual and nitrification/denitrification (NdN) preliminary design.

Aeration Tanks

The aeration tanks are designed to remove soluble BOD₅, NH₃, and nitrogen (NO₃ + NO₂) to meet the effluent limits. Data indicate good performance for the review period, although there are periods where NH₃, NO₂, or NO₃ temporarily increase or break through (Appendix 3E). The water reclamation plant is operated to maintain a consistently low concentration of effluent NH₃ to facilitate good control of the chloramination disinfection process. The key operating parameters for the step feed NdN activated sludge process are MLSS concentration, solids retention time (SRT), PE flow distribution, SVI, and process air consumption (PAC). These parameters were evaluated and are described in detail below.

MLSS Concentration: The MLSS concentration affects the operating SRT and treatment capacity. Laboratory data for average MLSS concentration in each pass in Stage 1 and Stage 2 units is shown in Table 3.10.

Table 3.10 Laboratory Data for MLSS Concentration

Pass	Average MLSS Concentration Stage 1 (ML_05), mg/L	Average MLSS Concentration Stage 2 (ML_01), mg/L
1	4,724	4,804
2	4,642	4,499
3	4,225	3,823
4	3,893	3,807
Average MLSS	4,371	4,233

Note that the concentrations decrease through passes 1 through 4, which is expected in a step feed process. All the RAS is fed to the first pass; therefore, the PE feed reduces the MLSS concentration at each step feed location in Pass 2 and 3.

The MLSS concentration in Pass 4 is particularly important to the overall performance of the secondary process as the feed to the FSTs. While higher MLSS concentrations result in longer SRTs and more aeration tank capacity, high MLSS also increases the solids load and reduces FST capacity. During wet weather events, it is critical to manage the MLSS inventory and maintain concentrations in Pass 4 that do not overload the FSTs. MLSS concentration for Pass 4 for the Stage 1 and Stage 2 aeration tanks is shown on Figure 3.5.

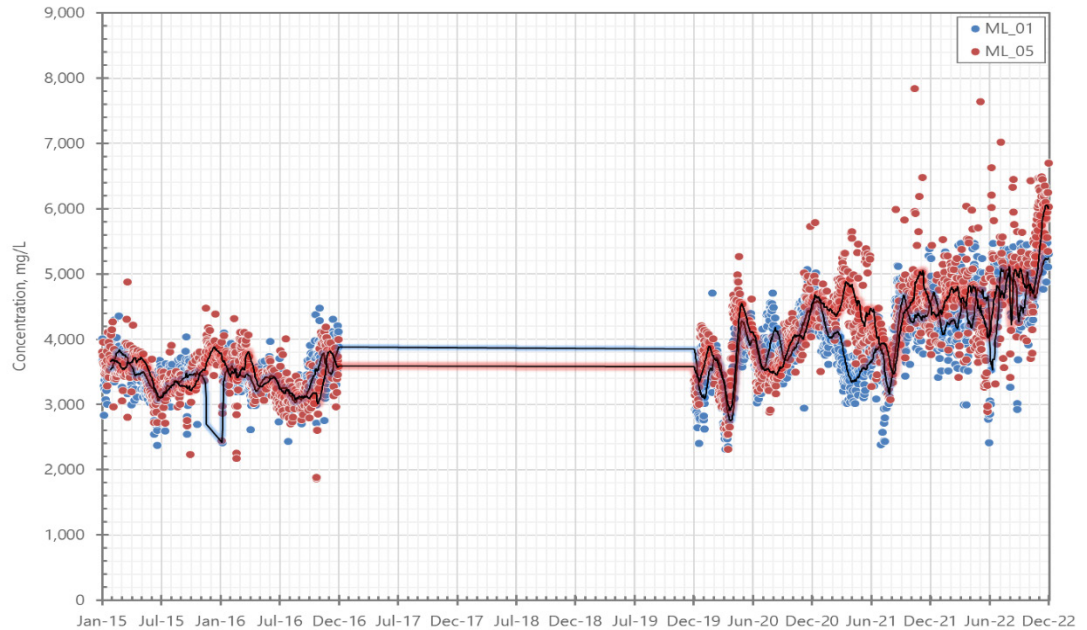


Figure 3.5 MLSS (Pass 4, Units 1 and 2)

SRT: Total SRT is defined as the total mass of solids in the aeration tanks divided by the total mass of the solids leaving the system in the waste activated sludge and secondary effluent, which is a measure of the average sludge age. The aSRT reflects the portion of the total MLSS that is under aerobic conditions. The combined total SRT and aSRT for both stages during the review period is shown on Figure 3.6. The combined total SRT for Stages 1 and 2 averages 11 days with an aSRT of 8 days.

The historical aSRT is sufficient to achieve NH_3 removal. The minimum recommended aSRT for LBWRP is approximately five days. The recommended minimum aSRT of five days is 2.5 - 3.5 times higher than the washout aSRT at the observed minimum month temperature of 23 degrees Celsius. The aSRT at which nitrifier growth rate is too low to accumulate a nitrifier population is called the nitrifier washout aSRT. The washout SRT is calculated as a function of nitrifier growth rate and minimum temperature. When operating below the washout aSRT, there is inadequate solids residence time to accumulate a nitrifier population. Therefore, a recommended safety factor of 2.5 - 3.5 is typically applied to the washout SRT to prevent changes in influent characteristics and operating conditions from impacting complete nitrification. The recommended operating aSRT of 5 days will maximize the secondary process capacity. Performance will not be negatively affected if the WRP is operated at a higher aSRT than recommended; however, the secondary treatment capacity will be reduced. Currently, the secondary process is controlled by maintaining a target MLSS instead of a target SRT or aSRT. For that reason, the MLSS concentration is relatively stable while the SRT varies as shown on Figure 3.5. It is recommended that the Sanitation Districts consider SRT control to maximize secondary process capacity, as discussed in TM 4.

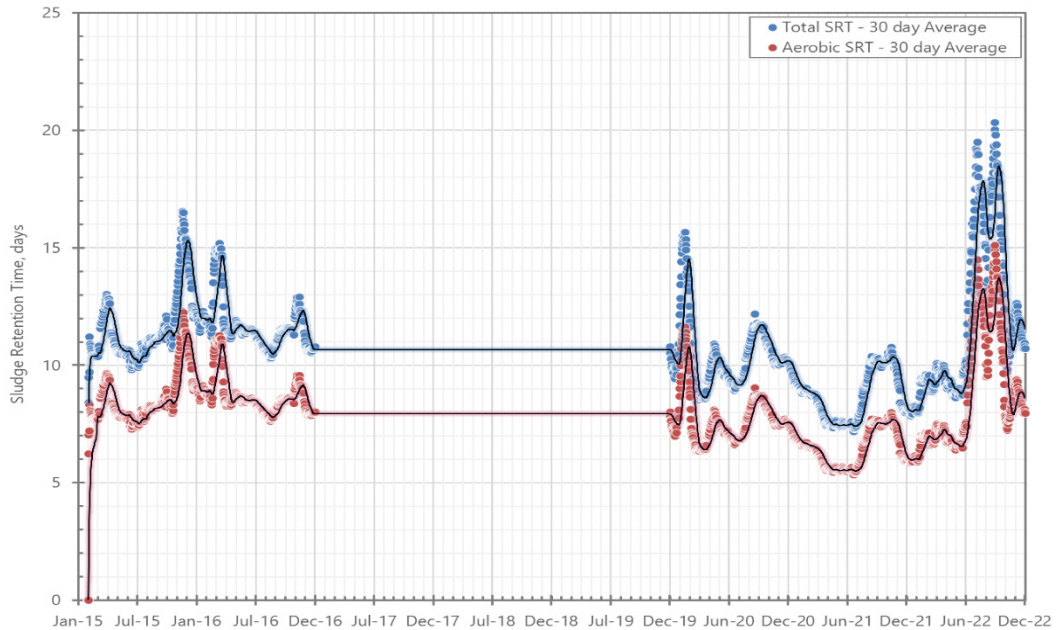


Figure 3.6 Total SRT and aSRT

PE Flow Distribution: Good PE flow distribution is important for maximizing secondary process capacity in a step feed process. PE enters each aeration tank in three locations and is controlled with manually operated sluice gates. Operations target a split of 55/30/15 in Passes 1, 2, and 3 and each gate has a dedicated propeller meter. Although in prior discussion with the Sanitation Districts’, it was indicated that the propeller meters might not be reliable.

Operations periodically adjust gate positions manually at the PE feed locations and rely on visual observation when changes to the split are required. The Sanitation Districts has investigated and evaluated improving the accuracy of the flow split. In general, it is most important to have accurate flow splits during peak flow periods, which has been the Sanitation Districts’ focus. If the flow split is not optimal at lower flow conditions, it is considered acceptable as it has not shown to have a negative impact on performance.

SVI: In addition to removing BOD₅ and nutrients, another performance objective of an aeration basin is to generate well-settling sludge, which is measured by the SVI. The SVI represents the volume of the solids in a mixed liquor sample after 30 mins of settling. In general, the lower the SVI, the faster the solids settle. The SVI is important as it directly affects the capacity of the downstream clarifiers and secondary process overall. Higher SVI can require that the aeration tanks maintain a lower MLSS concentration to avoid clarifier overload. A lower MLSS concentration results in less biomass for treatment, a lower SRT, and reduced overall secondary capacity. The opposite is true with lower SVI’s (i.e., can operate with higher MLSS resulting in more capacity).

SVI in Pass 4 at LBWRP has improved over the last few years as indicated on Figure 3.7. The average Stage 1 SVI is approximately 145 mL/g and average Stage 2 SVI is 127 mL/g which is indicative of an adequate settling sludge. However, there are periods during both dry and wet weather when the SVI is over 200 mL/g. An SVI less than 150 mL/g is considered good for a

nutrient removal activated sludge process. In 2022, the SVI's were consistently less than 150 mL/g the entire year.

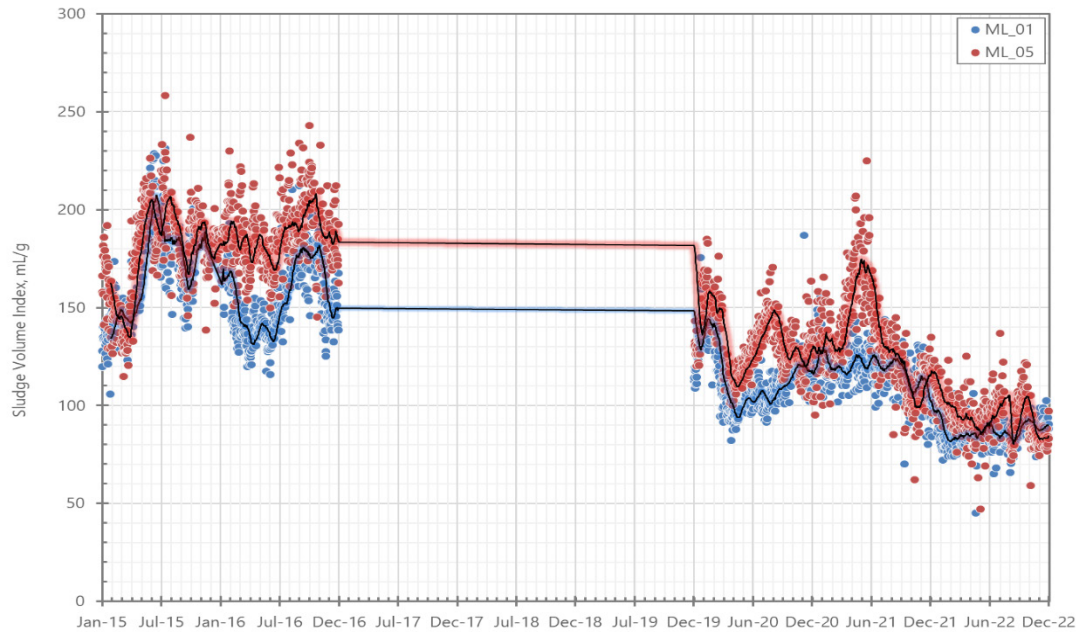


Figure 3.7 SVI (Units 1 and 2)

PAC: It is necessary to understand the alpha factor to understand the performance and capacity of an aeration system. Since all WWs are different, aeration equipment is usually specified under standard conditions for clean water (i.e., tap water). Alpha, along with several other parameters, is used to empirically convert clean water oxygen transfer to field conditions in WW. Alpha is a function of the WW characteristics and process operating conditions.

The BioWin process model for LBWRP was used to assess the aeration efficiency using historical aeration data. The model predicted aeration demands matched historical data at an average alpha value of 0.37. At the SRTs observed at LBWRP, a higher alpha of 0.50 or greater is more typical, and what has been reported by the Sanitation Districts with off-gas testing at LBWRP, which directly measures alpha. This difference suggests that the diffuser performance in the process model does not accurately match what is in operation. It is also possible that the diffuser 450 CT performance has degraded over time or that there are leaks in the aeration delivery system.

Chlorine Contact Tank

As described in TM 2, there are several instances where effluent did not meet 450 CT with the existing CCT volume. During the site visit, the operations staff mentioned the configuration of CCT is such that the reuse water is pulled directly from the CCT. Water level in the CCT varies due to variation in reuse demand, which results in operational difficulty to maintain 450 CT. CCT modifications and additional CCT volume may be required to meet 450 CT when the new WRRs are issued.

3.6 Capacity Evaluation

This section summarizes the results of the process capacity evaluation. AADF and PWWF capacity is developed for each unit process. The reliable AADF capacity was also estimated, which is the AADF capacity when one of the largest process units is taken OOS.

Capacities were estimated for each unit process and are dependent on a range of parameters including flow, influent WW characteristics, treatment objectives, process configurations, operational setpoints, and desired redundancy. Historical loading rates and performance were reviewed and recommended capacity rating criteria were developed for each unit process as part of the performance assessment. Capacities are based on the criteria summarized in Table 3.8.

3.6.1 Approach to Capacity

3.6.1.1 Peak Wet Weather Flow Capacity

The PWWF process capacity was estimated for all liquid stream facilities. The general approach for estimating PWWF capacity is summarized below:

- PWWF capacity was calculated based on the PWWF criteria summarized in Table 3.8 and compared to the projected flow in Table 3.6.
- Assumed all process units in service.
- Pump station capacity was calculated based on installed capacity.
- Aeration tanks and FSTs were assigned the same PWWF capacity since both processes are integral to each other, and depend on several factors including the SRT, MLSS concentration, SVI, temperature, and flow distribution.
 - BioWin process modeling was performed to establish MLSS concentrations at maximum month loading conditions, and peak day aeration demands for a range of flow and SRT scenarios. PE flow split between the aeration tanks was assumed to be equal. PE flow split to the step feed passes was assumed to meet the target of 55/30/15. An aSRT of five days was used in the capacity rating.
 - PWWF FST capacity was estimated for a range of MLSS concentrations using state point analysis. An SVI of 150 mL/g was used in the capacity rating as a reasonable “worst case” settleability. The SVI is mostly lower than 150 mL/g, especially in 2022.
 - The estimated capacity is based on the MLSS concentration that maximizes the overall secondary process capacity.
- Filter PWWF capacity was calculated based on the criteria in Table 3.8 and compared to the projected flow in Table 3.6 (using the Sanitation Districts’ procedure for effluent filter design criteria).
- CCT capacity was not based on PWWF but is instead expressed in terms of the PDWF and an assumed reactor hydraulic efficiency of 75 percent.

Note that the PWWF process capacity is based on treatment capacity and does not consider hydraulic capacity or restrictions. The hydraulic capacity was evaluated in Section 3.4 of this TM.

3.6.1.2 Average Annual Daily Flow Capacity

The AADF capacity was estimated for process facilities where sizing is established by AADF loading criteria, or influent BOD₅, NH₃, and TSS load to the plant. The general approach for estimating AADF capacity is summarized below:

- Applied AADF criteria summarized in Table 3.8.
- All process units were assumed to be in service.
- An equivalent AADF criteria was not estimated for the pump station because capacity is driven by PWWF.
- Aeration tanks and FSTs were assigned the same capacity as both processes are integral to each other, as described in the previous section. The same general approach was used to generate the AADF capacity, with a few additional items to note:
 - The AADF capacity for the FSTs was estimated by dividing the PWWF capacity by the appropriate peaking factor of 2.4 (PWWF/AADF = 2.4).
 - Blower capacity is based on firm capacity with the largest unit OOS.
- Filter AADF capacity was calculated based on the criteria in Table 3.8 and compared to the projected flow in Table 3.6.
- CCT capacity is limited by PDWF conditions, so an equivalent AADF rating was developed by dividing the PDWF capacity by the appropriate peaking factor of 1.73 (PDWF/AADF = 1.73).

3.6.1.3 Reliable Average Annual Daily Flow Capacity

The reliable AADF capacity is estimated for each process when the largest unit is OOS. When possible, preventative maintenance is normally performed during average, or dry weather periods, so this condition was only estimated for the AADF. The number of process units in service when estimating the reliable AADF capacity is summarized in Table 3.11. The units OOS for reliable AADF capacity were determined based on the typical number of units OOS for the review period.

Table 3.11 Reliable AADF Analysis Criteria

Process	Total	Units in Service	Units OOS
PSTs	4	3	1
Aeration Tanks	2	1	1
FSTs	13	12	1
Effluent Filters	10	8	2
CCT	3	3	0

3.6.2 Capacity Summary

The estimated capacity for each unit process at LBWRP is presented on Figure 3.8 and Table 3.12 based on the recommended criteria for capacity analysis in Table 3.8. All unit processes have sufficient capacity to handle the estimated 2050 projections except for the secondary process (aeration tanks and FSTs) and effluent filters as shown in Table 3.12. The effluent filter pump station has sufficient installed capacity for projected 2050 flows, however, there is not enough firm capacity with the largest unit OOS during the projected PWWF. A discussion is provided in the subsection below since the unit process capacity for secondary treatment is dependent on several factors.

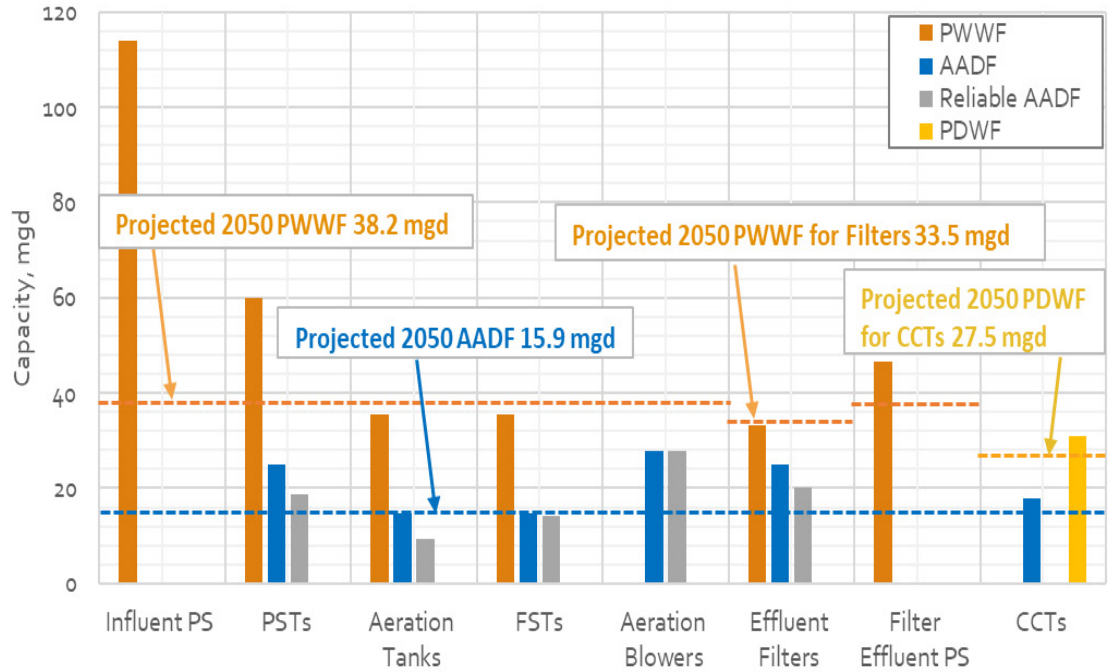


Figure 3.8 Unit Process Capacity

Table 3.12 Unit Process Capacity Ratings

Process	Controlling Condition	Capacity, mgd	Projected 2050 Flow, mgd	Capacity Deficit, mgd
IPS	PWWF	84 ⁽¹⁾ – 114 ⁽²⁾	38.2	0
PSTs	PWWF	60	38.2	0
	AAAF	25	15.9	0
	Reliable AAAF	19 ⁽³⁾	--	--
Aeration Tanks	PWWF	35.5 ⁽⁴⁾	38.2	2.6
	AAAF	14.8 ⁽⁴⁾	15.9	1.1
	Reliable AAAF	9.4 ⁽⁵⁾	--	--
FSTs	PWWF	35.5 ⁽⁴⁾	38.2	2.6
	AAAF	14.8 ⁽⁴⁾	15.9	1.1
	Reliable AAAF	14.2 ⁽⁶⁾	--	--
Aeration Blowers	AAAF	27.8	15.9	0
	Reliable AAAF	27.8	--	--
Filters	PWWF	33.2	33.5 ⁽⁸⁾	0.3
	AAAF	25.1	15.9	0
	Reliable AAAF	20.1 ⁽⁷⁾	--	--

Process	Controlling Condition	Capacity, mgd	Projected 2050 Flow, mgd	Capacity Deficit, mgd
Effluent Filter Pump Station	PWWF	34.1 ⁽¹⁾ – 46.5 ⁽²⁾	38.2	0 ⁽²⁾ – 4.1 ⁽¹⁾
Disinfection	PDWF	31.0 ⁽⁹⁾	27.5 ⁽¹⁰⁾	See Section 3.6.2.2 ⁽¹¹⁾
	AADF	17.9 ⁽¹⁰⁾	15.9	0

Notes:

- (1) Based on firm capacity (one pump OOS).
- (2) Based on total capacity (all pumps in service).
- (3) Based on one PST OOS.
- (4) Based on maximum month load conditions, which correspond to PE concentration of: COD = 580, BOD₅ = 296, TSS = 186, and NH₃ = 47 mg/L. Other assumptions include an aSRT of 5 days and resulting MLSS concentration of 2,800 mg/L, SVI of 150 mL/g, and a PWWF/AADF peaking factor of 2.4.
- (5) Based on one aeration tank unit OOS.
- (6) Based on one FST OOS.
- (7) Based on two filters OOS.
- (8) Based on the Sanitation Districts’ new procedure on effluent filter design criteria described in Appendix 3C.
- (9) Based on CCTs meeting Title 22 Requirements of 450 mg-min/L CT.
- (10) Calculated using a peaking factor (PDWF/AADF) of 1.73.
- (11) While the CCT volume is sufficient for future flows, the operation of the reuse system impacts water levels and prevents full use of the CCT volume. Modifications to recycled water permit will require full compliance with 450 CT in the future. Therefore, either additional volume must be added, or the reuse operations will need to be modified for compliance with future 450 CT requirements for PDWF.

3.6.2.1 Secondary Treatment Processes (Aeration Tanks and Final Sedimentation Tanks)

The steady state BioWin process model developed by the Sanitation Districts was used to simulate process conditions under maximum month loading to determine the AADF capacity of the aeration basins. The model was used to estimate the MLSS concentration in Pass 4 over a range of flow and aSRT conditions. Currently, the plant is typically operated at an aSRT of eight days. Simulations for capacity were performed at an 8-day and a 5-day aSRT. As described in the performance assessment section, the minimum recommended aSRT, to provide adequate safety factor to prevent nitrifier washout, is five days.

The FST capacity is based on its ability to settle sludge, which is dependent on the MLSS concentration and SVI. State point analysis was performed for an SVI of 150 mL/g and used to estimate the PWWF capacity over a range of recommended MLSS and settleability conditions. The PWWF capacity was converted to an equivalent AADF capacity using a PWWF/AADF peaking factor of 2.4. The aeration tank and FST AADF capacity over a range of SRT, MLSS concentration, and an SVI of 150 mL/g is shown on Figure 3.9.

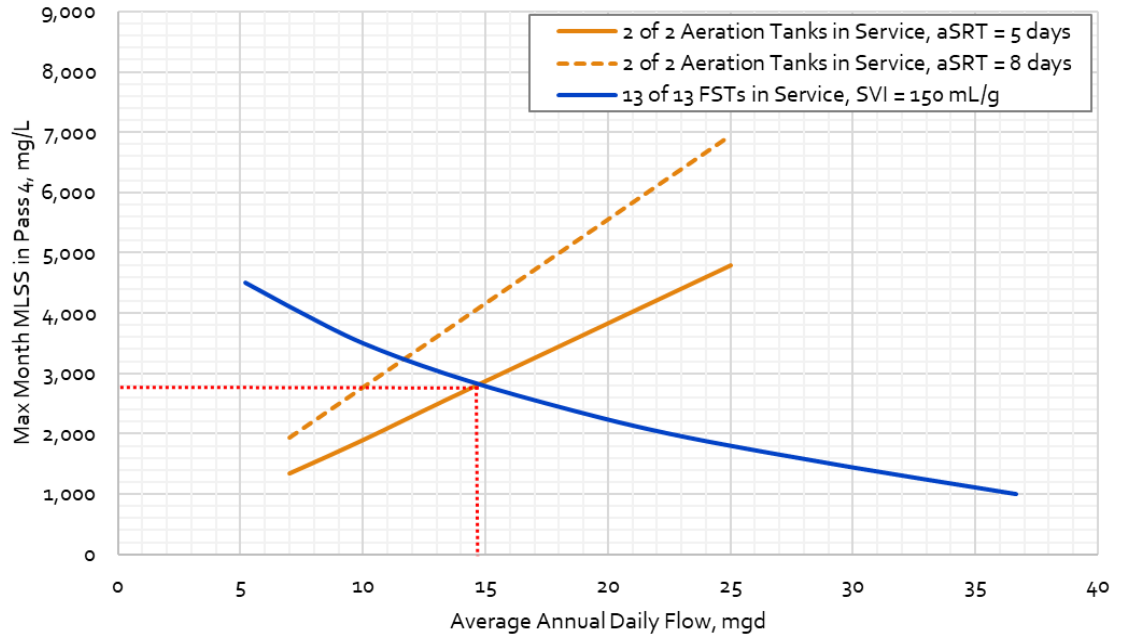


Figure 3.9 Secondary Treatment System Capacity With All Aeration Tanks and Clarifiers in Service

The recommended capacity rating for the LBWRP secondary facilities is 14.8 mgd AADF, which represents a target MLSS concentration of approximately 2,800 mg/L, and the intersection of the orange 5-day aSRT line and the blue FST capacity line estimated from the SVI of 150 mL/g. If the settleability were degraded, then the capacity would be reduced.

The reliable secondary treatment capacity assuming one aeration tank is taken OOS, or one FST is taken OOS is illustrated on Figures 3.10 and 3.11. The reliable AADF capacity is 9.4 mgd with one aeration tank OOS and 14.2 mgd with one FST OOS. The impact on operating MLSS by taking one aeration tank versus one FST OOS is slightly different. If an aeration tank is taken OOS, the target MLSS in Pass 4 is 3,600 mg/L. If one FST is taken OOS, then the target MLSS in Pass 4 is 2,800 mg/L.

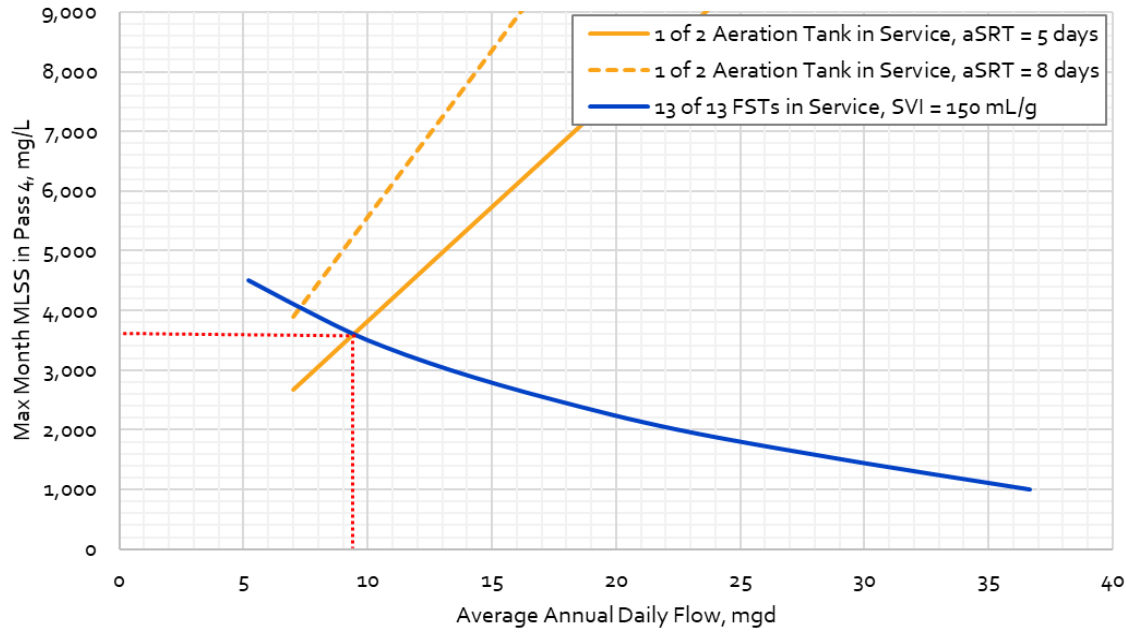


Figure 3.10 Secondary Treatment System Reliable Capacity With 1 of 2 Aeration Tanks in Service

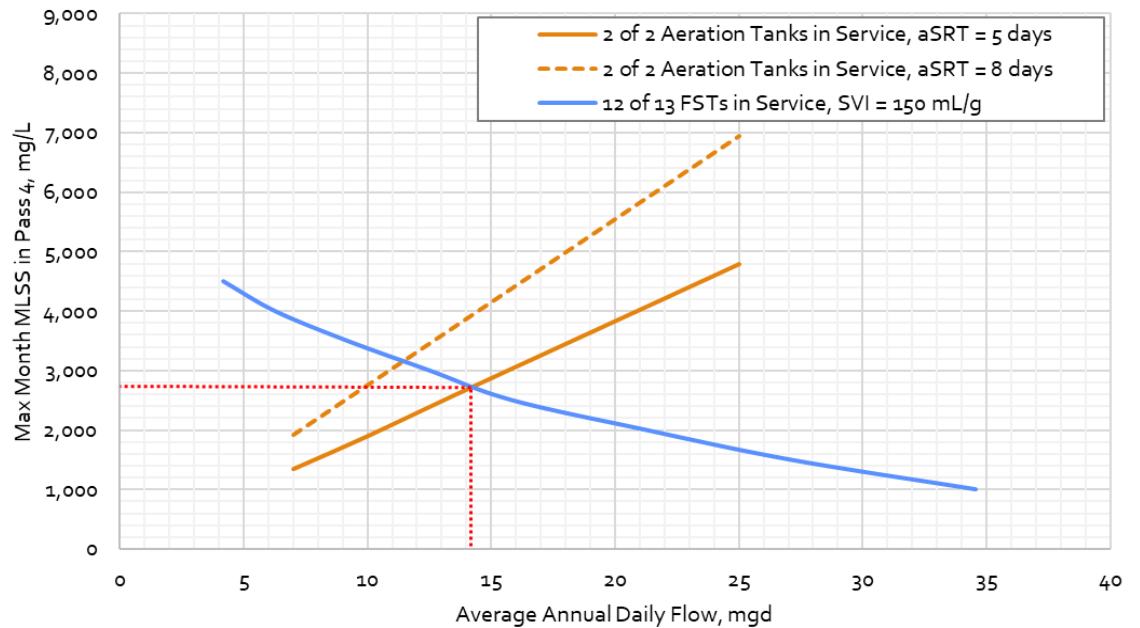


Figure 3.11 Secondary Treatment System Reliable Capacity With 12 of 13 FSTs in Service

The BioWin model was used to predict the air demand at peak day loads. The process aeration results are shown on Figure 3.12. The blower firm capacity exceeds the projected peak day air demand of 15.9 mgd AADF.

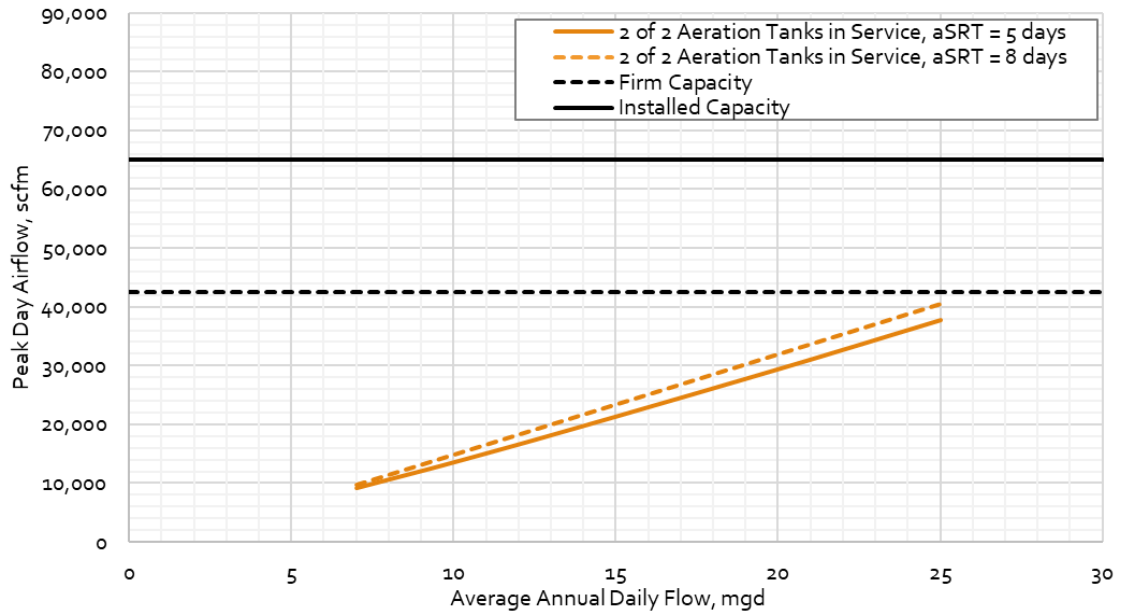


Figure 3.12 Projected Peak Day Process Air Demands

3.6.2.2 Disinfection Process (Chlorine Contact Tanks)

The existing CCT volume is adequate to meet the future 450 CT and 90-minute modal CT requirement for the 2050 projected PDWF. However, reuse operations cause water levels to fluctuate as described in Section 3.5.2, making future compliance with 450 CT difficult. Upgrades to the CCT structure to separate flows to reuse and Coyote Creek or additional CCT volume will be required to provide sufficient volume to meet the future 450 CT requirement. These upgrades are discussed in TM 4 – Gaps and Needs.

3.7 Recent Wet Weather Events and Operational Strategies

In February 2024, LBWRP experienced an unusual wet weather event resulting in high influent flows to the plant. The unusual wet weather event triggered a re-evaluation of the unit process capacity ratings. As part of this effort, Carollo analyzed the wet weather data from February 4-5, 2024 to determine the recurrence frequency of the event. Figure 3.13 shows the recurrence frequency of the wet weather event. As seen in the figure, the storm is categorized as a 10-year recurrence frequency based on a 24-hour event and between 25 to 50-year recurrence frequency based on a 48-hour event.

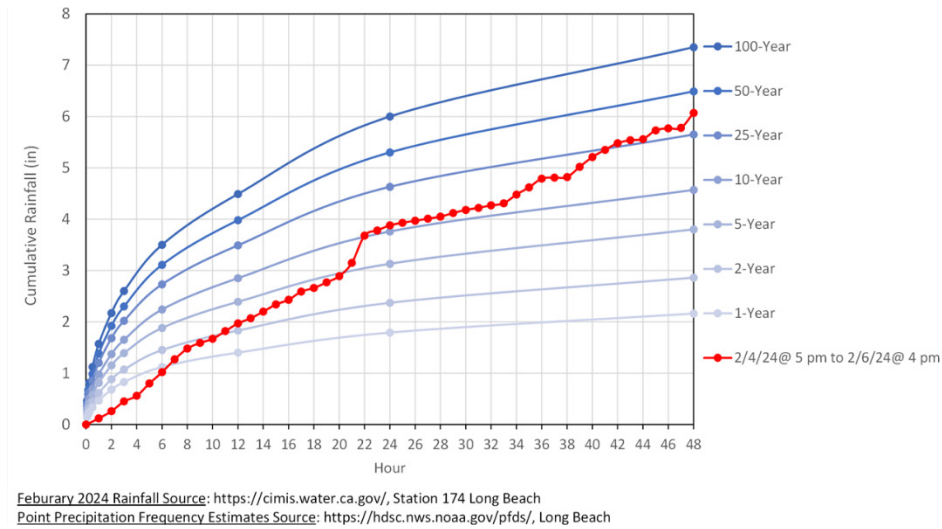


Figure 3.13 Recurrence Frequency for February 2024 Storm at Long Beach

Additionally, wet weather data for the last 10 years were analyzed, and peak wet weather peaking factors were developed. Table 3.13 shows the peaking factors starting 2015 through present. As seen in Table 3.13, the average PWWF peaking factor is 2.3 which is similar to the peaking factor of 2.4 used for performance and capacity evaluation. Therefore, the performance and capacity of LBWRP were not re-evaluated. Additionally, for wastewater treatment unit processes, it is typical to use average PWWF peaking factors for performance and capacity assessments instead of maximum PWWF peaking factors to avoid designing these systems for relatively rare wet weather flows.

Table 3.13 Peaking Factors for Wet Weather Analysis

Year	PWWF, mgd	AADF, mgd	Peaking Factor
2015 ⁽¹⁾	29.1	14.7	2.0
2016 ⁽¹⁾	26.2	16.4	1.8
2017 ⁽¹⁾	53.4	14.0	3.8
2018 ⁽¹⁾	29.3	12.7	2.3
2019 ⁽¹⁾	33.4	11.9	2.8
2020 ⁽¹⁾	28.7	15.2	2.3
2021 ⁽¹⁾	33.7	15.6	2.4
2022 ⁽¹⁾	24.3	15.1	1.8
2023 ⁽²⁾	36.1	16.4	2.6
2024 ⁽²⁾⁽³⁾	47.6	18.3	3.4
Average	34.2	15.0	2.3

Notes:

- (1) Hourly flow and peaking factors from TM 1.
- (2) Flow and peaking factors from hourly data.
- (3) Includes data through April 2024 only.

However, the Sanitation Districts and O&M staff would like to be better prepared for such storm events and therefore, Carollo put together a few wet weather strategies that could be implemented for severe storm events. The strategies are discussed in detail in TM 4 – Gaps and Needs.

Appendix 3A

PROCESS DESCRIPTION AND DETAILED DESIGN CRITERIA

Process Description and Detailed Design Criteria

Long Beach Water Reclamation Plant (LBWRP) consists of primary sedimentation, aeration, final sedimentation, filtration, and disinfection processes. Each unit process and design criteria are described below.

3A.1 Influent Pumping Station

The influent pump station pumps the influent wastewater from the wet well to Channel No. 1. The influent pumping station at LBWRP consists of four horizontal, dry-pit, non-clog, centrifugal pumps with variable frequency drives. The pump station building is located on the east side of the influent end of the primary sedimentation tanks. It houses four pumping units in two interconnected dry wells. The South dry well contains Influent Pump Nos. 1 and 2 and the North dry well contains Influent Pump Nos. 3 and 4.

3A.2 Primary Sedimentation Tanks

Primary sedimentation system removes coarse, gritty material, most of the floating material, and a major portion of the suspended solids from the raw sewage. There are four covered rectangular primary sedimentation tanks (PSTs) with influent and effluent channels, sludge collection and draw-off equipment, and scum removal equipment. The raw influent is received from Channel No. 1 through 14-inch circular openings equipped with upward opening 24-inch square electrically operated slide gates and diffusers. Channel No. 1 is also equipped with four 12-inch square, downward opening skimmings gates used to prevent the accumulation of floatable material in the channel.

As the raw influent passes through the tank, the heavy solids settle at the bottom of the tank and the primary effluent travels over weirs. It is collected in Channel No. 2 for transport to the aeration tanks via Channel No. 3. The Stage 1 process air compressors (PACs) draw foul air under the primary sedimentation tank covers. The foul air flows through the length of the tanks, across Channel No. 2 and into the Air Take-Off Structure. This arrangement purges the tanks of odors and hydrogen sulfide, which can corrode concrete and steel. Odors in the air drawn from beneath the tank covers are scrubbed out by the activated sludge in the aeration tanks.

Stage 1 PACs supply air to diffusers located in various channels throughout the plant to prevent settling of solids. Settled solids and floatable materials collected by the flight-type sludge collection and scum removal equipment are returned to the sewer by the plant sewer line. The flights move along the bottom of the tanks and push the solids to the two hoppers at the inlet of the tank and returning flights move along the water surface and push floatables towards the scum removal equipment at the launder end of the tank.

3A.3 Aeration Tanks

This process consists of eight aeration tanks, four PACs, fine bubble ceramic air diffusers, aeration piping and associated instrumentation. The aeration tanks and compressor stations were constructed in two stages, which operate as independent units, including having their own sedimentation tanks and return activated sludge (RAS) pump stations. Stage 1 consists of Aeration Tanks 5 through 8, and Stage 2 consists of Aeration Tanks 1 through 4. The conventional step feed nitrification process was modified to also denitrify.

Primary effluent enters each aeration tank in three locations and is controlled with manually operated sluice gates. Each gate has a dedicated propeller meter; however, the meters are prone to ragging and are potentially unreliable. The target distribution of primary effluent in the aeration tank is 55 percent in the first pass, 30 percent in the second pass, and 15 percent in the third pass. The RAS from the final sedimentation tanks (FSTs) is also introduced at the front end of the first pass (Tanks 4 and 8), forming mixed liquor. The front end of Pass Nos. 1 through 3 operates as an anoxic zone where the primary effluent is introduced, and denitrification takes place. After the anoxic zone, mixed liquor enters the aeration zone in the same pass to complete nitrification and chemical oxygen demand removal. Pass No. 4 is operated as an all-aerobic zone. The mixed liquor leaves the aeration tank after Pass No. 4 and discharges to Channel No. 5 for the distribution to the FSTs. Figure 3A.1 is a flow schematic of the aeration tanks.

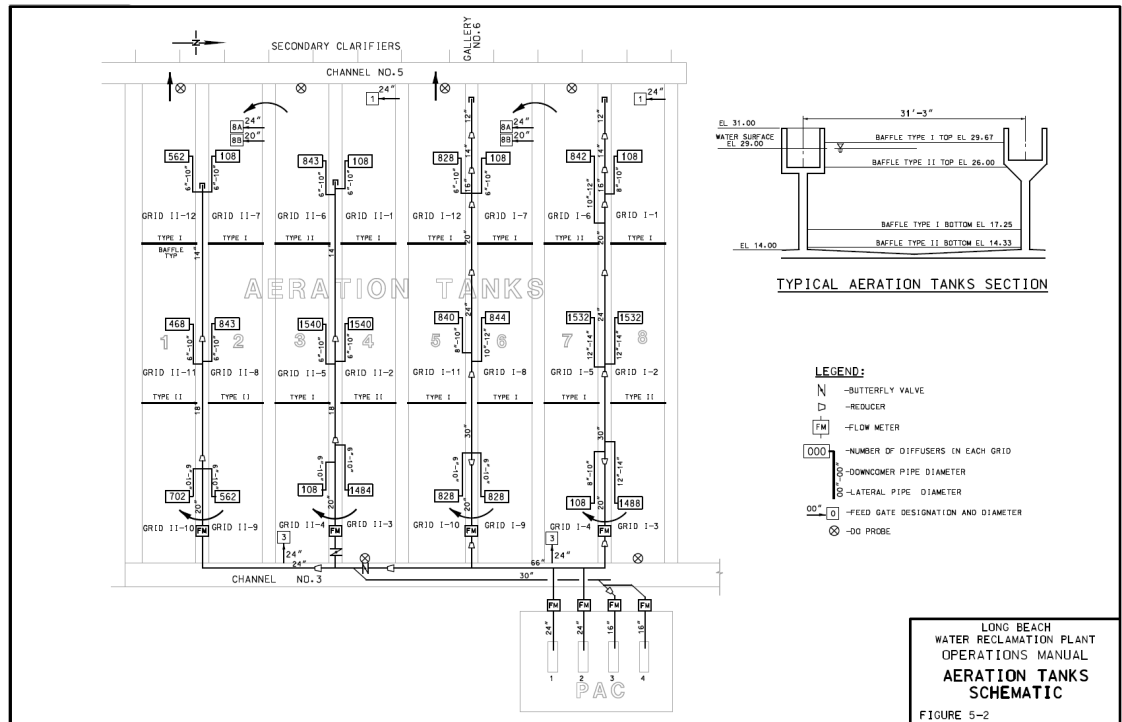


Figure 3A.1 Aeration Tanks Schematic

Each pass consists of three diffuser grids with variable number of fine bubble diffusers installed in each grid. Air is supplied to the diffusers through four PACs located in the PAC Building. Stage 1 PACs includes Units 1 and 2 and draw a combination of ambient air and foul air from the influent wet well and the primary sedimentation tanks. The Stage 2 PACs includes Units 3 and 4, which draw ambient air only. The PACs discharge into manifolds located in Gallery No. 3, which discharge in turn into air headers located between Pass Nos. 1 and 2 and Pass Nos. 3 and 4 of each aeration unit.

Each aeration unit has two locations where the flow of process air into the system is controlled. The first location is at the PAC where the inlet guide vanes modulate to regulate the total flow air entering the aeration unit. The second location is the control valve on the Pass No. 3/4 header, which modulated to control air flow to that header independently from the Pass No. 1/2 header, where the valve remains 100 percent open. Dissolved oxygen probes are installed at the end of the first, second, and fourth passes of each unit.

3A.4 Final Sedimentation Tanks

There are 13 FSTs, which include flight-type sludge collection equipment, skimmers, sludge draw-off valves, and control equipment. Tank Nos. 1 through 7 receive mixed liquor from the Stage 2 aeration tanks and supply RAS Pump Station No. 2, while Tank Nos. 8 through 13 receive mixed liquor from Stage 1 aeration tank and supply RAS Pump Station No. 1.

The mixed liquor from aeration tank enters each sedimentation tank from Channel No. 5 through three inlet diffusers, each controlled by a 14-inch square slide gate with a pedestal lift. Polymer is added to the mixed liquor to aid settling. The polymer system is described in Section 3A.9.1.1. Settled solids are collected by chain-and-flight type sludge collectors that move the sludge to hoppers (two per tank) at the effluent end of tanks. Floatable material is pushed by the upper flight of the sludge collector to the influent end of the tanks where it is removed by a vortex skimmer for disposal via the plant sewer. Clarified effluent flows into launders in each tank and then into Channel No. 7. From Channel No. 7, effluent flows through the Secondary Effluent Diversion Structure to the Effluent Filters.

3A.5 Return and Waste Activated Sludge Pump Station

Stage 1 and 2 RAS systems operate in a similar fashion but are completely independent of one another. The return sludge rate required is equal to 80 to 120 percent of the total plant flow as the aeration tanks are being operated in the nitrification/denitrification. RAS collection piping is located in Gallery No. 7. Settled activated sludge is removed from the two hoppers in each secondary clarifier through separate pipes that come together into a common sludge draw-off line for each tank. Each draw-off line is equipped with an in-line magnetic flowmeter with a flow range of 0 to 3 million gallons per day. The RAS flow from each tank is indicated locally and transmitted to the distributed control system (DCS) for summation and display. Each draw-off is also equipped with an electrically actuated butterfly valve, which the DCS modulates to attain the desired draw-off flow.

RAS from FSTs 8 through 13 discharges to the Stage 1 RAS Pump Station wet well. RAS from FSTs 1 through 7 discharges to the Stage 2 RAS Pump Station wet well. The Stage 1 RAS Pump Station is equipped with three vertical, single-stage mixed flow pumps with electric motors, variable frequency controls (VFCs), controls, and instrumentation. The Stage 2 RAS Pump Station includes two horizontal, single-stage mixed flow pumps with electric motors installed in a dry pit, VFCs, controls and instrumentation. The RAS pumps discharge the activated sludge to the aeration tanks. Waste activated sludge (WAS) is removed via 6-inch lines branching from each RAS return pipe and is discharged to the plant sewer. Each WAS line is equipped with a magnetic flowmeter and a flow control valve, which can be controlled with the DCS. The Stage 1 WAS control valve is located in Gallery No. 6 while that for Stage 2 is at the south end of Gallery No. 5.

3A.6 Effluent Filters

The filtration system consists of 10 dual-media rectangular filters which are 32-feet wide by 16-feet in length providing 512 square feet of media surface; a backwash system; an air wash system; an effluent pump station; and a backwash recovery pump station.

Flow from the secondary clarifiers pass through Channel No. 7 and enter the secondary effluent diversion structure. Ferric chloride is injected into the secondary effluent leaving Channel No. 7 to serve as a coagulant prior to filtration. Cationic polymer is also used as a coagulant. It is added

to the mixed liquor flowing from Stage 1 and Stage 2 biological treatment trains, just upstream of the secondary clarifier process. Sodium hypochlorite is added at the Filter Influent Channel to prevent biological growth and to satisfy the requirement for chloramination.

From the filter influent channel, flow is equally distributed to each filter as it passes over individual filter influent weirs, into influent channels, and into each filter through a 24-inch motor-operated butterfly valve. The filtered effluent is transported to filter effluent wet well.

Filters are backwashed regularly to remove the solids that are entrapped during filtration. The filters are first air washed to loosen the media and then washed with water to remove the solids. The second phase of filter backwash removes the solids by flushing water up through the media and into the two-fiberglass reinforced plastic troughs. The waste backwash overflows into the troughs, drains to the upper gullet, through the waste backwash valve, and to the backwash recovery pump station. The Stage 1 PAC supplied air for the air wash sequence of the backwash process.

The plant has a backwash recovery system to collect the waste backwash waste and discharge it to the plant or plant sewer. The waste backwash system, including the backwash troughs and drain valves, transports the waste via gravity to the recovery tank. The backwash recovery pumps then pump the waste backwash water to the plant sewer or to Channel No. 3.

3A.7 Chlorine Contact Tank

The purpose of the chlorine contact tanks (CCTs) is to allow time for chlorine to contact any pathogenic microbial organisms present in the filter effluent, thus disinfecting the effluent before it is discharged. The CCTs are located on the south side of the effluent filters. The filter effluent pump station pumps effluent into the CCT influent channel, where filter effluent is distributed into three CCTs. Effluent from the CCTs enters the effluent channel and overflows into the effluent forebay where the plant effluent flow is measured, chlorine residual is analyzed, and sodium bisulfite and defoamant is dosed prior to discharge to Coyote Creek. Chlorinated effluent is also drawn from the CCTs for reuse by the plant and indirect reuse by the City of Long Beach.

The Sodium Hypochlorite Station consists of two sodium hypochlorite storage tanks, secondary containment for the storage tanks, metering assemblies (two each for pre- and post-chlorination), chlorine residual analyzers, mechanical mixers, controls, and associated piping and equipment. The sodium hypochlorite solution gravity flows from the storage tanks to the metering assemblies and is injected to the filter inlet channel for prechlorination and the filter effluent channel for post-chlorination.

There are four chlorine analyzers measuring at the following locations—prechlorine at the filter influent channel, post chlorine at the CCT inlet channel, out-chlorine at the effluent of CCTs, and final residual chlorine to the effluent forebay.

3A.8 Dechlorination Facilities

The tertiary effluent is dechlorinated prior to the discharge to Coyote Creek. The dechlorination system injects sodium bisulfite into the plant effluent to dechlorinate the effluent. The system consists of sodium bisulfite storage tanks; secondary containment for the storage tanks; metering pumps; magnetic flowmeters; and associated piping, valves, instrumentation, and equipment.

3A.9 Auxiliary and Support Facilities

There are several auxiliary and support facilities including the plant water, potable water, plant sewers and compressed air utilities, chemical feed systems, power distribution and standby power, and instrumentation and control.

3A.9.1 Chemical Feed Systems

3A.9.1.1 Cationic Polymer

The cationic polymer is added to the aeration effluent prior to the FST for better solids settling. The cationic polymer station consists of the two neat polymer storage tanks, and Cationic Polymer System Nos. 1 and 2, which provide dilute polymer for Stage 1 and 2 aeration tanks, respectively. Each polymer system consists of transfer pumps, polymer mixing system, polymer metering systems, and associated piping and controls. The neat polymer pumps pump to the polymer mixing tank. The neat polymer is mixed with water and diluted with the help of the mixers in the tank. Once the diluted polymer is produced, the polymer transfer pumps pump it to the polymer feed tank. The polymer feed pumps then pump the diluted polymer to Channel No. 5 for Stage 1 and 2 aeration tanks.

3A.9.1.2 Ammonia Feed Station

The ammonia feed station is an important component of the disinfection system. Ammonia added to chlorinated plant effluent creates a combined chlorine-ammonia (chloramine) residual which has the advantages of superior bacterial (coliform) disinfection compared to free (uncombined) chlorine and reduction of trihalomethane formation. The Sanitation Districts' water reclamation plants (WRPs) use ammonia with a solution strength of 19.5 percent by weight with a pH of around 13. For comparison, household ammonia has solution strength ranging from 2 to 12 percent with a pH range of 11 to 12.

The ammonia feed station consists of two ammonia storage tanks, secondary containment for the storage tanks, two ammonia feed pumps with associated valves and piping, two flowmeters, two emergency eyewash and safety showers, one ammonia sensor and emergency beacon, one portable ammonia scrubber. The feed system equipment also includes ammonia analyzers located in Gallery No. 7, drawing from Channel No. 7.

3A.9.1.3 Defoamant

The operations and maintenance manual mentions that defoamant is used to control the foam in the tertiary effluent. The system is located at the southeast end of the chlorine contact tanks in the Final Sampling Building. The system consists of two diaphragm defoamant pumps, transferring defoamant from a storage tank into the two defoamant feed lines. There is a mixer inserted into the tank to keep the defoamant solution mixed. Defoamant added to the metering structure controls foam for effluent discharging to Coyote Creek.

3A.9.2 Support Systems

3A.9.2.1 Standby Generator and Other Electrical Systems

Two 2,000-kilowatt Caterpillar 3516C diesel engine generator sets, located at the east side of compressor building, are provided to supply power to operate treatment in the event of commercial power failure. Each standby generator set consists of a diesel engine, a generator, a 5,000-gallon subbase fuel tank, a sound acoustic enclosure, and an electrical control system. Appendix 3B includes the plant one-line diagram.

3A.9.2.2 Instrumentation and Controls

The instrumentation system includes several control panels and many field-mounted meters and controls. Instrumentation provides the necessary information for the operator or the DCS to control processes.

3A.9.2.3 Plant Sewers

LBWRP is part of the Joint Outfall System (JOS). The JOS is a network of seven treatment plants and over 1,300 miles of trunk sewers. The WRPs discharge solids to the sewer for processing at the A.K. Warren Water Resource Facility (Warren Facility). The Warren Facility processes the solids producing electricity and reusable biosolids in the process.

LBWRP has four influent plant sewers, in-plant sewers, and one effluent sludge sewer. Most of the flow comes from the Long Beach Interceptor pumping plant via a 30-inch diameter JOC - 1A interceptor Section 2 force main. The force main discharges into the 54-inch diameter JOC - 7A Replacement gravity sewer at MH C330A along with the 33-inch JOC - 7B sewer (Los Coyotes Diagonal).

3A.9.2.4 Instrument Air Compressors

The instrument air systems provide continuous, dry, clean, compressed air sources for Gallery No. 1 primary sludge draw off pneumatic valves and the filter backwash recovery tank level bubbler. The instrument air system consists of two independent instrument air systems in the plant. One instrument air system is located in Gallery No. 1, and the other is located at the filters. The instrument air system includes air compressors, a desiccant air dryer, an air receiver, piping, and control valves.

3A.9.2.5 Washwater and Non-Potable Water Systems

The washwater and non-potable water systems combine two sources of water, plant effluent withdrawn from the CCTs and potable water from the City of Long Beach. The washwater system, utilizing plant effluent, is the primary source of water used throughout the plant for hose bibs, irrigation, fire hydrants, pump seal water, and for the Bisulfite Station. The washwater pump station consists of two washwater pumps and one fire water pump. The pumps are located at the south end of the chlorine contact tanks where suction to the pumps is drawn from the last pass of the contact tanks. The non-potable water systems consists of one 8-foot cube air gap wet well and two non-potable pumps. In the event of a failure of the washwater pumps, the non-potable system uses potable water supply to maintain the washwater line pressure.

3A.9.2.6 Potable and Fire Water Systems

The potable water system consists of two horizontal centrifugal pumps; a 290-gallon air gap tank; a 400-gallon hydropneumatic tank, an air compressor, piping, and control valves. It provides water to the plant's laboratory, washrooms, lunchrooms, and shower/eyewash stations. The City of Long Beach provides potable water to the plant for treatment purposes and for fire water. The fire water supply originated as a branch off the main potable water supply line located upstream of the potable water pump station and is protected by a backflow preventer. A fire department connection is available next to the fire water supply backflow preventer and is located behind the ferrous chloride chemical station. There are five fire hydrants along the main plant road at the east edge of the property line. There are three additional hydrants on the other side of the road that are supplied by the washwater and non-potable water system.

3A.9.2.7 Influent Chemical System

The influent chemical station consists of a ferric chloride system located within the property boundary of the LBWRP adjacent to the front entrance gate. The Long Beach Sulfide Control Facility continuously injects 34 percent ferrous chloride into the LBWRP sludge gravity sewer at MH LB1, and subsequently into the JO "C".

3A.10 Detailed Design Criteria

Detailed design criteria are included in Attachment 3A.1 - Operations Manual for Long Beach Water Reclamation Plant - Design Criteria.

Attachment 3A.1

OPERATIONS MANUAL FOR LONG BEACH WATER RECLAMATION PLANT - DESIGN CRITERIA

CHAPTER 1 - INTRODUCTION

Table 1.2 Long Beach Water Reclamation Plant, Influent Loadings

Plant Flow	Stage I & II
Average Plant Flow	25 mgd
Minimum Plant Flow (factor = 0.24)	6 mgd
Maximum Peak Sanitary Flow (factor = 1.36)	34 mgd
Maximum Peak Storm Flow (factor = 2.4)	60 mgd
Wastewater Loadings (Average)	
Suspended Solids (SS)	350 mg/L
Suspended Solids (SS)	73,000 lb/day
Chemical Oxygen Demand (COD)	530 mg/L
Chemical Oxygen Demand (COD)	110,500 lb/day
Biochemical Oxygen Demand (BOD ₅)	270 mg/L
Biochemical Oxygen Demand (BOD ₅)	56,300 lb/day

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

INFLUENT PUMP STATION		
Pumps	Type	Centrifugal Non-Clog
	Number	4
	Make	Fairbanks-Morse
	Model	2 at 5420 2 at 5720
	Capacity per Pump	2 at 21,000 gpm 2 at 27,100 gpm
	Head per Pump	2 at 34 ft 2 at 35 ft
Motors	Horsepower	2 at 250 hp 2 at 350 hp
	Voltage	460 V
	Phase	3
	Speed	2 at 1185 rpm 2 at 1188 rpm
	Type of Drive	Variable Frequency Controller
PRIMARY SEDIMENTATION TANKS		
	Number	4
	Length	300 ft
	Width	20 ft
	Average Water Depth	12 ft
	Weir Length/tank	100 ft
	Over Flow Rate (Avg. Flow)	1042 gpd/sf
	Detention Time (Avg. Flow)	2.07 hrs
	Weir Overflow Rate (Avg. Flow)	62,500 gpd/ft
	Flow Through Velocity (Avg. Flow)	2.42 ft/min
	Flight Speed	3 ft/min
	SS Removal (Average)	67 %
	COD Removal (Average)	41 %
	BOD ₅ Removal (Average)	37 %

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)		
Primary Tank Flight Drives	Number	2
	Make & Model	Falk 7KZ4-02As Eurodrive R120-80ZS
	Horsepower	1 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1 at 1,750 rpm 1 at 1,740 rpm
	Type of Drive	Constant Speed
	Gear Reducer Ratio	1 at 986 : 11 at 830 : 1
Primary Tank Skimmer Drives	Number	2
	Horsepower	1/2 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1760 rpm
	Type of Drive	Constant Speed
	Gear Reducer Ratio	1,464 : 1
Primary Sludge Grinders	Number	2
	Make & Model	Muffin Monster 30004T-1208
	Horsepower	3 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1725 rpm
Primary Tank Dewatering Pump	Number	1
	Make & Model	Vaughan Chopper Pump HR7D2
	Capacity	n/a
	Horsepower	25 hp
	Voltage	460 V
	Phase	3
	Speed	1800
AERATION TANK LOADINGS (Average)		
	Suspended Solids (SS)	115 mg/l
	Suspended Solids (SS)	24,000 lb/day
	Chemical Oxygen Demand (COD)	370 mg/l
	Chemical Oxygen Demand (COD)	77,100 lb/day
	Biochemical Oxygen Demand (BOD ₅)	170 mg/l
	Biochemical Oxygen Demand (BOD ₅)	35,400 lb/day
AERATION TANKS		
	Number	8 (4 passes per unit)
	Length	225 ft
	Width	30 ft
	Average Water Depth	15 ft
	Detention Time @ Q + 100% RAS	2.91 hrs

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

Aeration Diffuser Equipment	Type	Fine Bubble	
	Make	Sanitaire Diffusers	
	Type of diffuser	9" Ceramic Domes	
	Orifice Size	13/64 in	
	Grids per Aeration Unit	12	
Aeration Tank Drain Pump	Number	1	
	Type	Self-Priming, Horizontal Non-Clog	
	Make	Gorman-Rupp	
	Model	T6A3S-B	
	Capacity	2500 gpm	
	Head	21 ft	
	Horsepower	30 hp	
	Voltage	460 V	
	Phase	3	
	Speed	1755 rpm	
Type of Drive	Constant Speed		
<u>PROCESS AIR COMPRESSORS</u>			
	Type	Centrifugal	
	Number	4	
	Make & Model	2 at Dresser 01B 2 at HV KA22S	
	Discharge Pressure	21.6 psig	
	Capacity per Compressor	2 at 22,500 scfm 2 at 10,000 scfm	
	Horsepower per Compressor	2 at 800 hp 2 at 500 hp	
	Voltage	4160 V	
	Phase	3	
	Speed	1780 rpm	
	Type of Drive	Constant Speed	
	Speed-Increasing Gear Drive Ratio	3.35 : 1	
	<u>SECONDARY CLARIFIERS</u>		
		Number	13
Length		150 ft	
Width		20 ft	
Normal Depth		10 ft	
Weir Length per Tank		200 ft	
Over Flow Rate (Average Flow)		694 gpd/sf	
Detention Time @ Q + 100% RAS (Average Flow)		1.29 hrs	
Weir Overflow Rate (Average Flow)		10,420 gpd/ft	
Flow Thru Velocity (Average Flow)		0.97 ft/min	
Flight Speed		3 ft/min	

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)		
Secondary Clarifier Flight Drives	Number	7
	Make & Model	3 Falk 3KRZ5-20AS 1 Foote Jones 743Y-184 3 Eurodrive K100K70ZS
	Horsepower	1 hp
	Voltage	460 V
	Phase	3
	Speed	1 at 1740 rpm 1 at 1755 rpm 5 at 1750 rpm
	Gear Reducing Ratio	3 at 968.7 : 1 1 at 875.8 : 1 3 at 854 : 1
	Type of Drive	Enclosed Gear
Secondary Clarifier Drain Pump	Number	1
	Make & Model	Gorman-Rupp T6A3S-B
	Capacity	600 gpm
	Head	11 ft
	Horsepower	5 hp
	Voltage	230/460 V
	Phase	3
	Speed	1755
	Type of Drive	Constant
<u>RETURN SLUDGE PUMP STATION</u>		
	Number of Pump Stations	2
	Number of Pumps per Station	3 for RAS PS No. 1 2 for RAS PS No. 2
	Type of Pumps	3 Vertical Wet Pit 2 Horizontal Dry Pit
	Make & Model	3 Fairbanks Morse 8312AWF 2 Worthington 16MN19
	Capacity per Pump	3 at 5000 gpm 2 at 5900 gpm
	Head per Pump	2 at 22 ft 2 at 18.7 ft
	Horsepower per Pump	60 hp 50 hp
	Voltage	460V
	Phase	3
	Speed	3 at 1190 rpm 2 at 585 rpm
	Type of Drive	Variable Frequency Controller
<u>EFFLUENT FILTERS</u>		
	Type	Gravity
	Media	Anthracite / Sand
	Configuration	Dual - Deep Bed

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

	Number	10
	Length	32 ft
	Width	16 ft
	Depth	23 ft
	Support Gravel	55.75 in
	Inert Media	36 in
	Surface Area per Filter	512 sf
	Filtration Rate (Avg. flow)	3.4 gpm/sf
	Backwash Rate (maximum)	20 gpm/sf
	Surface Wash (fixed Nozzle)	0.7 gpm/ft
	Backwash Cycle	0.5 hrs
	Air Scour Rate (maximum)	5 scfm/ft
<u>FILTER EFFLUENT PUMPS</u>		
	Type	Vertical Wet Pit
	Number	4
	Make & Model	Aurora 18LM
	Capacity per Pump	2 at 7500 gpm 2 at 8650 gpm
	Head per Pump	2 at 21.5 ft 1 at 25.1 ft 1 at 25.5 ft
	Horsepower per Pump	2 at 60 hp 2 at 75 hp
	Voltage	460 V
	Phase	3
	Speed	2 at 890 rpm 2 at 885 rpm
	Type of Drive	3 Mag Drive 1 Constant Speed
<u>FILTER BACKWASH PUMPS</u>		
	Type	Vertical Wet Pit
	Number	2
	Make & Model	Aurora 18LM
	Capacity per Pump	10,000 gpm
	Head per Pump	27 ft
	Horsepower	100 hp
	Voltage	460 V
	Phase	3
	Speed	880 rpm
	Type of Drive	Constant Speed
<u>FILTER WASTE BACKWASH RECOVERY WELL</u>		
	Number	1
	Effective Volume	224,000 gallons

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

FILTER WASTE BACKWASH PUMPS

Type	Vertical Turbine
Number	2
Make & Model	Aurora 14RH
Capacity per Pump	1900 gpm
Head per Pump	36 ft
Horsepower	30 hp
Voltage	460 V
Phase	3
Speed	1175 rpm
Type of Drive	Constant Speed

FILTER EFFLUENT WETWELL

Capacity (usable)	40,212 gallons
Capacity (total)	47,750 gallons
Length	32 ft
Width	21 ft
Height	9.5 ft

PRE-CHLORINATION MIXER

Mixer I	Impeller Type	Axial Flow
	Number	1
	Make & Model	Philadelphia Mixer 3859MQ
	Horsepower	40 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1755 rpm
Mixer II (Spare)	Impeller Type	Axial Flow
	Number	1
	Make & Model	Chemineer 3HTD-7.5
	Impeller Diameter	40 in
	Shaft Speed	84 rpm
	Horsepower	7.5 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1755 rpm

POST CHLORINATION MIXER

	Impeller Type	Axial Flow
	Number	1
	Make & Model	Chemineer 3HTD-7.5
	Impeller Diameter	40 in
	Shaft Speed	84 rpm
	Horsepower	7.5 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1755 rpm

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

CHLORINE CONTACT TANKS		
	Number	3
	Length (each pass)	287.5 ft
	Width	20 ft
	Average Water Depth	20 ft
	Detention Time	2.5 hrs
	Volume per tank	862,000 gallons
CHEMICAL EQUIPMENT		
Ammonia System		
Ammonia Storage Tank	Number	2
	Capacity (usable)	4,150 gallons
	Capacity (total)	4,875 gallons
	Dimensions	8 ft dia. x 10 ft ht
Ammonia Feed Pumps	Number	2
	Type	Positive Displacement Peristaltic
	Make & Model	Watson Marlow 624U
	Capacity per pump	0.6 gpm
	Head per pump	30 ft
	Horsepower	0.1 hp
	Voltage	115 V
	Phase	1
	Speed	165 rpm
	Type of Drive	Electromagnetic
Ammonia Mixer	Number	1
	Impeller Type	Radial Flow
	Make & Model	Philadelphia Mixer 3855MQ
	Horsepower	15 hp
	Voltage	460 V
	Phase	3
	Speed	1800 rpm
Ferric Chloride System		
Ferric Chloride Tank	Number	55 gallon barrel
Ferric Chloride Feed Pumps	Number	1
	Make & Model	Stenner Pump 85MHP5
	Capacity	5 gpd
	Rated Pressure	400 psi
	Voltage	120 V
	Phase	1
	Speed	1800 rpm
Alum System (Backup)		
Alum Storage Tank	Number	1
	Capacity (usable)	6,200 gallons
	Capacity (total)	6,500 gallons
	Dimensions	10 ft \varnothing x 11 ft ht.
Liquid Alum Feed Pumps	Number	3
	Type	Diaphragm

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

	Make & Model	BIF 1731-62-4819
	Capacity per Pump	128 gph
	Head per Pump	150 psig
	Horsepower	3/4 hp
	Voltage	50 / 100 V
	Speed	1725 rpm
	Type of Drive	Constant Speed
	Gear Reducing Ratio	9:1
Cationic Polymer System No. 1		
Neat Polymer Storage Tank	Number	1
	Capacity (usable)	5,000 gallons
	Capacity (total)	5,288 gallons
	Dimensions	10 ft ø x 9 ft ht.
Neat Polymer Transfer Pump No. 1	Number	1
	Type	Rotary Lobe
	Make	Viking
	Model	KK 724
	Capacity	8 gpm
	Head	n/a
	Horsepower	3 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1735 rpm
Type of Drive	Constant Speed	
Gear Reducing Ratio	6.4:1	
Common Spare Neat/Transfer Pump	Number	1
	Type	Rotary Lobe
	Make & Model	Viking Pump KK724
	Capacity	70 gpm
	Head	n/a
	Horsepower	3 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1140 rpm
	Type of Drive	Constant Speed
Gear Reducing Ratio	3.43 : 1	
Polymer Mixing Tank	Number	1
	Capacity (usable)	850 gallons
	Capacity (total)	950 gallons
	Dimensions	6 ft ø x 4.5 ft ht.
Polymer Mixer	Number	1
	Make & Model	Leeson
	Horsepower	½ hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1725 rpm
	Impeller Type	Blade Propeller

CHAPTER 1 - INTRODUCTION

Dilute Polymer Transfer Pump	Number	1
	Type	Rotary Lobe
	Make	MID
	Model	215-L6S1F1XAANS
	Capacity	60 gpm
	Head Per Pump	n/a
	Horsepower	3 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1730 rpm
	Type of Drive	Constant Speed
	Gear Reducing Ratio	3.669 : 1
	Polymer Metering Tank	Number
Capacity (usable)		850 gallons
Capacity (total)		1200 gallons
Dimensions		6 ft ø x 4.5 ft ht.
Polymer Metering Pumps (Channel No. 5)	Number	3
	Type	Rotary Lobe
	Make	Viking
	Model	HL 4724
	Capacity per Pump	2 at 7 gpm 1 at 9 gpm
	Head per Pump	125 psi
	Horsepower	2 at 2 hp 1 at 3 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1 at 1750 rpm 1 at 1745 rpm 1 at 3450 rpm
	Type of Drive	Variable Speed
Polymer Metering Pumps (Waste Backwash)	Number	2
	Type	Diaphragm
	Make	BIF
	Model	1 at 1732-62-9810 1 at 1731-42-9810
	Capacity Per Pump	1 at 128 gph 1 at 220 gph
	Head Per Pump	150 psig
	Horsepower	3/4 hp
	Voltage	115 / 230 V
	Phase	1
	Speed	1725 rpm
	Type of Drive	Constant Speed

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

Cationic Polymer System No. 2		
Neat Polymer Storage Tank	Number	1
	Capacity (usable)	5,900 gallons
	Capacity (total)	6,169 gallons
	Dimensions	9.5 ft ø x 10.5 ft ht.
Neat Polymer Transfer Pump	Number	1
	Type	Gear
	Make & Model	Viking AK4197
	Capacity	15 gpm
	Head	40 psig
	Horsepower	2 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1,140 rpm
	Gear Reducer Ratio	3.43 : 1
Type of Drive	Constant Speed	
Polymer Mixing Tank	Number	1
	Capacity (usable)	1,100 gallons
	Capacity (total)	1,200 gallons
	Dimensions	6 ft ø x 5.5 ft ht.
Polymer Mixer	Number	1
	Make & Model	Chemineer
	Shaft Speed	126 rpm
	Horsepower Per Mixer	2 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1725 rpm
Impeller Type	Hydrofoil	
Dilute Polymer Transfer Pumps	Number	2
	Type	Gear
	Make & Model	Viking KK4724
	Capacity per Pump	40 gpm
	Head per Pump	20 psig
	Horsepower	3 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1 at 1725 rpm 1 at 1750 rpm
	Gear Reducing Ratio	4.19 : 1
Type of Drive	Constant Speed	
Polymer Metering Tank	Number	1
	Capacity (usable)	1,100 gallons
	Capacity (total)	1,200 gallons
	Dimensions	6 ft ø x 5.5 ft ht.
Polymer Metering Pumps	Number	2
	Type	Gear
	Make & Model	Viking HL4724
	Capacity per Pump	0.4 to 4.0 gpm

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)		
	Head per Pump	25 psi
	Horsepower	1 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1160 rpm
	Gear Reducing Ratio	2.76 : 1
	Type of Drive	Variable Speed
Defoamant Chemical System		
Defoamant Chemical Solution Tank	Number	1
	Capacity (usable)	155 gallons
	Capacity (total)	165 gallons
	Dimensions	30 in ø x 50 in H.
Defoamant Chemical Solution Feed Pump	Number	2
	Type	Diaphragm
	Make	Milton Roy MLI
	Model	8731-75HU
	Capacity	108 gpd
	Head	50 psig
	Horsepower	1/4 hp
	Voltage	115 / 230 V
	Phase	1
	Speed	1725 rpm
	Type of Drive	Variable Speed
Defoamant Chemical Solution Mixer	Number	1
	Make	Lightin
	Model	EV5P33
	Horsepower	1/3 hp
	Voltage	208 / 230
	Phase	3
	Speed	1725 rpm
Defoamant Chemical Solution Transfer Pump	Number	1
	Type	Centrifugal
	Make & Model	TEEL 2P390 TJ
	Horsepower	1/2 hp
	Voltage	115 V
	Phase	1
	Speed	3450 rpm
Sodium Hypochlorite System		
Sodium Hypochlorite Storage Tank	Number	2
	Capacity (usable)	18,500 gallons
	Capacity (total)	19,827 gallons
	Dimensions	15 ft ø x 15 ft ht.
Transfer Pump (can also be used for Sodium Bisulfite)	Number	1
	Type	Peristaltic
	Make & Model	Watson Marlow GRD2010-50-B3

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)		
	Capacity	39 gpm
	Head	15 ft
	Horsepower	3 hp
	Voltage	115 V
	Phase	1
	Speed	60 rpm
	Type of Drive	Electromagnetic
Sodium Hypochlorite Mixer	Number	1
	Impeller Type	Radial Flow
	Make & Model	Philadelphia Mixer 3859MQ
	Horsepower	40 hp
	Voltage	460 V
	Phase	3
	Speed	1800 rpm
Sodium Bisulfite System (Lead CC-SBS Chemical Feed System)		
Sodium Bisulfite Storage Tank	Number	2
	Capacity (usable)	9,700 gallons
	Capacity (total)	10,260 gallons
	Dimensions	12 ft ø x 12.5 ft ht.
Sodium Bisulfite Feed Pump	Number	3
	Type	Peristaltic
	Make & Model	Watson Marlow C20U
	Capacity per Pump	3 gpm
	Horsepower	0.1 hp
	Voltage	115 V
	Phase	1
	Speed	165 rpm
Standby Sodium Bisulfite System		
Sodium Bisulfite Storage Tank	Number	1
	Capacity	1,000 gallons
Pneumatically operated discharge valves	Number	3
	Instrument air compressor (to provide control)	1
	Chlorine residual analyzer (effluent sampling building)	1
	DeOx residual analyzer (effluent sampling building)	1
Ferrous Chloride System (Not in use)		
Ferrous Chloride Storage Tank	Number	1
	Capacity (usable)	10,150 gallons
	Capacity (total)	10,575 gallons
	Dimensions	12 ft ø x 12.5 ft ht.
NON-POTABLE WATER PUMPS		
	Type	Vertical Turbine
	Number	2
	Make & Model	Peerless 6x6x12

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

	Capacity per Pump	160 gpm
	Head per Pump	230 ft
	Horsepower	15 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	3510 rpm
	Type of Drive	Constant Speed
<u>WASHWATER PUMPS</u>		
	Type	Horizontal Centrifugal
	Number	2
	Make & Model	Griswold A40
	Capacity per Pump	475 gpm
	Head per Pump	230 ft
	Horsepower	75 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	3550 rpm
	Type of Drive	Constant Speed
<u>FIREWATER PUMP</u>		
	Type	Horizontal Centrifugal
	Number	1
	Make & Model	Aurora 411-BF
	Capacity	2,400 gpm
	Head	124 ft
	Horsepower	100 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1780 rpm
	Type of Drive	Constant Speed
<u>FOAM SPRAY PUMPS</u>		
	Type	Vertical Turbine
	Number	2
	Make & Model	1 Peerless 6x6x12 1 Worthington 10M41-1
	Capacity per Pump	370 gpm
	Head per Pump	42 ft
	Horsepower	7.5 hp
	Voltage	460 V
	Phase	3
	Speed	1 at 3,450 rpm 1 at 3,500 rpm
	Type of Drive	Constant Speed
<u>INSTRUMENT AIR COMPRESSORS (Gallery 1)</u>		
	Type	Reciprocating 2-Stage

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

	Number	2
	Make	Champion Pneumatic
	Model	HR7D2
	Capacity	33.5 cfm
	Discharge Pressure	130 psi
	Horsepower	7.5 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1760 rpm
	Type of Drive	Constant Speed
Refrigerant Dryer	Make	Ingersoll Rand
	Model	DXR25
	Pressure	250 psig
Air Receiver	Number	1
	Capacity	266 gallons
	Dimensions	8 ft ø x 80 in. L
<u>INSTRUMENT AIR COMPRESSORS (Filters)</u>		
	Type	Single Stage
	Number	2
	Make & Model	Ingersoll Rand
	Capacity	n/a
	Discharge Pressure	40 psi
	Horsepower	1/3 hp
	Voltage	115 V
	Phase	1
	Speed	1,725 rpm
	Type of Drive	Constant Speed
Air Receiver	Capacity	30 gallons
	Dimensions	16 in ø x 38 in. L
<u>POTABLE WATER STATION</u>		
Air Gap Tank	Number	1
	Capacity (usable)	235 gallons
	Capacity (total)	290 gallons
	Dimensions	3 ft ø x 5.5 ft ht.
Pumps	Number	2
	Type	Horizontal Centrifugal
	Make & Model	Peerless 1.5x1.6 Goulds 1x1.5-6
	Capacity per Pump	70 gpm
	Head per Pump	100 ft
	Horsepower	5 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1 at 3,500 rpm 1 at 3510 rpm
	Type of drive	Constant Speed

CHAPTER 1 - INTRODUCTION

Table 1.3 Long Beach Water Reclamation Plant Design Criteria (Stage I & Stage II)

Hydropneumatic Tank	Number	1
	Capacity (usable)	400 gallons
	Capacity (total)	449 gallons
	Dimensions	3 ft ø x 7.5 ft L
	Orientation	Horizontal
CITY OF LONG BEACH REUSE PUMP STATION		
	Number	5
	Type	Vertical turbine
	Make & Model	Floway Pumps 14EKH
	Capacity per Pump	2500 gpm
	Head per Pump	189 ft
	Horsepower	150 hp
	Voltage	230 / 460 V
	Phase	3
	Speed	1780 rpm
	Type of drive	Variable Speed Controller
STANDBY POWER SUPPLY		
	Number	1
	Type	Gas Turbine
	Make & Model	Solar GSE-3000
	Capacity	800 KW
	Fuel Type	Diesel
	Fuel Consumption Rate	126 gph
	Fuel Tank Capacity	10,000 gallons

Table 1.4 Long Beach WRP Capital Investments

Project	Contract No.	Date Accepted	Contract Amount
Long Beach Water Renovation Plant Site Purchase Agreement	1816	08/28/68	
Long Beach Water Renovation Plant Site Development	1907	08/26/70	\$ 263,194
Long Beach Water Renovation Plant Architectural Services	1916		\$ -
Storm Water Basin	1932		\$ -
Long Beach Water Renovation Plant Stage One (12.5 mgd)	1953	07/11/73	\$ 3,828,000
Long Beach WRP & JWPCP Arch Services	2014		\$ -
Power Service Ducts	2033	04/26/72	\$ 15,430
Long Beach WRP- Tertiary Treatment Study	2102		\$ -
Landscaping for Long Beach Water Renovation Plant (<i>dechlorination part of filter project</i>)	2115	04/24/74	\$ 159,801
Long Beach WRP Filters	2188		\$ -
Long Beach Water Reclamation Plant Effluent Filters	2213	05/10/78	\$ 2,340,048
Long Beach WRP Interceptor Sewer PP	2381	08/26/81	\$ 1,336,005
Long Beach Water Reclamation Plant Stage Two (12.5 mgd)	2471	05/23/84	\$ 12,138,866
Long Beach Water Reclamation Plant Fire Water Line	2716	01/23/85	\$ 56,721
Installation of Leak Detection & Monitoring Systems for Underground Storage Tanks	2901	03/08/89	\$ 177,400

CHAPTER 1 - INTRODUCTION

Project	Contract No.	Date Accepted	Contract Amount
Concrete & Protective Coating Repair	3200	12/09/92	\$ 625,221
Concrete & Protective Coating Repair - Phase II	3361	12/13/95	\$ 2,039,854
Roof Repairs at Long Beach, Whittier Narrows, San Jose Creek - East, Pomona and Los Coyotes WRPs	3453	10/22/97	\$ 194,418
Long Beach Water Reclamation Plant Installation of Sludge Collection Equipment for Final Sedimentation Tanks	PO#17029	11/13/96	\$ 280,000
Stage One Aeration System Modifications	3483	03/11/98	\$ 1,434,299
Sodium Hypochlorite and Bisulfite Facilities	3511	07/26/00	\$ 671,579
Pavement Repairs - 2000	3761	04/11/01	\$ 108,064
Skimmer Modifications of Primary Tanks at Various Water Reclamation Plants	3848	01/28/04	\$ 1,470,846
Stage Two Aeration System Replacement	3913	03/12/03	\$ 303,616
Ammonia Addition Station	3927	11/26/03	\$ 706,444
Interceptor, Section 1, Sludge Force Main Replacement	4021	01/12/05	\$ 1,432,081
Long Beach WRP Nitrification/Denitrification Facility	4185	08/13/08	\$ 5,455,596
Emergency Generators and Influent Pumps Replacement	4467	12/18/12	\$ 3,448,880
Plant Electrical System Modifications	4521		\$ 1,894,500
Abandonment of Underground Diesel Fuel Storage Tank	4609	8/10/11	\$ 33,904
Interceptor and Bloomfield PPs Installation of Pumps	4689	01/09/13	\$ 118,700

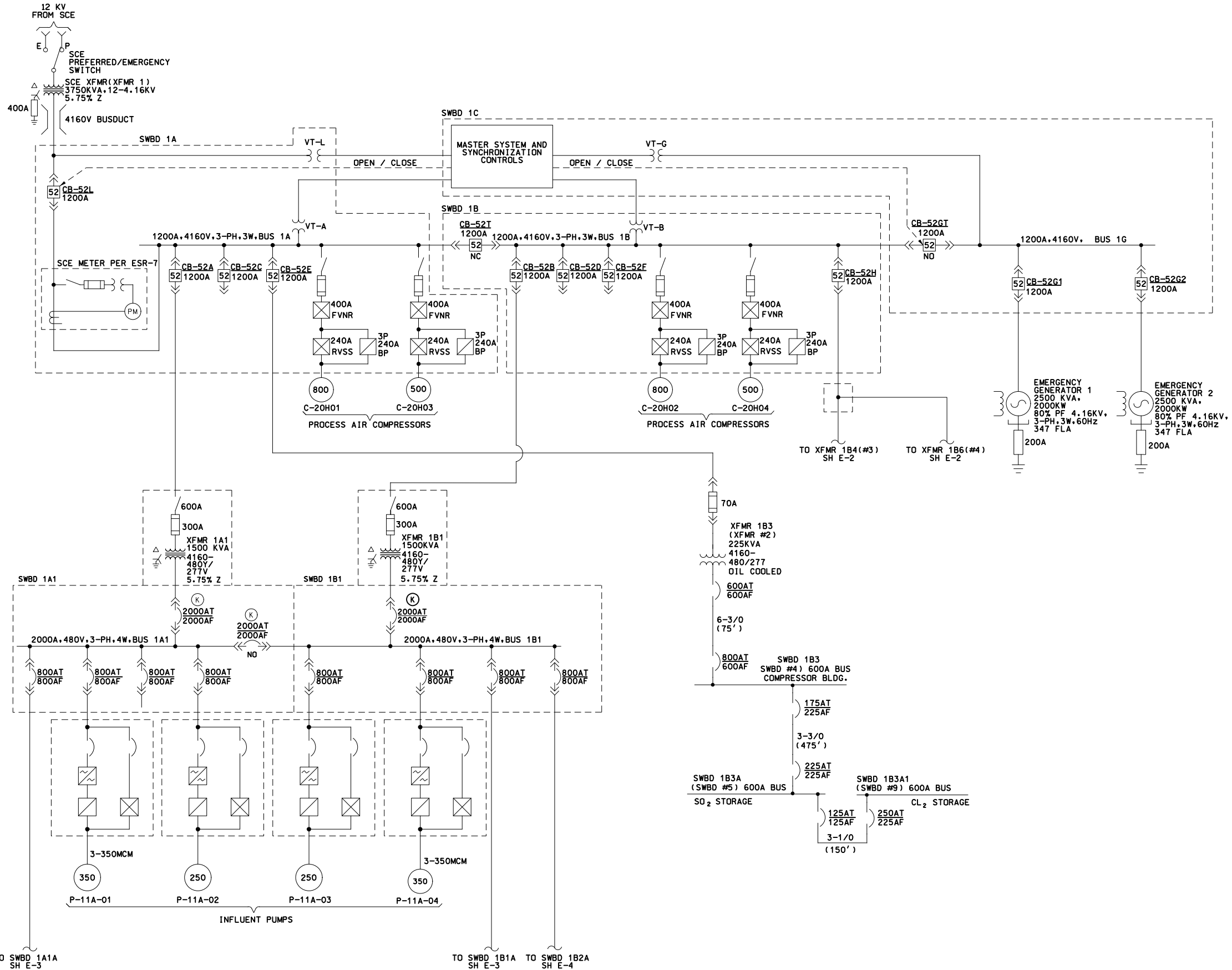
1.4 Plant Rounds

Normal operation of this waste treatment facility consists of the procedures and conditions outlined in the subsequent sections of this manual. The most important routine task that the Operator must perform is making Plant Rounds.

During these rounds, the Operator checks all plant operations for any abnormal conditions. All equipment is checked for proper operation. Figure 1-7 shows an example Plant Round Check List that is provided to facilitate the operator in conducting Plant Rounds. Individual equipment plant round sheets will be included in the relevant chapters. These status report sheets are to be filled out as the Operator makes his/her Plant Rounds. Seven day of plant status can be recorded on each form. The completed sheets are to be kept on file at the Operators desk in the Control Room.

Appendix 3B

ONE-LINE DIAGRAM



FOR INTERNAL USE ONLY

COUNTY SANITATION DISTRICT NO. 2 OF LOS ANGELES COUNTY, CALIFORNIA

DESIGNED	K. JITPATIMA	DATE	APRIL, 2011
DRAWN	C. CUEVAS	DATE	APRIL, 2011
CHECKED	K. JITPATIMA	DATE	APRIL, 2011
REVIEWED		DATE	

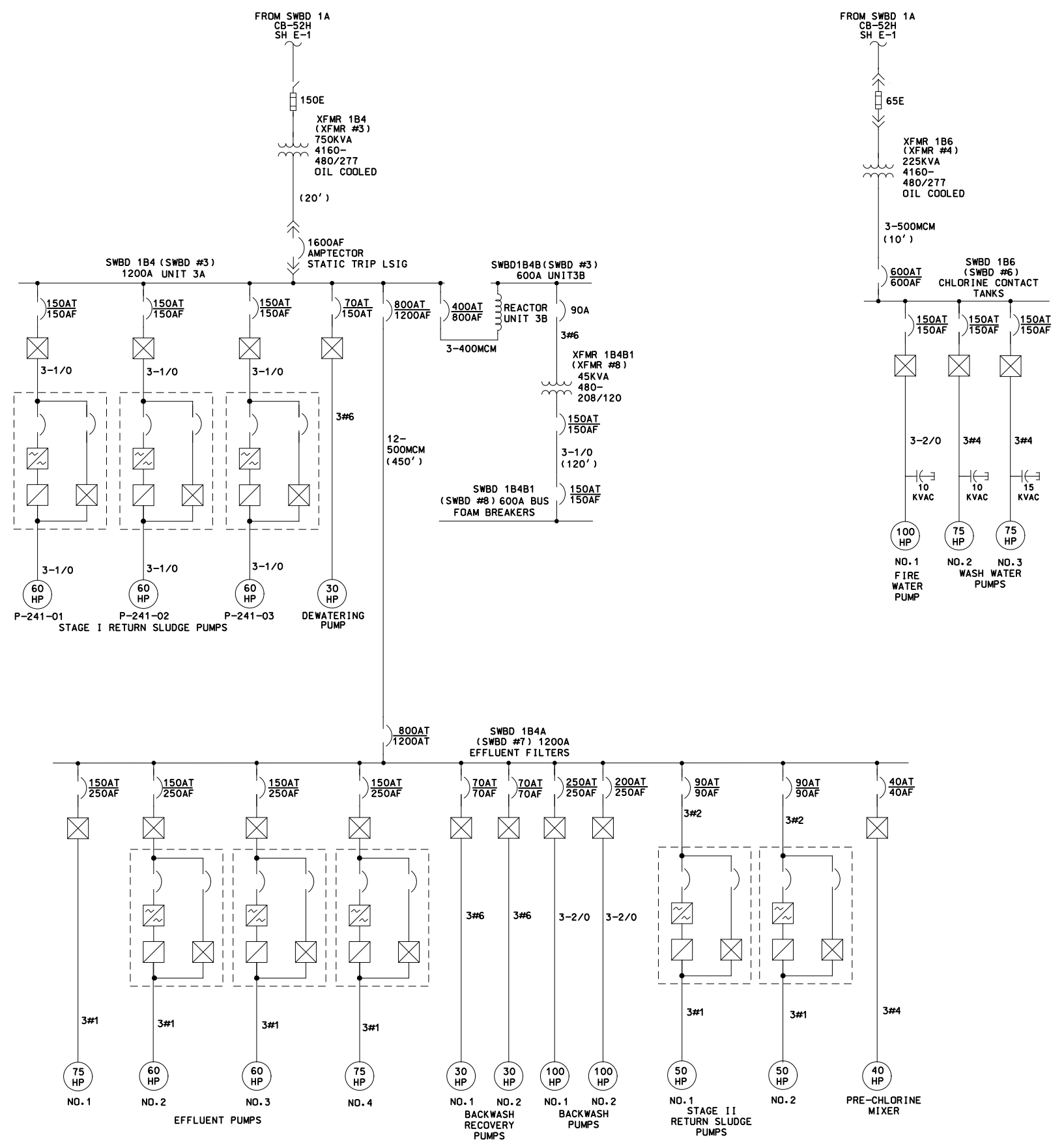
LONG BEACH WATER RECLAMATION PLANT

ELECTRICAL ONE-LINE DIAGRAM I

SCALE:	NONE
SHEET NO.	E-1
DWG. NO.	

WARNING $\leftarrow \frac{1}{2} \rightarrow$ IF THIS LINE IS NOT 1" LONG, THEN DWG IS NOT TO SCALE.

REVISIONS: H, G, T, FT, C, B, A
 INITIALS: H, G, T, FT, C, B, A
 DATE: H, G, T, FT, C, B, A
 R:\Operations\OTHER\ONE LINE DIAGRAM\LONG BEACH (INTERNAL USE)\E01.dwg
 LACS 12.08.04



FOR INTERNAL USE ONLY

COUNTY SANITATION DISTRICT NO. 2
OF LOS ANGELES COUNTY, CALIFORNIA

DESIGNED	K. JITPATIMA	DATE	APRIL, 2011
DRAWN	C. CUEVAS	DATE	APRIL, 2011
CHECKED	K. JITPATIMA	DATE	APRIL, 2011
REVIEWED		DATE	

LONG BEACH WATER RECLAMATION PLANT

ELECTRICAL
ONE-LINE DIAGRAM II

SCALE:	NONE
SHEET NO.	E-2
DWG. NO.	

WARNING $\leftarrow \frac{1}{2} \rightarrow$ IF THIS LINE IS NOT 1" LONG, THEN DWG IS NOT TO SCALE.

29-AUG-2014 10:08 ccuevas

1 2 3 4 5 6 7 8 9 10 11 12

R:\Operations\OTHER\ONE LINE DIAGRAM\LONG BEACH (INTERNAL USE)\E02.dgn LACS0 12/08/04

Appendix 3C

FILTER DESIGN CRITERIA

Hem Vora

From: Lydia Holmes
Sent: Monday, October 24, 2022 1:54 PM
To: Hem Vora; Andre Gharagozian; Rajesh Doppalapudi
Subject: FW: Sanitation Districts' General Design Criteria for Effluent Filters

From: Lanza, Jodie <JLanza@lacsds.org>
Sent: Monday, October 24, 2022 1:50 PM
To: Lydia Holmes <LHolmes@carollo.com>
Cc: Sayre, Jaime <jaimesayre@lacsds.org>
Subject: Sanitation Districts' General Design Criteria for Effluent Filters

CAUTION: This email originated from outside Carollo Engineers. Do not open attachments or click links unless you recognize the sender.

Lydia, one of our outstanding action items was to come up with general design criteria for effluent filters at our WRPS. After internal discussions, we have come up with the criteria below. Please apply this to your work on SJC.

Rationale

The purpose of this email is to establish general design criteria for effluent filters at the Districts' water reclamation plants. One of the Districts' Guiding Principles is to maximize use of our assets and resources, including recycled water. Although Title 22 requirements must be met for reuse of recycled water, some allowance can be made for non-compliance during extreme storm events when demand for recycled water is low. The Districts intend to make recycled water available for reusers at all times; however it is impractical, not cost-effective, and wasteful of available space at the treatment plants to design facilities to meet Title 22 requirements for rare, extreme storm events. As such, the design criteria below is intended to size filters to permit reuse during most, but not all, wet weather events over the 30-year planning horizon.

This design criteria is a starting point, and the Engineer must apply sound engineering judgement that balances conservatism and projections vs. actual flow data to arrive at an appropriate design flow. The calculations are primarily based on historic peak wet weather filter effluent flows, modest population-based sanitary flow projections, and the Title 22 prescriptive filter loading rate of 5 gpm/sf (see Title 22 §60301.320 Filtered Wastewater). Per input from WRP Operations, it is assumed that all filters are in operation during a peak wet weather event, and for the design check at average daily design flow it is assumed one filter is out of service and one filter is in backwash (N+2). Also, for plants with primary effluent flow equalization, it is assumed the flow equalization system is in-service during the peak wet weather event, which provides some dampening of the peak flows.

At many treatment plants throughout California, the SWRCB Division of Drinking Water (DDW) has allowed Title 22 compliance at a higher filter loading rate of 7.5 gpm/sf, subject to demonstration testing. Although not considered in the below design criteria, it is important to note that a rerate of the filters to this higher loading rate may be possible in the future to accommodate increases in flow beyond the current projections.

Criteria

1. Determine Historical Peak Wet Weather Filter Effluent Flow.
 - For previous 10-15 years, use OSI-Pi to find peak hourly filter effluent flow rate for each calendar year, which typically occurs during wet weather events (consider all days, rain or no rain). For plants with primary effluent flow equalization, assume the equalization facilities are in-service during the peak wet weather event – using filter effluent flow data accounts for the effects of equalization on peak tertiary flows. Review the data and remove any clear outliers.

- Select the 50th percentile of these peak annual flows, which correlates to a wet weather flow that has been exceeded ~5-7 times in the previous 10-15 years (or once every other year). Do not select the ultimate peak wet weather flow from the data.
2. Determine incremental average daily flow (ADF) increase between current and projected ADF for 30-year planning period.
 - Conduct an Area Study to determine 30-year population-based flow projection (per PCGR and SCAG growth rate); or obtain 30-year treatment plant flow projections from the Wastewater Planning Section.
 - Calculate the incremental flow increase between the current ADF and 30-year projected ADF (i.e., 30-year flow projection minus current ADF).
 3. Find Effluent Filters Peak Wet Weather Design Flow (MGD) = [50th Percentile Historic Peak Wet Weather Filter Effluent Flow] + [Incremental ADF Increase per 30-year Projected Flow]. Round to the nearest 0.5 MGD.
 4. Determine required surface area (square feet, SF) of filters at Peak Wet Weather Design Flow using the Title 22 max filter loading rate = 5.0 GPM/SF and assuming all filters in service (i.e., no filters in backwash or on standby).
 5. Based on site constraints and typical filter sizes at other WRPs, assume a length x width for each individual filter with all filters in service to determine the number of filters needed at Peak Wet Weather Design Flow.
 - Iterate this process until optimal filter layout is achieved.
 - As a check, calculate filter loading rate at 30-year projected average daily flow for N+2 filters in service (i.e., 1 filter out-of-service and 1 filter in backwash). Loading rate should be approximately 2.5 – 3.5 GPM/SF (typical design range).

Test Cases

To evaluate the effectiveness of this design approach, the criteria was tested at two WRPs: Valencia WRP (primary effluent flow equalization), and Long Beach WRP (no flow equalization). Here are the results:

Valencia WRP

1. 50th Percentile Historic Peak Wet Weather Filter Effluent Flow = 35.2 MGD
2. Incremental Flow Increase = 22.0 MGD (30-year projected ADF) – 14.5 MGD (current ADF) = 7.5 MGD
3. Effluent Filters Peak Wet Weather Design Flow = 35.2 MGD + 7.5 MGD = 42.7 MGD → Use 42.5 MGD (note: current WW design flow = 48.6 MGD)
4. Total required surface area = 42.5 MGD x 694 gpm/MGD / 5.0 gpm/SF = 5,900 SF
5. Try 34' x 18' filter size (approx. size of filters at SJCW) = 612 SF → 5,900 SF / 612 SF/filter = 9.6 filters → 10 total filters needed (can adjust LxW to optimize)
 - Check at 30-year projected ADF for N+2 filters in service (N = 8) = 22.0 MGD x 694 gpm/MGD / (8 filters x 612 SF/filter) = 3.1 gpm/sf → OK

Long Beach WRP

1. 50th Percentile Historic Peak Wet Weather Filter Effluent Flow = 45.3 MGD
2. Incremental Flow Increase = 15.3 MGD (30-year projected ADF) – 12.4 MGD (current ADF) = 2.9 MGD
3. Effluent Filters Peak Wet Weather Design Flow = 45.3 MGD + 2.9 MGD = 48.2 MGD → Use 48.0 MGD (note: current WW design flow = 60 MGD)
4. Total required surface area = 48.0 MGD x 694 gpm/MGD / 5.0 gpm/SF = 6,660 SF
5. Try 32' x 16' filter size (approx. size of ex filters at LB) = 512 SF → 6,660 SF / 512 SF/filter = 13.0 filters → 13 total filters needed
 - Check at 30-year projected ADF for N+2 filters in service (N = 11) = 15.3 MGD x 694 gpm/MGD / (11 filters x 512 SF/filter) = 1.9 gpm/sf → Low, but OK

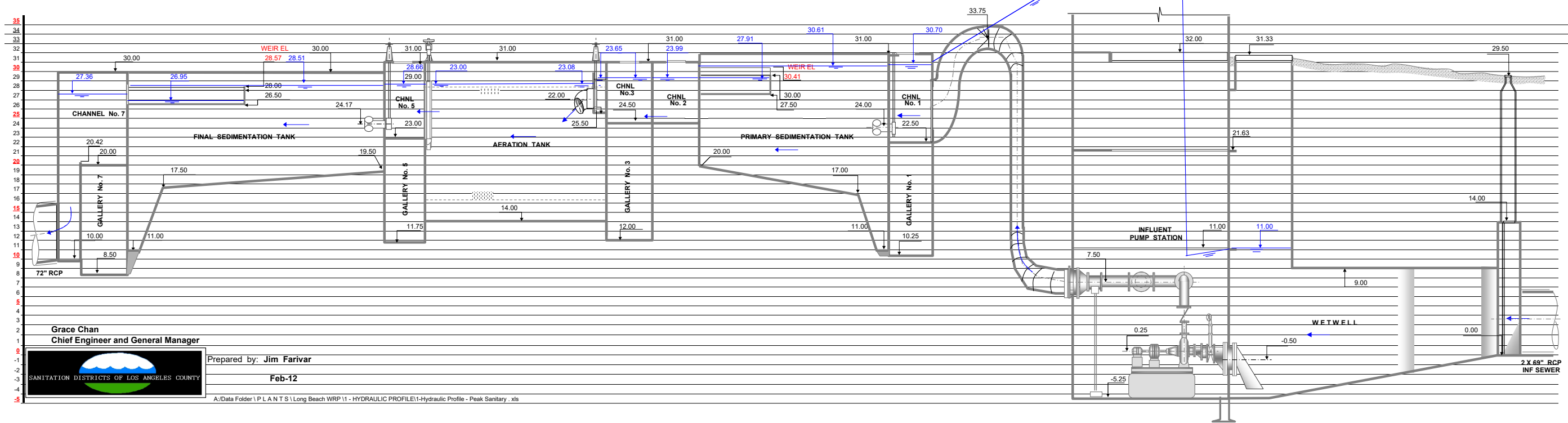
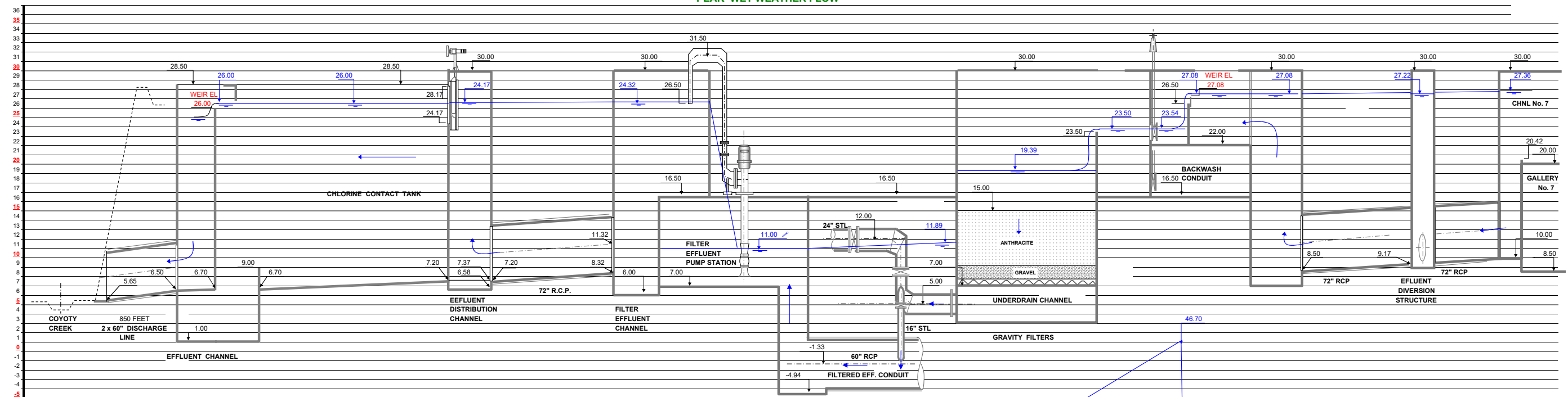
Note: LBWRP currently has 10 effluent filters, and does not meet Title 22 filter loading rate at peak wet weather flow.

Appendix 3D

HYDRAULIC CAPACITY SUMMARY

LONG BEACH WATER RECLAMATION PLANT

Profile Flow = 38.16 MGD
PEAK WET WEATHER FLOW



Grace Chan
Chief Engineer and General Manager

Prepared by: Jim Farivar
Feb-12

A:\Data Folder\PLAN\T\S\Long Beach WRP\11 - HYDRAULIC PROFILE\1-Hydraulic Profile - Peak Sanitary .xls

Appendix 3E
PERFORMANCE ASSESSMENT

Performance Assessment

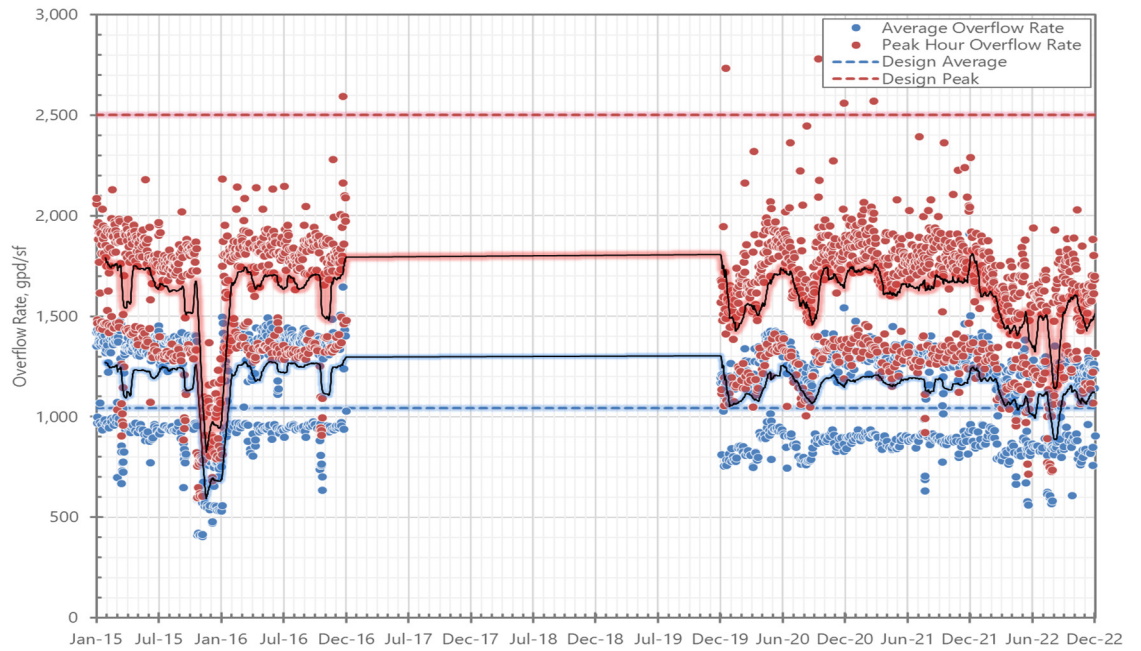


Figure 3E.1 PST Overflow Rate

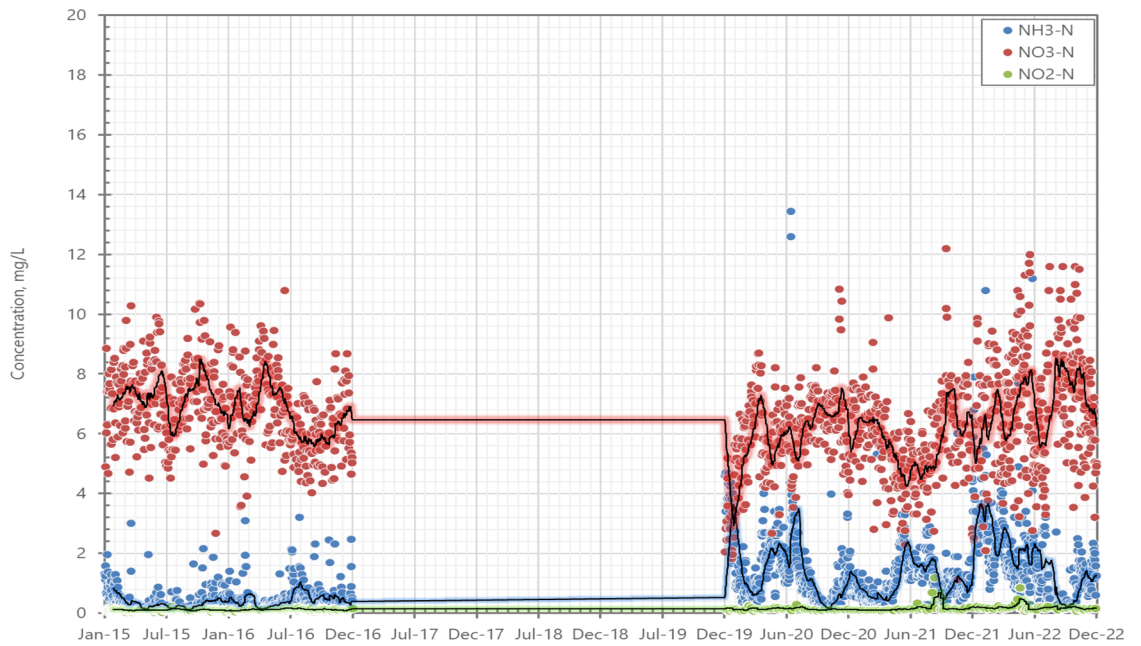


Figure 3E.2 Secondary Effluent Ammonia, Nitrite, and Nitrite Plus Nitrate (Sec01)

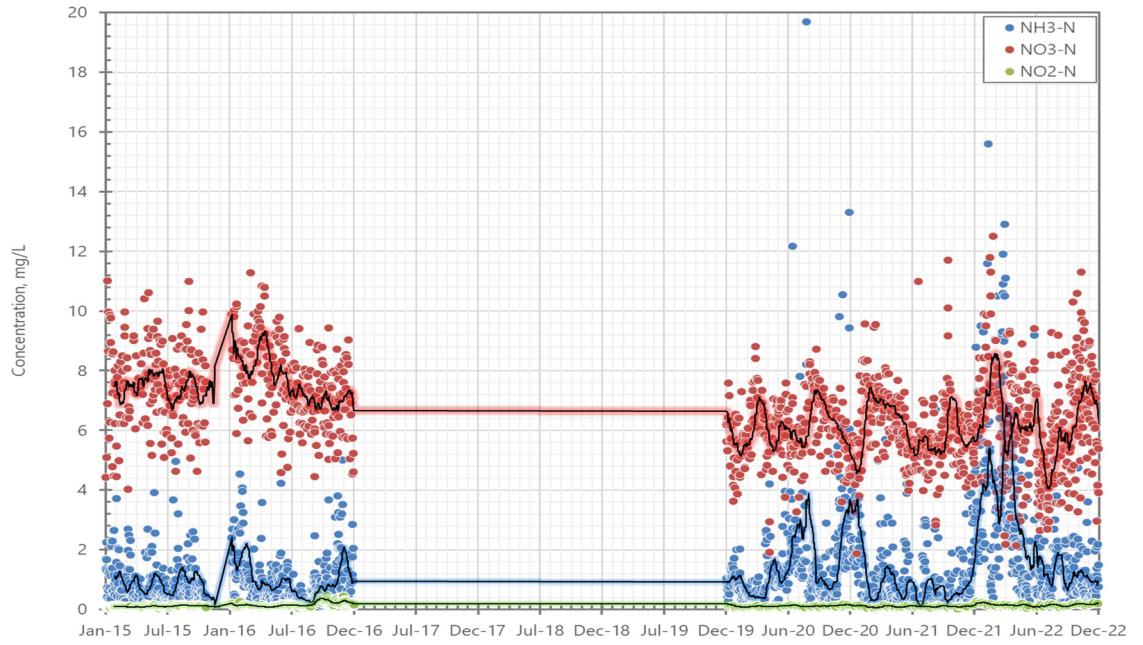


Figure 3E.3 Secondary Effluent Ammonia, Nitrite, and Nitrite Plus Nitrate (Sec02)

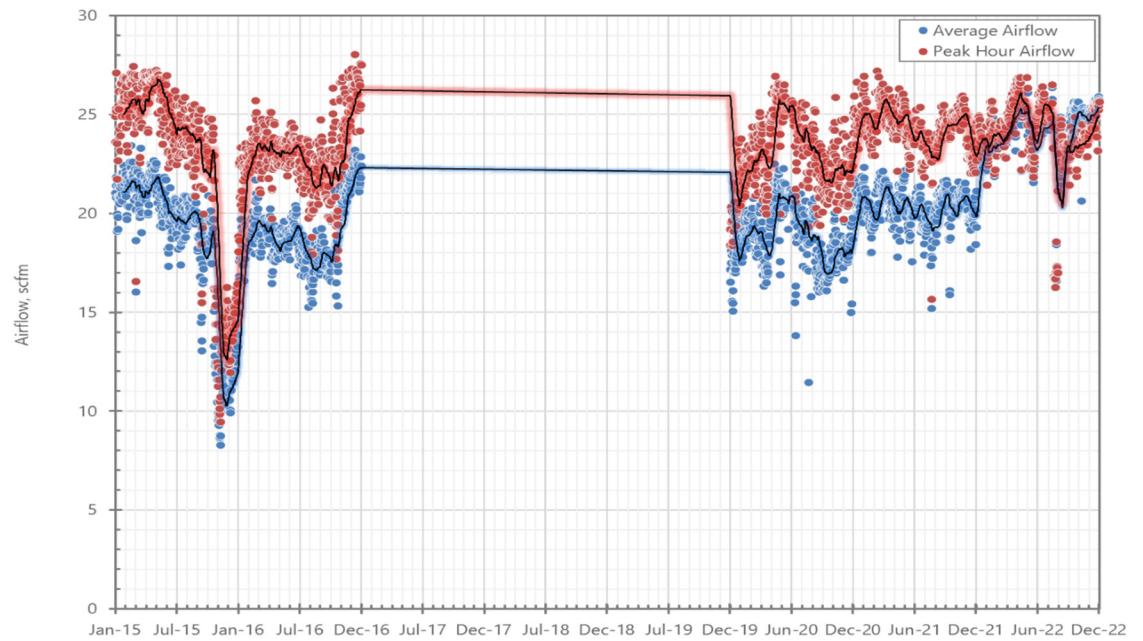


Figure 3E.4 Process Aeration (Blowers Total)

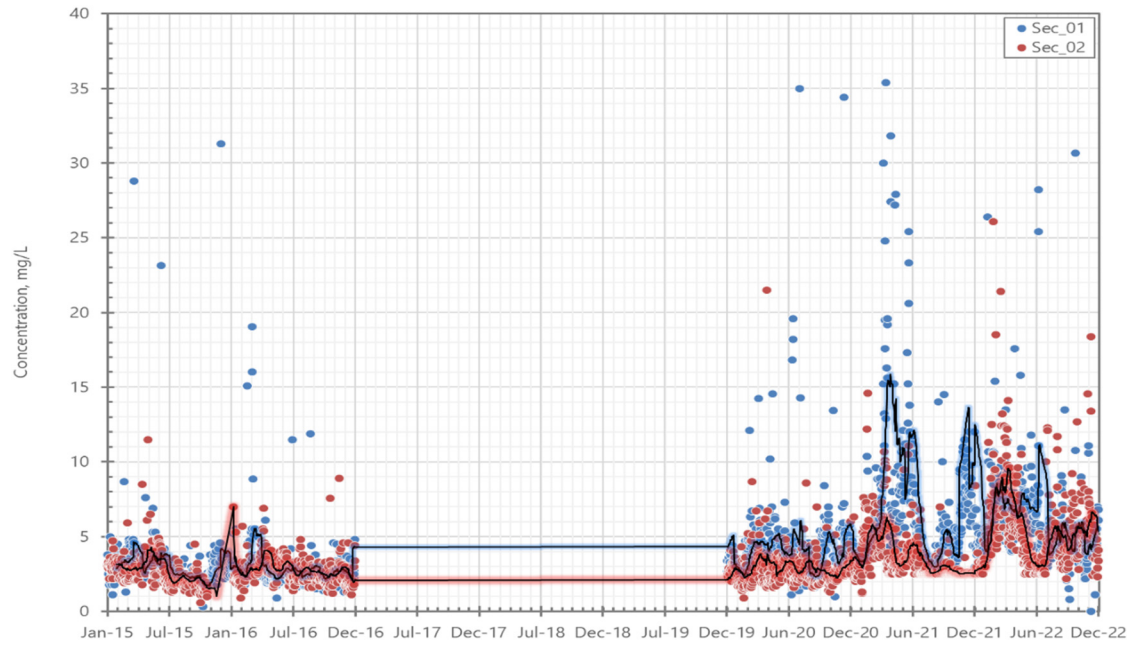


Figure 3E.5 Secondary Effluent TSS

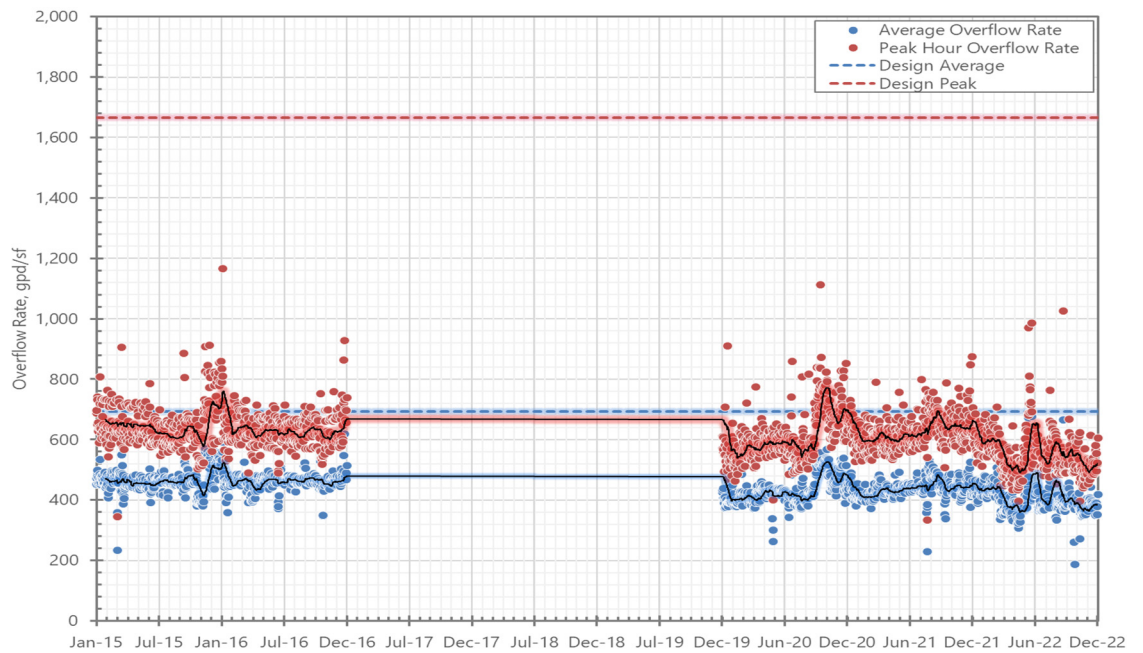


Figure 3E.6 FST Overflow Rate

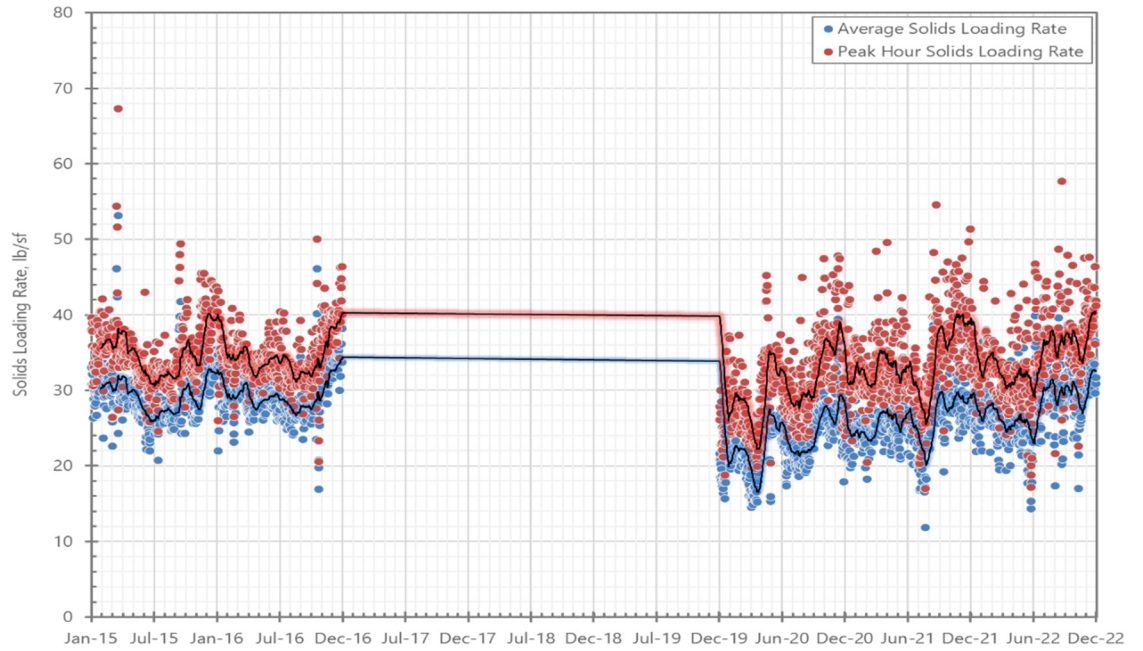


Figure 3E.7 FST Solids Loading Rate

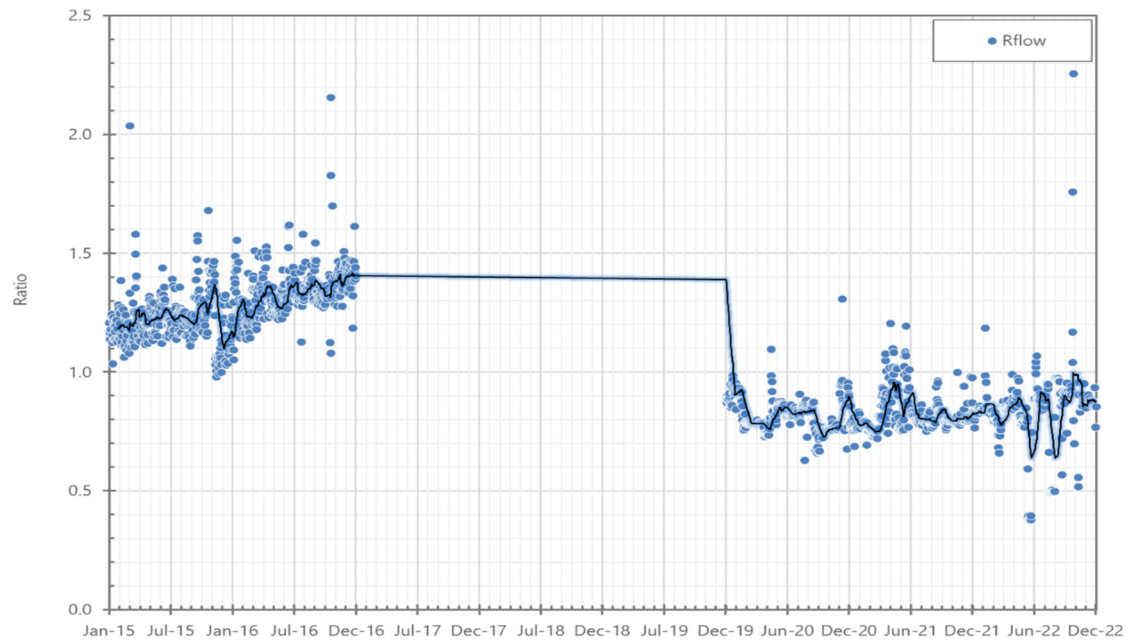


Figure 3E.8 RAS Flow Ratio (Stage 1 and 2)

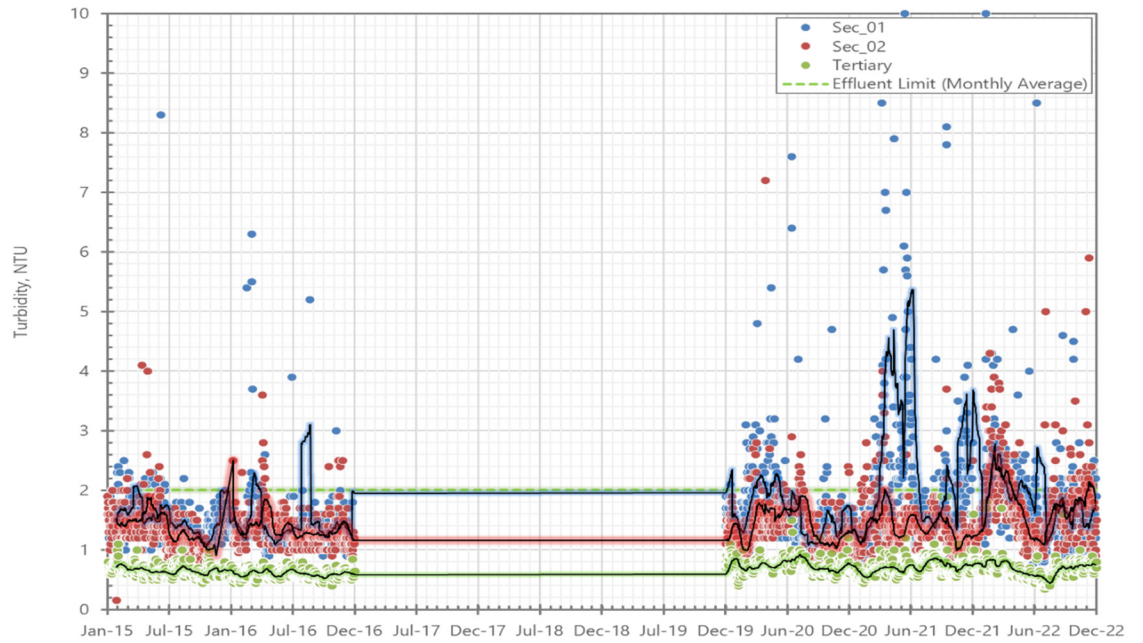


Figure 3E.9 Stage 1 and 2 Influent and Total Effluent Turbidity

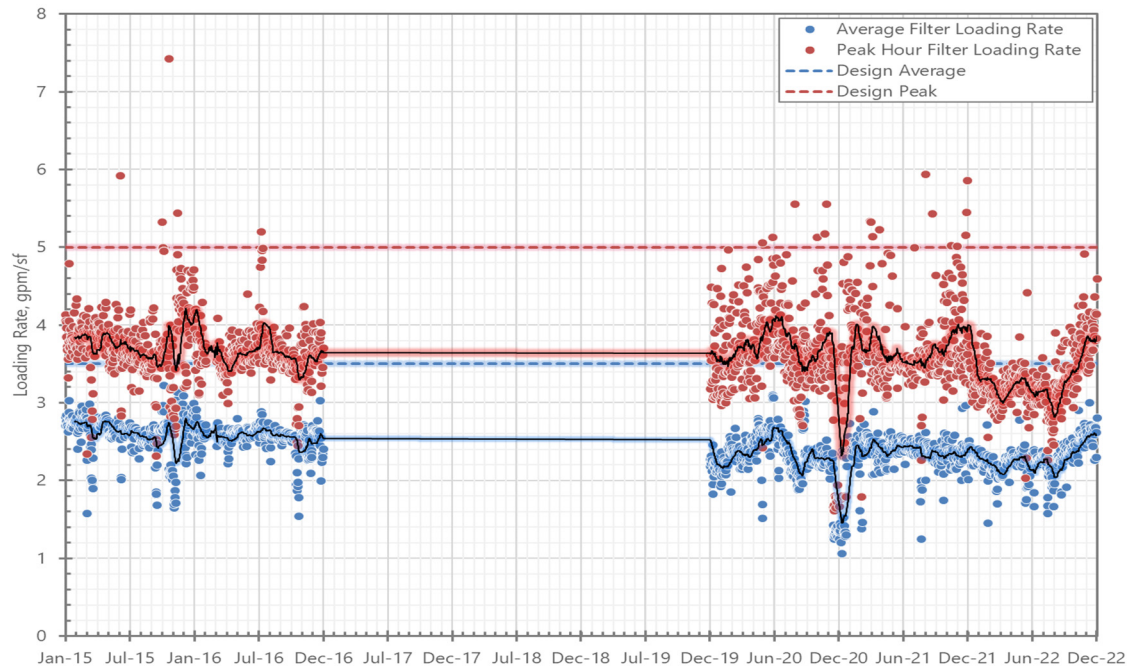


Figure 3E.10 Filter Loading Rate

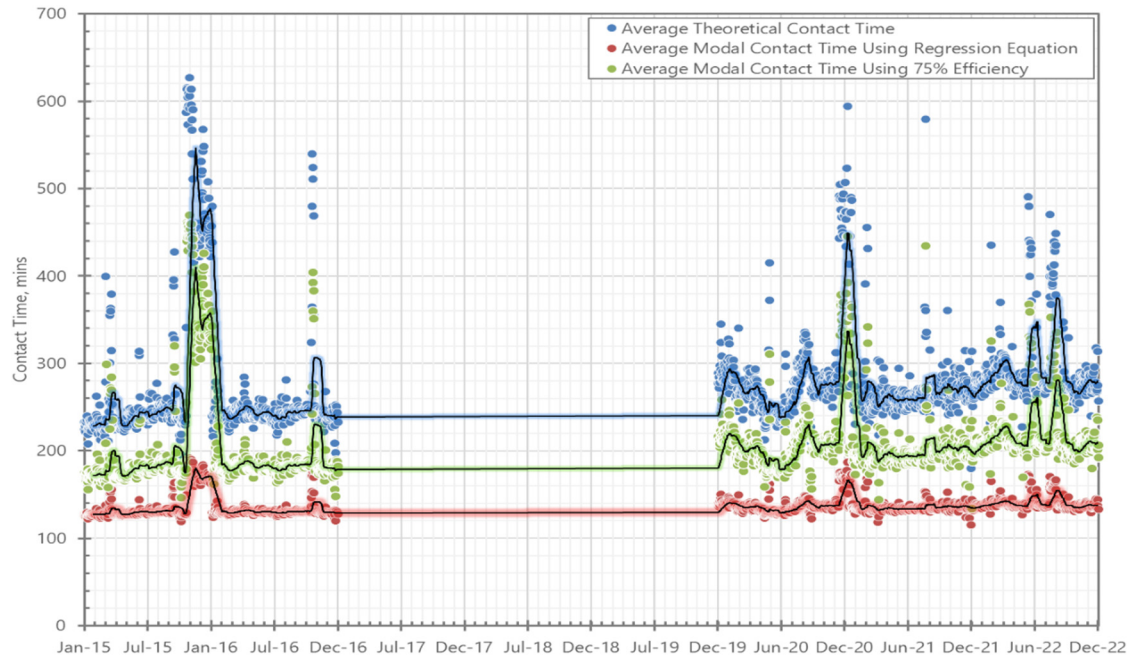


Figure 3E.11 Theoretical Chlorine Contact Time



Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 4
LONG BEACH WATER RECLAMATION
PLANT GAPS AND IMMEDIATE
NEEDS

FINAL | January 2025





**LOS ANGELES COUNTY
SANITATION DISTRICTS**
Converting Waste Into Resources

Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 4
LONG BEACH WATER RECLAMATION PLANT
GAPS AND IMMEDIATE NEEDS

FINAL | January 2025

Digitally signed by Andre Gharagozian
Contact Info: Carollo Engineers, Inc.
Date: 2025.01.06 09:04:39 -0800



Contents

Technical Memorandum 4 - Long Beach Water Reclamation Plant Gaps and Immediate Needs

4.1 Introduction and Summary	4-1
4.2 Phase 1 - Rehabilitate Aging Assets and Optimize Performance	4-3
4.2.1 Site Visit Projects	4-3
4.2.2 Optimization Process Elements	4-4
4.2.3 Phase 1 Costs and Benefits	4-16
4.2.4 Capacity Summary After Phase 1	4-19
4.2.5 Trigger Plots	4-20
4.2.6 Phasing of Optimization Projects	4-22

Tables

Table 4.1	Phase 1 Projects	4-2
Table 4.2	Site Visit Projects	4-3
Table 4.3	Process Optimization Opportunities	4-4
Table 4.4	Wet Weather Operational Strategy – FE Tank	4-7
Table 4.5	Densification Sizing Criteria	4-10
Table 4.6	Wet Weather Operational Strategy – Step Feed Distribution Modification	4-14
Table 4.7	Phase 1 - Rehabilitation and Optimization Costs and Benefits	4-18
Table 4.8	2050 Flow Projections Summary From TM 3	4-19
Table 4.9	Capacity Summary After Phase 1	4-19

Figures

Figure 4.1	FE Tank Volume	4-6
Figure 4.2	FE Facilities Site Layout	4-6
Figure 4.3	Settleability Improvements Flow Schematic	4-9
Figure 4.4	Settleability Improvements Site Layout	4-10
Figure 4.5	MLR Impact	4-13
Figure 4.6	MLR Pumping Improvements Overview	4-13
Figure 4.7	CCT Optimization Site Layout	4-15
Figure 4.8	Modifications to the Existing Effluent Channel	4-16
Figure 4.9	Secondary Process Capacity	4-16

Figure 4.10	Optimized Capacity	4-20
Figure 4.11	Secondary Process Trigger Plot	4-21
Figure 4.12	Tertiary Process Trigger Plot	4-21

Abbreviations

AADF	annual average daily flow
B&C&B	biostimulations, cyanotoxins, and biological
CCT	chlorine contact tank
CT	contact time
DO	dissolved oxygen
EIA	Energy Information Administration
ENR	<i>Engineering News-Record</i>
FE	flow equalization
FST	final sedimentation tank
gpm	gallons per minute
hp	horsepower
IPS	influent pump station
JOS	joint outfall system
kWh	kilowatt-hour
lb	pound(s)
LBWRP	Long Beach Water Reclamation Plant
LCWRP	Los Coyotes Water Reclamation Plant
MFP	Master Facilities Plan
MG	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
mL/g	milliliter per gram
MLR	mixed liquor recirculation
MLSS	mixed liquor suspended solids
NDMA	N-nitrosodimethylamine
NPDES	National Pollutant Discharge Elimination System
O&M	operations and maintenance
PAC	process air compressor
PDWF	peak dry weather flow
PE	primary effluent
PF	peaking factor
PFAS	per- and polyfluoroalkyl substances
PFOA	perfluorooctanoic acid
ppd	pounds per day
PST	primary sedimentation tank
PWWF	peak wet weather flow

RAS	return activated sludge
Sanitation Districts	Los Angeles County Sanitation Districts
SCADA	supervisory control and data acquisition
SJCWRP	San Jose Creek Water Reclamation Plant
SRT	solids retention time
SVI	sludge volume index
TIN	total inorganic nitrogen
TM	technical memorandum
TOC	total organic carbon
TSS	total suspended solids
Warren Facility	A.K. Warren Water Resource Facility
WAS	waste activated sludge
WNWRP	Whittier Narrows Water Reclamation Plant
WRP	water reclamation plant
WRR	water recycling requirement

Technical Memorandum 4

LONG BEACH WATER RECLAMATION PLANT GAPS AND IMMEDIATE NEEDS

4.1 Introduction and Summary

The Los Angeles County Sanitation Districts (Sanitation Districts) operate the A.K. Warren Water Resource Facility (Warren Facility) and six water reclamation plants (WRPs) in the Joint Outfall System (JOS). The WRPs include the Long Beach WRP (LBWRP), Los Coyotes WRP (LCWRP), Pomona WRP, San Jose Creek WRP (SJCWRP), and Whittier Narrows WRP (WNWRP). The sixth WRP, La Cañada WRP, does not have a National Pollutant Discharge Elimination System (NPDES) permit and is excluded from the Master Facilities Plan (MFP). The upstream reclamation plants are connected by more than 1,400 miles of District trunk sewers, Joint Outfall sewers, and Interceptors to the Warren Facility located in the city of Carson. Only a portion of flow treated at the LBWRP, LCWRP, and SJCWRP can be bypassed during dry peak flow periods due to limited downstream sewer capacity. To assist in the planning for future uses and permit compliance, the Sanitation Districts are preparing MFPs for the (1) SJCWRP, (2) LBWRP and LCWRP, and (3) Pomona WRP and WNWRP. The MFP for the SJCWRP was prepared as part of this effort. Preparation of MFPs for the remaining WRPs will be completed as part of future efforts, based on the technical memoranda (TM) developed during this effort.

The purpose of this TM is to summarize the gaps and immediate needs over the next 10 years at LBWRP. The information used to identify gaps and needs for the 20-year planning horizon builds on TMs developed for the MFP including: TM 1 - Flow and Load; TM 2 - Regulatory Review; and TM 3 - Existing Facilities, Performance, and Capacity Assessment. TM 3 identified minor capacity deficiencies with the secondary process and effluent filters as well as operational challenges with the disinfection system. This TM will identify improvement alternatives and capital improvement projects that should be implemented over the next 10 years to restore capacity and improve the plants' performance.

Similar to the previous effort for SJCWRP, immediate needs for LBWRP were prioritized in two phases as follows:

1. Phase 1 - rehabilitate aging assets and optimize performance.
2. Phase 2 - capacity expansion (if needed).

However, with the proposed improvements in Phase 1, it is anticipated that LBWRP will have sufficient capacity through the planning period of 2050. Therefore, Phase 2 (capacity expansion) might not be required for LBWRP. Recommended Phase 1 projects are summarized in Table 4.1.

Table 4.1 Phase 1 Projects

Process Area	Optimization Element	Estimated Capital Cost, \$ million ⁽¹⁾	Project Description and Other Comments
Phase 1 - Rehabilitation and Optimization			
FE Facilities	New FE Tank	\$29	Project includes a 2.5 MG FE tank, pump station, and odor control system.
Secondary ⁽²⁾	PAC Replacement Diffuser and Air Piping Replacement	\$36 ⁽³⁾	Project includes new PACs, air diffusers, MLR, and biotrickling filters for odor control. Capital cost reflects the addition of aeration grid control valves, air distribution piping, PAC control strategy, foam control, and sludge blanket indicators.
	Settleability Improvements (Densification and Surface Wasting)	\$9	Project consists of installing hydrocyclones to separate lighter biomass (which is wasted) while the denser biomass is returned to secondary process. Benefit of project will be improved settleability, process stability, and filter influent quality.
	Stability Improvements	\$4	Project will result in improved biological nutrient removal performance, less variability in effluent quality, and reduced power usage. Aeration control (DO) and SRT control are the two main elements of this project.
	MLR ⁽⁴⁾	--	The cost of this project is included in the PAC and diffuser replacement project. The optimization goal for MLR upgrade is to increase capacity. However, the MLR upgrade could also be used for improved biological nutrient removal performance if needed in future with no capacity increase.
Tertiary	Expand Filter Backwash Equalization	\$4	Provide 150,000-gallon backwash FE tank.
	Disinfection Optimization	\$18	Project includes addition of a new CCT and new effluent weir upstream of the existing effluent weir.
Total Phase 1 - Rehabilitation and Optimization		\$100	

Notes:

Abbreviations: CCT - chlorine contact tank; DO- dissolved oxygen; FE - flow equalization; MG - million gallons; MLR - mixed liquor recirculation; PAC - process air compressor; SRT - solids retention time.

(1) Based on August 2023 dollars using an *Engineering News-Record* (ENR) ranking of 15179 for Los Angeles.

(2) Mixers are planned to be added to the anoxic zones, which will increase nitrogen removal efficiency through more uniform mixed liquor suspended solids (MLSS) concentration in the anoxic zones.

(3) Sanitation Districts estimate from DMS-#6539317-v6-LBWRP (August 2022).

(4) This project will be implemented as part of the PAC and diffuser replacement; therefore, the cost of this project is included in the estimated capital cost of \$36 million for the PAC and diffuser replacement project.

In addition to the immediate needs, new treatment facilities will be required for long-term needs (10 to 20 years). Long term needs are largely driven by anticipated future regulations for effluent temperature, total organic carbon (TOC), per- and polyfluoroalkyl substances (PFAS)/perfluorooctanoic acid (PFOA), and biostimulations, cyanotoxins, and biological (B&C&B) condition provisions. Long term needs (if needed) for LBWRP are evaluated in TM 5 - Alternatives and they are prioritized in four additional phases as follows:

- Phase 3 - temperature.
- Phase 4 - TOC compliance.
- Phase 5 - PFOS/PFOA compliance.
- Phase 6 - B&C&B requirements for nutrients.

4.2 Phase 1 - Rehabilitate Aging Assets and Optimize Performance

Recommended projects needed to rehabilitate aging assets from the site visit observations as well as projects to optimize process performance are summarized in this section. Anticipated benefits and costs for optimizing process performance are also summarized. The Sanitation Districts do not have existing documentation on condition assessments for LBWRP.

4.2.1 Site Visit Projects

A site visit with the MFP team and Sanitation Districts operations and maintenance (O&M) and engineering staff was conducted on January 24, 2023. During the site visit, some challenges and issues with existing facilities were identified. Challenges and projects identified during the site visit are listed in Table 4.2. A number of these projects are being addressed by staff, so they do not need to be included in the TM recommendations.

Table 4.2 Site Visit Projects

Process Area	Projects/Challenges Identified	Project Status
IPS and PSTs	Odor control for IPS is inadequate, install new odor control.	In Design
	Install chopper pumps in the influent channel to address floating plastics and rags.	To be addressed In house by Sanitation Districts
	Some of the PSTs are used to store flows on weekends and then drained to sewer. Treat equalized/stored flows from 3rd and 4th PSTs instead of sending it to the sewer.	--
	Influent gate replacement to resolve the issues with disengaging the gates.	To be addressed In house by Sanitation Districts
Aeration Tanks and FSTs	Install backup pump for RAS Stage 2.	To be addressed In house by Sanitation Districts
	Significant foaming issues. Install foam spray piping and collection and surface wasting to address foaming problems.	--
	Minimal aeration controls available to operators.	--
	PAC replacement.	In Design
	Air piping is 25 years old. Replacement of ceramic diffusers with membrane diffusers.	Completed by Sanitation Districts

Process Area	Projects/Challenges Identified	Project Status
Filters	Discharge filter backwash wastewater to a new FE tank to address issues with hydraulics during backwash in wet weather.	--
	Filter backwash valves do not seal properly.	--
Disinfection	Additional improvements needed to mitigate impact from recycle pump station causing variation in water levels and CT.	--
	Move dechlorination facilities closer to CCT to prevent scaling in pipe. A temporary dosing system has been installed that is closer to the CCT. A permanent dosing system is in design.	--
	There is settlement in washwater pump area. Pumps need to be moved.	--
	Issues with NDMA prompted switch to emulsion polymer.	Completed by Sanitation Districts

Notes:

Abbreviations: CT - contact time; FST - final sedimentation tanks; IPS - influent pump station; NDMA - N-nitrosodimethylamine; PST - primary sedimentation tanks; RAS - return activated sludge.

4.2.2 Optimization Process Elements

The Regulatory Review in TM 2 and evaluation of Existing Facilities, Performance, and Capacity Assessment in TM 3 confirmed that the LBWRP is in compliance with the NPDES permit requirements, however, some unit processes are operating under stressed conditions or near their capacity limits. In addition, O&M staff identified several areas where optimization would be beneficial during the site visit on January 24, 2023 and at project meetings. The benefits of process optimization include achieving improved performance, operational stability, additional capacity, and reduced power and chemical usage. Process optimization opportunities are summarized in Table 4.3.

Table 4.3 Process Optimization Opportunities

Process	Optimization Opportunities
FE Facilities	<ul style="list-style-type: none"> Addition of an FE tank improves overall process performance and provides additional capacity.
Secondary Process	<ul style="list-style-type: none"> PE flow distribution between treatment units and step feed locations. Diffuser and air piping replacement. MLR pumping. Settleability Improvements (densification and surface wasting). Stability Improvements (optimized SRT, sludge blanket, aeration controls and nutrient monitoring). Addition of mixers to anoxic zones.
Effluent Filtration	<ul style="list-style-type: none"> Provide 150,000-gallon backwash FE tank.
Disinfection	<ul style="list-style-type: none"> Addition of a new 4th CCT tank and new effluent weir upstream of the existing effluent weir to mitigate varying CT caused by reclaimed water pump operation, which draws down water levels.

Notes:

Abbreviations: PE - primary effluent.

A discussion of the optimization opportunities for each process area is provided below.

4.2.2.1 Influent Pump Stations and Primary Sedimentation Tanks

LBWRP does not have influent screens. Floating plastics in the PSTs were visible during the site visit. O&M staff have also indicated that rags accumulate and get carried to PST influent channel and results in jamming the channel. Rag accumulation could also cause issues at the influent and RAS pumping stations. The Sanitation Districts are planning to install chopper pumps in the PST influent channel at LBWRP as they have observed significant reduction in rag accumulation in the influent channel and reduced derailing of chain on sludge collection equipment at the SJCWRP with installation of chopper pumps. The installation of chopper pumps project is scheduled for completion in 2024.

4.2.2.2 Flow Equalization Facilities

The addition of a FE tank will enhance the biological treatment and improve effluent quality at the LBWRP. The Sanitation Districts designed and constructed FE facilities for the SJCWRP which resulted in the following benefits:

Increased recycled water production - stored flows are treated during nighttime hours of low influent flows, resulting in increased recycled water production at night when recycled water demands are the highest.

- Improved and more consistent unit process performance - FE tank allows for diversion of high ammonia peak morning flows which significantly improves the quality of secondary effluent.
- Improved stormflow management - allows for reduction in wet weather peaks, therefore reducing the impact of high stormflows on WRP performance.
- Reduced peak energy demands - steady operation allows for reduction in peak power demands.

The addition of an FE tank at the LBWRP would result in similar benefits and allow for additional capacity by reducing the storm flow peaking factors. Diurnal flow data for dry- and wet- weather flows were analyzed, and a sensitivity analysis was performed for various wet weather events to determine the appropriate volume of the FE tank, as shown on Figure 4.1. The 2015 peak wet weather event was the most significant event in the reviewed data set. For the 2015 event, a 2.5 MG FE tank would have allowed peak equalized flow to be reduced to 28.5 million gallons per day (mgd) which would have reduced the peak wet weather peaking factor from 2.40 to 1.85. For 2016 and 2021 storm events shown on Figure 4.1, a 2.5 MG FE tank would have reduced the peaking factor to even less than 1.85. For planning purposes, a 2.5 MG FE tank is recommended. The addition of an FE tank at LBWRP will increase the average dry weather capacity by 1.6 mgd.

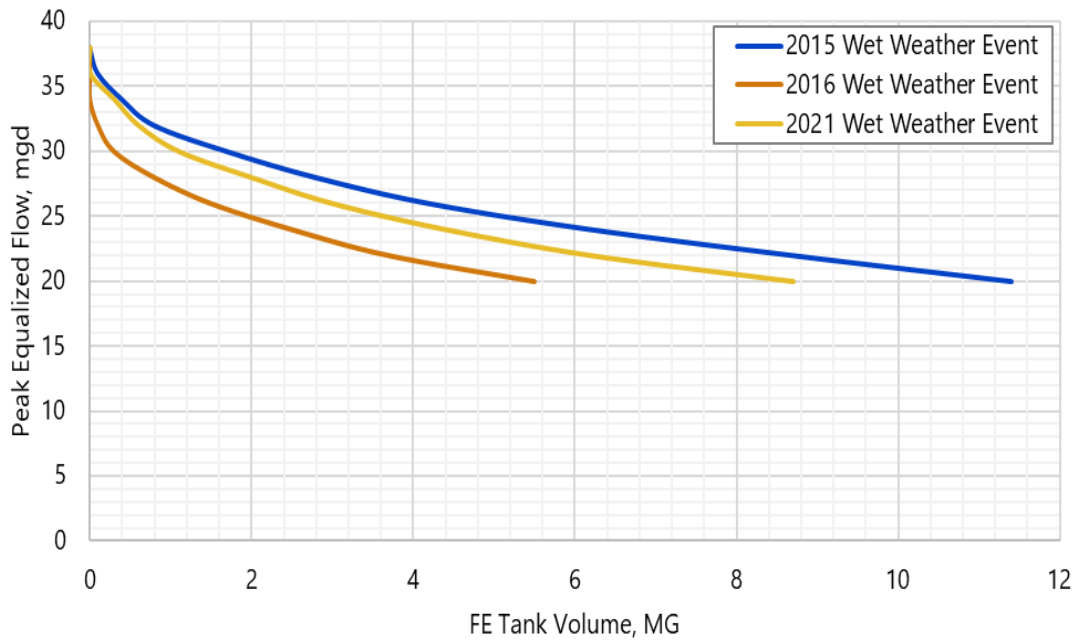
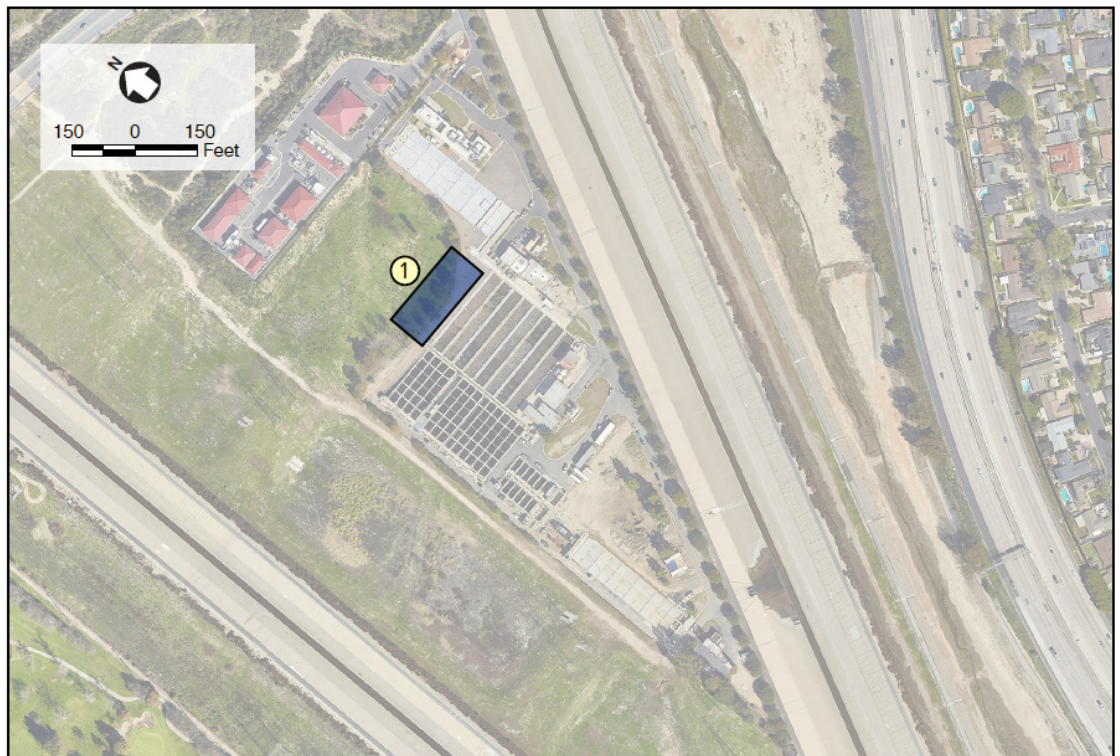


Figure 4.1 FE Tank Volume

Figure 4.2 shows the proposed location for the FE tank.



① Flow Equalization Facilities (225'x100')

Figure 4.2 FE Facilities Site Layout

With the recent wet weather event in February 2024, the peak wet weather flow (PWWF) and peaking factors (PF) were reevaluated as described in TM3 and strategies were developed. Since the 10-year average PWWF PF is 2.3 (lower than the 2050 PWWF peaking factor of 2.4), the recommended 2.5 MG FE tank was not re-sized. To demonstrate the benefit of the FE tank during wet weather events, a higher PWWF from 2017 (53.4 mgd) was used. In Approach 1 in Table 4.4, by keeping the EQ volume at 2.5 MG, the peak flows are reduced to 43.6 mgd. This approach demonstrates that the FE tank would help reduce peak flows during wet weather events. Approach 2 is to keep the equalized flow the same for this higher PWWF event by increasing tank size. As shown in Table 4.4, Approach 2 results in a tank volume of 17.5 MG, which would require significant space and cost to implement. Therefore, it is not recommended to re-size the FE tank based on the maximum peak flow in the last 10 years. The recommendation is to proceed with the 2.5 mgd FE Tank size.

Table 4.4 Wet Weather Operational Strategy – FE Tank

Parameter	Current Recommendation	Approach 1 – Same FE Tank Volume	Approach 2 – Same Equalized PF
PWWF, mgd	37.0	53.4	53.4
AADF, mgd	15.4	15.4	15.4
PWWF PF	2.4	3.5	3.5
FE Tank Volume, MG	2.5	2.5	17.5
Peak Equalized Flow, mgd	28.5	43.6	28.5
Equalized PF	1.85	2.83	1.85

Notes:

Abbreviations: AADF - annual average daily flow.

4.2.2.3 Secondary Process

Several optimization opportunities were identified for the secondary process and are discussed in the following sections.

Primary Effluent Flow Distribution

PE flow distribution between treatment units and between step feed locations is achieved through submerged gates. Each feed location has a propeller flow meter, and gate positions have been adjusted to match the desired distribution. A significant amount of effort has been invested by the Sanitation Districts to study and optimize the flow distribution at the SJCWRP. However, optimizing the flow distribution has been challenging especially due to the system's size and overall complexity. In addition, the accuracy of the flow distribution is reliant on accurate flow and total suspended solids (TSS) measurements in the aeration tanks. O&M staff has indicated that the flow meters are not reliable as they are not always flowing full and are also prone to plugging. On-line monitoring of suspended solids requires frequent maintenance and calibration to maintain accuracy.

Based on the effort for optimizing the flow distribution at SJCWRP, it was determined that the potential for performance improvements at LBWRP is minimal. Therefore, no specific optimization improvement is included for the LBWRP flow distribution.

Settleability Improvements (Densification and Surface Wasting)

The average mixed liquor settleability is near the "typical" range of 150 milliliter per gram (mL/g) (as sludge volume index (SVI) measurement). However, the SVI has a lot of variability and has fluctuated between 75 mL/g to 175 mL/g in the recent years. The common impacts of higher SVIs and overall variability are as follows:

- Reduced secondary flow capacity.
- Higher sludge blankets in the FSTs.
- Reliance on emulsion polymer as a settling aid.
- Overload effluent filters require occasional bypass during wet weather events.

A common approach to improving activated sludge settleability is incorporating selectors into the process, which are unaerated zones that favor the growth of microbiology that settles well. It is unlikely that modifying the anoxic zones will provide significant benefit because the anoxic zones in the step feed process already function as selectors.

Another approach is to selectively waste the poorly settling organisms while retaining the organisms that settle well. Selective wasting can be accomplished through a process called densification. With densification, a portion of the RAS is processed through hydrocyclones, which separates the denser biomass from the lighter biomass. The lighter biomass (which doesn't settle as well) would be wasted at whatever rate is necessary for maintaining the target SRT, while the denser biomass would be returned to the secondary process. There are more than a dozen demonstration and full-scale facilities in the United States, and most have shown significant improvements in SVI. Metro Water Recovery in Denver operated a 10 million gallons per day (mgd) demonstration facility for three years and found the average SVI was 126 mL/g and 75 mL/g for the control and demonstration facility, respectively. The 75th and 98th percentile values also showed a similar benefit. For the purposes of this analysis, it is assumed that implementing these improvements would result in an SVI of 100 mL/g.

Figure 4.3 is a flow schematic of the proposed densification facilities.

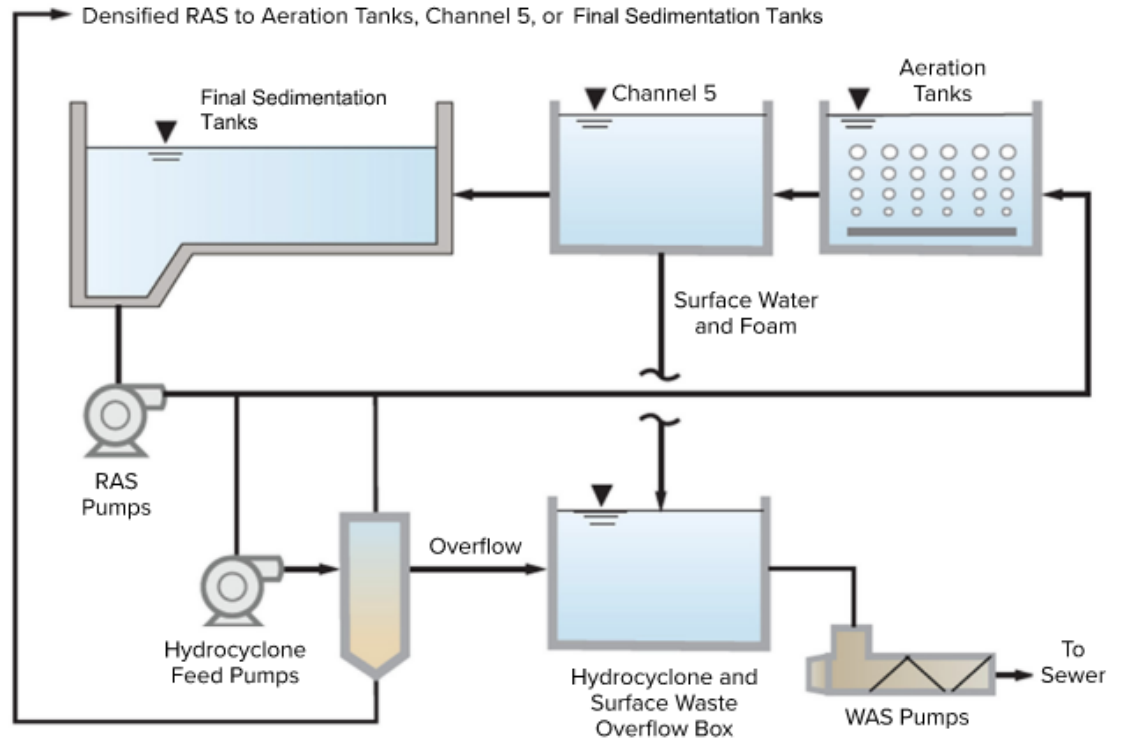


Figure 4.3 Settleability Improvements Flow Schematic

Proposed locations for each system are estimated and shown on Figure 4.4. The proposed locations for the settleability improvements are near existing RAS Pump Stations 1 and 2. It is recommended that, during design, the Sanitation Districts consider whether the facilities can be located near the mixed liquor channels to provide an opportunity to waste foam that accumulates on the top of the channels. This feature provides additional capability to remove lighter biomass that does not settle well, which can help improve settleability.



- ① Densification/Surface Wasting Facility, Stage 1 (30'x20')
- ② Densification/Surface Wasting Facility, Stage 2 (30'x20')
- ③ Channel modifications with downward opening weir gate

Figure 4.4 Settleability Improvements Site Layout

Table 4.5 summarizes basic sizing criteria for the densification facilities.

Table 4.5 Densification Sizing Criteria

Item	Unit	LBWRP Criteria
Flow	mgd	15.9
Flow per Train	mgd	7.8
WAS to Sewer, TSS (Maximum Month)	ppd	24,586
RAS Concentration	mg/L	6,000
WAS Flow to Sewer at RAS Concentration	gpm	525
Hydrocyclone Feed Pumps		
Number		3+1
Capacity, each	gpm	176
Firm Capacity	gpm	528
Power, each	hp	15

Item	Unit	LBWRP Criteria
Hydrocyclones		
Number of Skids	--	4
Number of Cyclones/Skid	--	4
Total Number of Cyclones	--	16
Inlet flow per Cyclone	gpm	44
Firm Capacity	gpm	704
Hydrocyclone Overflow, (Maximum Month)	gpm	420
Hydrocyclone Concentration	mg/L	4,875
Hydrocyclone Load, TSS (Maximum Month)	ppd	24,586
WAS Pumps		
Number	--	3+1
Capacity, each	gpm	210
Firm Capacity	gpm	630
Power, each	hp	7.5

Note:

Abbreviations: gpm - gallons per minute; hp - horsepower; mg/L - milligrams per liter; ppd - pounds per day; WAS - waste activated sludge.

Stability Improvements

Stability improvements are centered around improving process control and stabilizing process performance. The goal of the stability improvements is to reduce the variability in the process operating conditions as well as effluent ammonia and nitrate concentrations.

One element of the stability improvements is implementing SRT control. Historically, the WRPs have controlled sludge wasting by maintaining target MLSS concentrations. While this approach results in a more consistent quantity of biomass inventory and solids loading on the FSTs, the SRT varies significantly as the influent loads change. Maintaining a more consistent SRT has been demonstrated to reduce foaming and settleability issues and result in more predictable air usage and nutrient removal performance. The Sanitation Districts could implement SRT control at LBWRP with in-house engineering support, or through purchasing a commercially available package. In either case, SRT control would include using on-line TSS monitoring data throughout the aeration tanks and FSTs to establish wasting requirements. The Sanitation Districts would likely need to purchase additional TSS instrumentation and make programming changes to the supervisory control and data acquisition (SCADA). More sophisticated systems incorporate model-predictive control and "Big Data" analytics to fine tune control based on prior performance.

Another element of the stability improvements is implementing nutrient instrumentation in the aeration tanks to be able to monitor performance. Nutrient instrumentation is currently being evaluated through full-scale testing at the Pomona WRP. These improvements would provide information that could be used to further optimize and enhance aeration control as well as PE flow distribution between passes at LBWRP. Like the SRT control, the Sanitation Districts could implement nutrient instrumentation as an in-house project or use a commercially available

package. Model predictive and "Big Data" analytics could also be utilized to maximize the benefit to performance. Instrumentation that would be considered include analyzers for:

- PE ammonia.
- Ammonia in aerobic zones.
- Nitrate and oxidation reduction potential in anoxic zones.
- Secondary effluent nitrate.

It is anticipated that the LBWRP will see a reduction in power usage with these improvements. In addition, it is expected that the LBWRP could reliably operate at a lower aerobic SRT of 4.5 days with the enhanced controls as compared to 5.0 days.

Mixed Liquor Recirculation Pumping

MLR is a common approach for improving denitrification efficiency. Additional nitrate reduction can be achieved by recirculating from aerobic zones where nitrate is present to anoxic zones with sufficient carbon compared to what is achieved with the step feed process alone. The plant process model was used to simulate a range of recirculation flow rates and configurations to evaluate the optimal approach. The impact of MLR pumping on the effluent nitrate concentration is illustrated on Figure 4.5. On Figure 4.5, MLR 21 means MLR pumping from Pass 2 to Pass 1, MLR 31 means MLR pumping from Pass 3 to Pass 1, and MLR 43 means MLR pumping from Pass 4 to Pass 3.

Results of the modeling indicate the following:

- MLR pumping from Pass 4 to Pass 3 and Pass 2 to Pass 1 (solid orange line) results in the lowest nitrate effluent nitrate concentration, however, implementation of MLR pumping at two separate locations might not be feasible.
- MLR pumping from the end of the aerobic zone in Pass 2 to the anoxic zone in Pass 1 (solid blue line) at a rate of 150 percent of the influent flow is optimal and provides approximately 2 mg/L of reduction in the effluent nitrate concentration.
- MLR pumping from the end of the aerobic zone in Pass 3 to the anoxic zone in Pass 1 (solid yellow line) at a rate of 300 percent of the influent flow provides comparable reduction in the effluent nitrate concentration.

There are other benefits to implementing this optimization element in addition to improving nutrient removal performance:

- Implementing MLR pumping will ultimately be needed if the Sanitation Districts upgrade the Step Feed process to a 4- or 5-Stage Bardenpho process to meet more stringent nutrient requirements. MLR pumping from Pass 3 to Pass 1 will be ideal for conversion to a 4- or 5-Stage Bardenpho process.
- Nutrient removal performance will be less sensitive to the PE flow distribution between step feed locations.
- The anoxic zone in Pass 3 could be converted to aerobic and the PE feed to Pass 3 could be reduced or eliminated if improved nutrient removal performance is not needed. This change would simplify the PE flow distribution and offer an opportunity to further increase process capacity. This option for additional capacity is recommended for the near term since nutrient removal is not yet required.

Although MLR pumping from Pass 3 to Pass 1 has comparable benefits, it is recommended that MLR pumping from Pass 2 to Pass 1 be implemented in Phase 1. Implementing MLR pumping from Pass 2 to Pass 1 will be more economical and easier to implement. MLR pumping can be modified in later phases if the 4- or 5-Stage Bardenpho conversion is required for future B&C&B requirements for nitrogen.

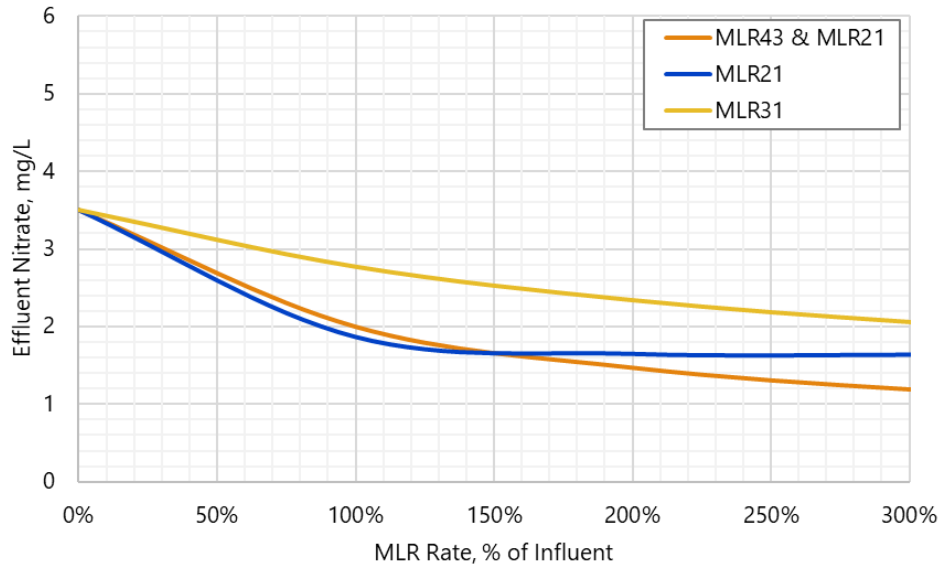


Figure 4.5 MLR Impact

Proposed improvements for the treatment units is illustrated on Figure 4.6.

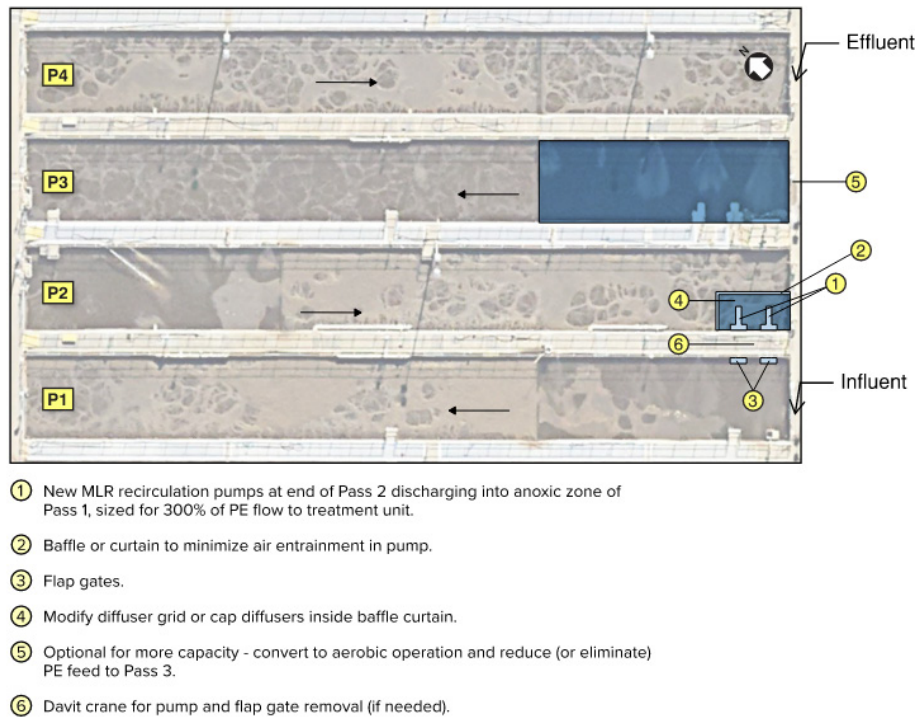


Figure 4.6 MLR Pumping Improvements Overview

4.2.2.4 Step Feed Modification for Wet Weather Events

As an operational strategy, the primary effluent flow distribution can be modified during severe storm events to handle higher incoming flows. The current PE flow distribution at LBWRP is approximately 55/30/15 between Pass 1, 2, and 3 respectively. If all the incoming flows are sent to Pass 3 directly (PE flow distribution of 0/0/100), the secondary treatment can overall handle higher peak flows by maintaining the MLSS concentration such that the FSTs are not overloaded. With this strategy, the final effluent concentrations for ammonia and total inorganic nitrogen (TIN) will be higher and, therefore, this strategy should only be implemented for a short period to handle wet weather events. Table 4.6 demonstrates the benefits of modifying PE flow distribution of step feed at LBWRP.

Table 4.6 shows the current operation at LBWRP at the current secondary PWWF capacity of 35.5 mgd (TM3 Section 3.6) with existing PE flow distribution of 55/30/15 between Pass 1, 2, and 3 respectively, and an MLSS of 2,800 mg/L. This produces an effluent TIN concentration of 3.6 mg/L that is well below the current discharge limit without overloading FSTs. In a modified step feed approach, the PE flow distribution is modified to 0/0/100 between Pass 1, 2, and 3. Table 4.6 shows this modified step feed approach for two conditions: 1) at the current PWWF capacity and 2) at a higher flow to demonstrate that a modified step feed approach can handle additional flows. For Condition 1, with PWWF of 35.5 mgd the resulting MLSS concentration would be 2,000 mg/L. For Condition 2 peak flows are increased to 53.4 mgd and the resulting MLSS is 2,700 mg/L which is still lower than the current operation. Therefore, a modified step feed operation (PE flow distribution at 0/0/100 between Pass 1, 2, and 3 respectively) would allow higher flows without overloading the FSTs. However, it is important to note that for both Conditions 1 and 2, the TIN performance will be degraded during this type of operation. Therefore, as mentioned above, this strategy should only be implemented for a short period of time to handle wet weather peak flows and not extended peak day or peak week conditions.

Table 4.6 Wet Weather Operational Strategy – Step Feed Distribution Modification

Parameter	Current Operation – Current Secondary PWWF Capacity	Condition 1 - Modified Step Feed Operation at Current Secondary PWWF Capacity	Condition 2 - Modified Step Feed Operation at Highest Peak Flow
PE Flow Distribution, %	55/30/15	0/0/100	0/0/100
PWWF, mgd	35.5	35.5	53.4
AADF, mgd	14.8	14.8	19.8
PWWF PF	2.4	2.4	2.4
MLSS in Pass 4, mg/L	2,800	2,000	2,700
Effluent Ammonia, mg/L	0.05	8.1	8.3
Effluent TIN, mg/L	3.6	16.6	16.8

4.2.2.5 Effluent Filtration

There were some filter bypasses over the last few years despite sufficient capacity and good filter performance. The filter bypasses were likely due to high solids loads during wet weather events.

The current system pulls backwash from the filter effluent channel resulting in large instantaneous flow swings, which makes stable disinfection process chlorine dose control challenging. A 150,000-gallon backwash FE tank is recommended to store water for filter backwash to stabilize the chlorine dose control. Additionally, there should be an improvement in the filter performance due to reduced peak flows with the new proposed FE facilities.

4.2.2.6 Disinfection

Title 22 compliance issues with the future water recycling requirements (WRR) were identified in TM 2 and TM 3. There were several instances where the effluent did not meet the 450-CT requirement with the existing volume and operation. Direct reuse water withdrawal from the CCTs causes variation in the water level and results in an operational challenge to comply with the 450 CT requirement.

An optimization project that includes addition of a new CCT tank and modifications to the influent and effluent channels was identified to overcome this challenge. The modifications include:

- New effluent weir upstream of the existing effluent weir to control the water level in the CCTs.
- New effluent box connecting to the existing reuse channel such that the effluent from CCT No. 4 will be only reused, not discharged.
- Closure of the existing effluent weir at CCT No. 3. CCT Nos. 1 and 2 effluent flow will be discharged to Coyote Creek if it is not reused.

The site layout for disinfection optimization is shown on Figure 4.7 and a section view of the proposed modifications to the existing effluent channel is shown on Figure 4.8.



① New Chlorine Contact Tank (287.5'x20')

Figure 4.7 CCT Optimization Site Layout

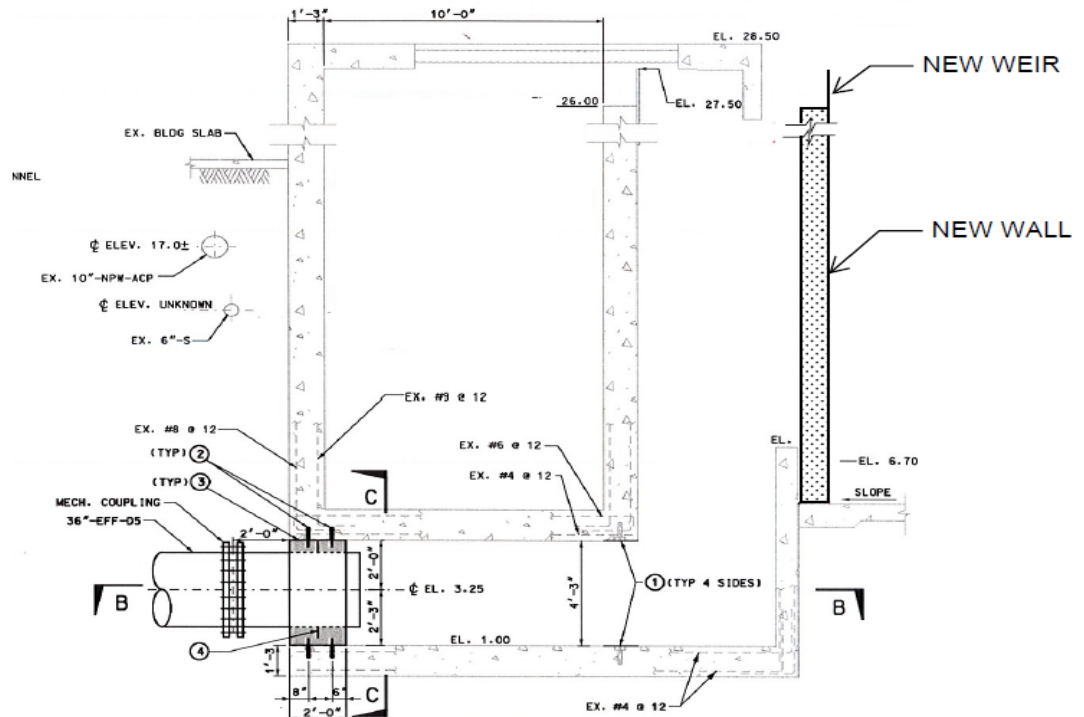


Figure 4.8 Modifications to the Existing Effluent Channel

4.2.3 Phase 1 Costs and Benefits

The Sanitation Districts should see significant capacity benefit from implementing all the secondary process optimization elements. The impact for the LBWRP is illustrated on Figure 4.9.

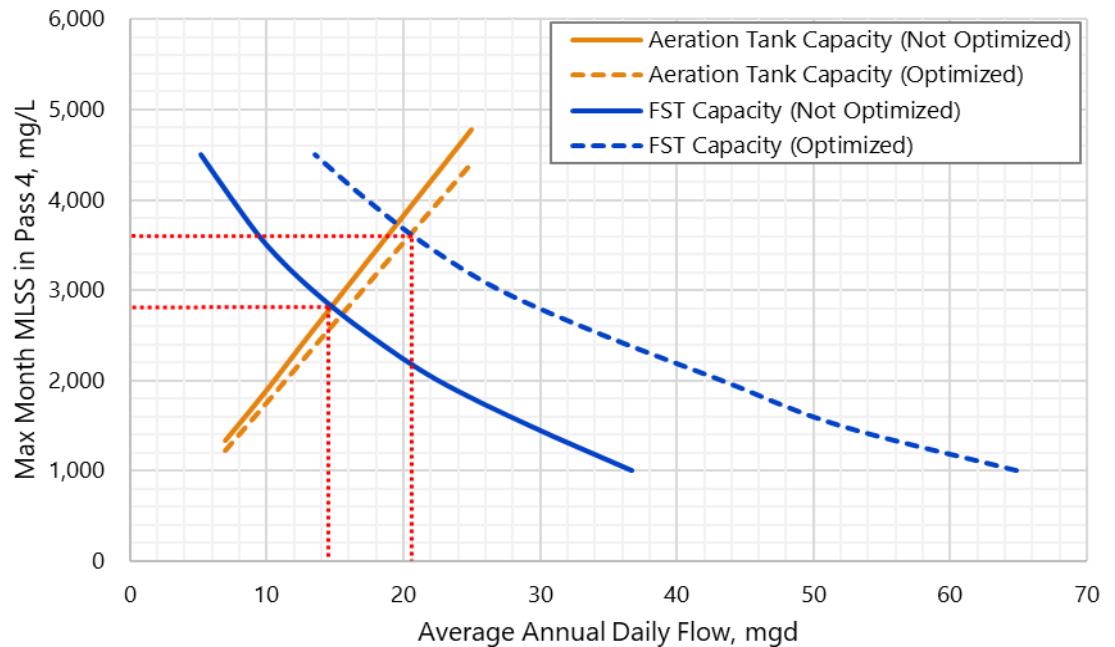


Figure 4.9 Secondary Process Capacity

The orange lines reflect aeration tank capacity, while the blue lines represent FST capacity. The solid lines and their intersection reflect the capacity of the existing facilities, which results in an annual average daily flow (AADF) capacity of 14.8 mgd. The intersection of the dotted lines reflects the capacity with optimization projects implemented, which corresponds to an AADF capacity of 20.4 mgd.

The estimated capital costs, O&M impacts, and pros and cons for each of the key Phase 1 projects including rehabilitation, replacement, and optimization elements is summarized in Table 4.7. Note that the capacity impact and cost for each individual element of optimization is shown in Table 4.7; however, it is recommended that all the optimization elements be implemented. The capacity benefit may not be fully realized without implementation of all the improvements.

Table 4.7 Phase 1 - Rehabilitation and Optimization Costs and Benefits

Process Area	Optimization Element	Estimated Capital Cost, \$ million ⁽¹⁾	AADF Capacity Increase, mgd	Cumulative AADF Capacity, mgd	Changes to Power Usage, kWh/year	Changes to Annual Power Cost, \$ ⁽²⁾	Changes to Polymer Usage, lb active/year	Changes to Annual Polymer Cost, \$ ⁽³⁾	Project Description and Other Comments
	Current			14.8					Rated capacity from TM 3.
	Install Flow Equalization Facilities	\$29	1.6	16.4	-112,000	-\$24,000	0	0	Project includes a 2.5 MG FE tank, pump station, and odor control system.
Secondary	PAC Replacement Diffuser and Air Piping Replacement	\$36 ⁽⁴⁾	0 0	0 0	0 -626,000 ⁽⁵⁾	0 -\$134,000	0 0	0 0	Project includes new PACs, air diffusers, MLR, and biotrickling filters for odor control. Capital cost reflects addition of aeration grid control valves, air distribution piping, PAC control strategy, foam control, and sludge blanket indicators.
	Settleability Improvements (Densification and Surface Wasting)	\$9	2.7 ⁽⁶⁾	19.1	295,000	\$63,000	-97,000 ⁽⁷⁾	-\$194,000	Improved process stability and filter performance during wet weather.
	Stability Improvements (DO Control and Nutrient Monitoring; and SRT Control and Sludge Blanket Control)	\$4	0.7 ⁽⁸⁾	19.8	-173,000	-\$37,000	0	0	Improved biological nutrient removal performance and less variability.
	MLR Upgrade	--	0.6 ⁽⁹⁾	20.4	219,000	47,000	0	0	Project includes adding MLR pumping to provide additional capacity.
Tertiary	Filter Backwash Equalization	\$4	0	20.4	0	0	0	0	Provide 150,000-gallon backwash flow FE tank.
	Disinfection Optimization	\$18	0	20.4	0	0	0	0	Project includes addition of a new CCT tank and new effluent weir upstream of the existing effluent weir. Does not add capacity but is needed for CT compliance.
Total Rehabilitation and Optimization Projects⁽¹⁰⁾		\$100	5.6	20.4	-397,000	-\$85,000	-97,000	-\$194,000	

Notes:

Abbreviations: kWh - kilowatt-hour; lb - pound.

- (1) Based on August 2023 dollars using an ENR ranking of 15179 for Los Angeles.
- (2) Calculated using March 2023 from U.S. Energy Information Administration (EIA) data for California (\$0.2131/kWh).
- (3) \$2.00/lb active emulsion polymer.
- (4) Sanitation Districts estimate from DMS-#6539317-v6-LBWRP (August 2022).
- (5) Achieving standard oxygen transfer efficiency of 29 percent.
- (6) Achieving 90th percentile SVI of 100 mL/g.
- (7) Assuming emulsion polymer usage reduction by 100 percent from 2 mg/L.
- (8) Operation at 4.5 days aerobic SRT instead of 5.0 days.
- (9) Operating anoxic zone in Pass 3 as aerobic.
- (10) Does not include other rehabilitation needs identified by Sanitation Districts. Many projects are being undertaken by Sanitation Districts staff.

4.2.4 Capacity Summary After Phase 1

A summary of the projected future flows from TM 3 is shown in Table 4.8 for average annual and peak wet weather conditions. Note that the PWWF projection for the filters is slightly lower than the projection for the secondary system. As discussed in TM 3, Title 22 requirements must be met for all the effluent but some allowances can be made for non-compliance when the reuse demand is low or nonexistent during extreme storm events. Sanitation Districts intend to make recycled water available at all times; however it is impractical, not cost-effective, and wasteful of available space at the treatment plants to design facilities to meet Title 22 requirements for rare, extreme storm events. Therefore, the filters are sized to permit reuse during most but not all wet weather events. Deliveries to recycled water users cease during the rare occurrence of filter bypass.

Table 4.8 2050 Flow Projections Summary From TM 3

Flow Condition	2050 Projected Flow, mgd	Peaking Factor (2015 - 2016, 2020 - 2022)	Original Flow Design Basis
AADF	15.9	--	25.0
PDWF	27.5	1.73	34.0
PWWF - Secondary	38.2	2.40	60.0
PWWF - Filters	33.5	2.10	--

Notes:

Abbreviations: PDWF - peak dry weather flow.

Capacity after implementation of all optimization elements in Phase 1 is summarized in Table 4.9. This table compares the optimized capacity to the 2050 projections presented in Table 4.8 and identifies where capacity limitations or deficits would exist after Phase 1 is implemented. There are no capacity deficits after Phase 1 as indicated in Table 4.9. Unit process capacity of the optimized WRP and the targets for buildout are also illustrated on Figure 4.10.

Table 4.9 Capacity Summary After Phase 1

Unit Process	Controlling Condition	Capacity, mgd	Buildout Capacity Required in 2050, mgd	Capacity Deficit, mgd ⁽¹⁾
IPS	PWWF	84 ⁽²⁾ - 114 ⁽³⁾	38.2	--
PSTs	PWWF	60.0	38.2	--
	AADF	25.0	15.9	--
Secondary Process ⁽⁴⁾	PWWF	49.0	38.2	--
	AADF	20.4	15.9	--
Filters	PWWF	49.0	33.5	--
	AADF	20.4	15.9	--

Unit Process	Controlling Condition	Capacity, mgd	Buildout Capacity Required in 2050, mgd	Capacity Deficit, mgd ⁽¹⁾
Effluent Filter Pump Station	PWWF	34.1 ⁽²⁾ - 46.5 ⁽³⁾	38.2	--
Disinfection	PDWF	41.3 ⁽⁵⁾	27.5	--
	AADF	23.9 ⁽⁶⁾	15.9	--

Notes:

- (1) Capacity deficit = Buildout capacity required in 2050 - Total Rated Capacity.
- (2) Based on firm capacity (one pump out of service).
- (3) Based on total capacity (all pumps in service).
- (4) Based on implementing all secondary process optimization, which allows operation at an aerobic SRT of 4.5 days, achieving SVI of 100 mL/g, and converting anoxic zone in Pass 3 to aerobic. Results in an operating MLSS concentration in Pass 4 at maximum month ranging from 3,200 to 3,400 mg/L. Assuming PWWF/AADF peaking factor of 2.4.
- (5) Based on CCTs meeting Title 22 requirements of 450 mg-min/L CT.
- (6) Calculated using a peaking factor (PDWF/AADF) of 1.73.

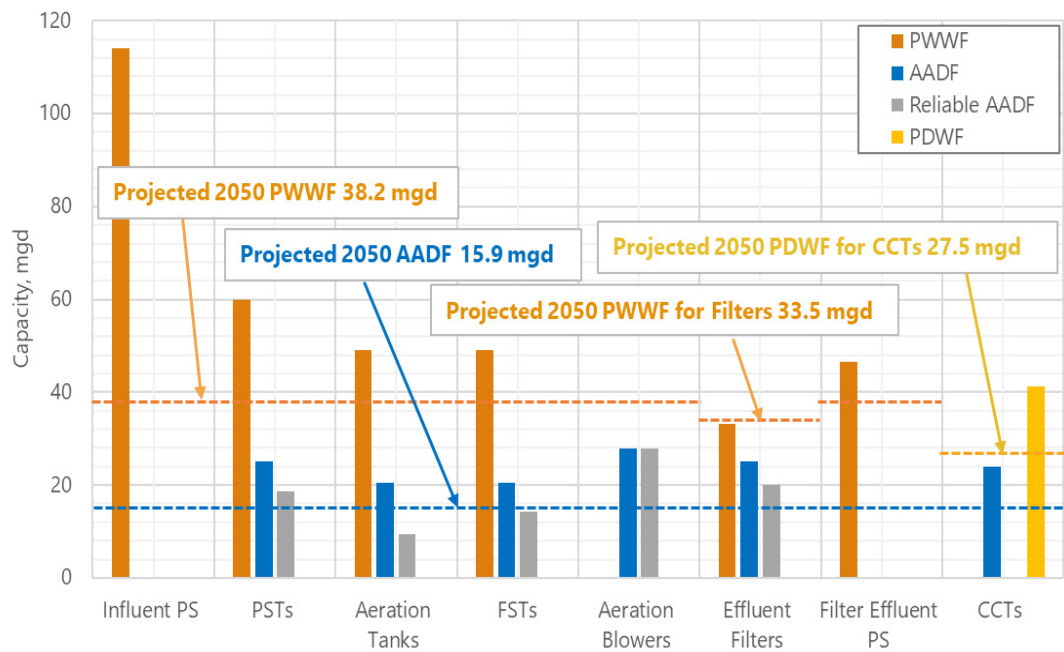


Figure 4.10 Optimized Capacity

4.2.5 Trigger Plots

Phase 2 capacity expansion will not be needed for LBWRP after implementation of all optimization projects as indicated in Table 4.7. However, it is still recommended that the Sanitation Districts perform a capacity evaluation after all the optimization projects are implemented.

A trigger plot comparing the projected annual average flows, current secondary capacity and optimized secondary capacity is shown on Figure 4.11. There will be additional capacity beyond the projected AADF for the planning period of 2050 assuming all optimization projects are implemented as shown on Figure 4.11. Therefore, the optimization projects can be phased for the planning period. The recommendations for phasing are described in Section 4.3.2.

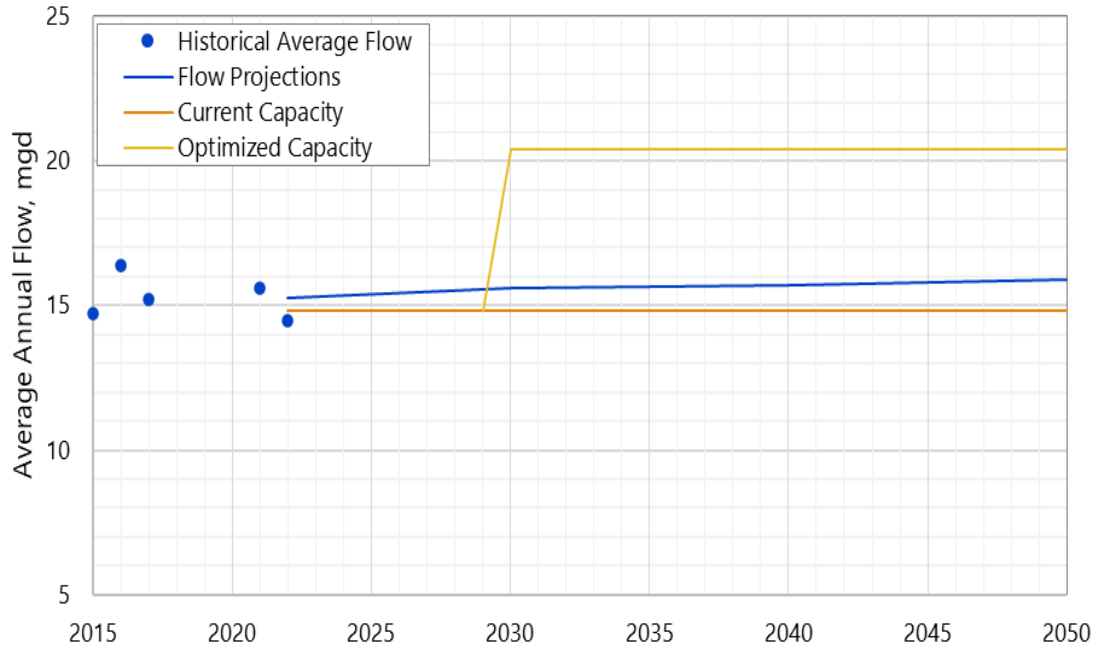


Figure 4.11 Secondary Process Trigger Plot

The trigger plot comparing PDWF projections, current CCT capacity, and optimized CCT capacity is shown on Figure 4.12. No capacity deficit was identified with respect to the available volume of the CCTs. However, due to operational challenges there were instances where the effluent did not meet 450 CT which will be a requirement when future regulations are implemented (i.e., the new WRR permit is issued). The addition of a new CCT provides additional disinfection capacity, redundancy, and mitigates operational challenges.

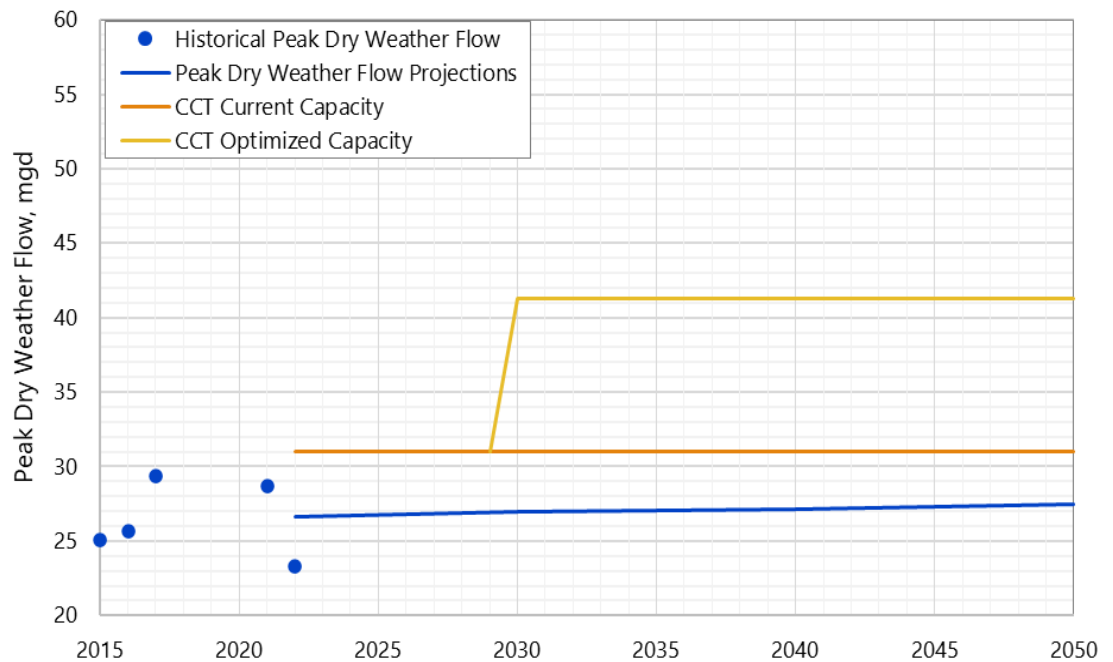


Figure 4.12 Tertiary Process Trigger Plot

4.2.6 Phasing of Optimization Projects

It is not necessary to implement all optimization projects in Phase 1 (Rehabilitation and Optimization) because capacity deficit is minor for the planning period. The Sanitation Districts can implement some of the optimization projects in Phase 1 which allows capacity recovery to achieve the 2050 projected flows. The remaining optimization projects can be implemented in later phases when additional capacity or limit of technology is required. There are two feasible phasing options for the projects to be implemented in Phase 1:

- Addition of an FE Tank in Phase 1 - This option allows the Sanitation Districts to improve overall performance of the LBWRP as well as provide additional secondary treatment capacity by reducing wet weather flow peaks. With this option, the settleability and stability improvements can be implemented in later phases when additional capacity or meeting stringent nutrients limitations is required.
- Settleability and Stability Improvements - This option provides the LBWRP additional secondary capacity needed for the planning period. With this option, the addition of the FE tank can be implemented in later phases when additional capacity or meeting stringent nutrients limitations is required.

After considering both the phasing options and subsequent discussion with the Sanitation Districts, it is recommended that the implementation of the first option be considered in Phase 1. The addition of an FE tank in Phase 1 has more overall benefits on the process performance and would help mitigate operational issues downstream of the secondary process for filtration and disinfection. Regardless of the phasing option, the remaining optimization projects including PAC and diffuser replacement, MLR upgrade, filter backwash equalization, and CCT modifications should be implemented in Phase 1 for improved performance and operation.



Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 5
LONG BEACH WATER RECLAMATION
PLANT ALTERNATIVES AND
RECOMMENDATIONS

FINAL | January 2025





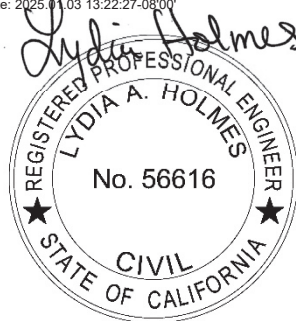
**LOS ANGELES COUNTY
SANITATION DISTRICTS**
Converting Waste Into Resources

Los Angeles County Sanitation Districts
JOS WRP Master Facilities Plan

Technical Memorandum 5
LONG BEACH WATER RECLAMATION
PLANT ALTERNATIVES AND RECOMMENDATIONS

FINAL | January 2025

Digitally signed by Lydia A. Holmes
Contact Info: Carollo Engineers, Inc.
Date: 2025.01.03 13:22:27-08'00'



Contents

Technical Memorandum 5 - Long Beach Water Reclamation Plant Alternatives and Recommendations

5.1 Introduction and Summary	5-1
5.2 Screening of Treatment Options	5-3
5.2.1 Temperature Reduction Options	5-5
5.2.2 Nutrient Removal	5-11
5.2.3 Summarizing the Selected Options	5-14
5.3 Alternative Development	5-14
5.4 Cost Estimate	5-17
5.4.1 Power, Chemical, and Consumables Costs	5-17
5.4.2 Life Cycle Costs	5-18
5.5 Engineer's Recommendation and Summary of all Phases	5-18

Appendices

Appendix 5A	Summary of Nutrient Removal Technologies
Appendix 5B	Screening Results

Tables

Table 5.1	Recommended Projects for Phases 3 and 6	5-2
Table 5.2	Current Concentration and Future Limits	5-4
Table 5.3	Screening Criteria Description	5-4
Table 5.4	Temperature Screening Results	5-7
Table 5.5	Phase 6 Screening Evaluation	5-13
Table 5.6	Selected Technology Summary	5-14
Table 5.7	Project Costs	5-17
Table 5.8	Cost Assumptions	5-18
Table 5.9	Power, Chemical, and Consumables Cost, in millions	5-18
Table 5.10	Life Cycle Cost	5-18
Table 5.11	Summary of Recommended Projects	5-20

Figures

Figure 5.1	Regulatory Timeline and Phased Approach to Compliance	5-3
Figure 5.2	NPDES Discharges and Reuse Flows	5-5
Figure 5.3	Phased Alternative with Timeline	5-15
Figure 5.4	Site Layout	5-16

Abbreviations

AADF	annual average daily flow
AGS	aerobic granular sludge
AWT	advanced wastewater treatment
B&B	biointegrity and biostimulatory
B&C&B	biostimulations, cyanotoxins, and biological conditions
BNR	biological nutrient removal
Carollo	Carollo Engineers, Inc.
CIP	capital improvements projects
DO	dissolved oxygen
F	Fahrenheit
FE	flow equalization
JOS	Joint Outfall System
LBWRP	Long Beach Water Reclamation Plant
LCWRP	Los Coyotes Water Reclamation Plant
MBR	membrane bioreactor
MFP	Master Facilities Plan
MG	million gallons
mg/L	milligrams per liter
mgd	million gallons per day
MLR	mixed liquor recirculation
NPDES	National Pollutant Discharge Elimination System
NPV	net present value
O&M	operations and maintenance
PAC	process aeration compressor
PFAS	per- and polyfluoroalkyl substances
Sanitation Districts	Los Angeles County Sanitation Districts
SCADA	supervisory control and data acquisition
SJCWRP	San Jose Creek Water Reclamation Plant
SRT	solids retention time
TM	technical memorandum
TOC	total organic carbon
Warren Facility	A.K. Warren Water Resource Facility
WRP	water reclamation plant
WWTP	wastewater treatment plant

Technical Memorandum 5

LONG BEACH WATER RECLAMATION PLANT ALTERNATIVES AND RECOMMENDATIONS

5.1 Introduction and Summary

The Los Angeles County Sanitation District (Sanitation Districts) operate the A.K. Warren Water Resource Facility (Warren Facility) and six water reclamation plants (WRPs) in the joint outfall system (JOS). The WRPs include the Long Beach WRP (LBWRP), the Los Coyotes WRP (LCWRP), the Pomona WRP, the San Jose Creek WRP (SJCWRP), and the Whittier Narrows WRP. The sixth WRP, the La Cañada WRP, does not have a National Pollutant Discharge Elimination System (NPDES) permit and is excluded from this Master Facilities Plan (MFP). The upstream treatment plants are connected by more than 1,400 miles of joint outfalls, interceptors, and Sanitation Districts' trunk sewers to the Warren Facility located in Carson. The collection system capacity is insufficient for peak dry weather flow bypass of LBWRP. To assist in the planning for future uses and permit compliance, the Sanitation Districts is preparing MFPs for the (1) SJCWRP, (2) LBWRP and LCWRP, and (3) Pomona WRP and Whittier Narrows WRP. The MFP for the LBWRP SJCWRP will be prepared as part of this effort. Preparation of MFPs for the remaining WRPs will be completed as part of future efforts, continuing the process based on the technical memoranda developed during this effort.

Technical Memorandum (TM) 2 - Regulatory Review and TM 3 - Existing Facilities, Performance, and Capacity Assessment presented the future compliance and capacity requirements for LBWRP. TM 4 - Gaps and Immediate Needs presented the potential projects for optimizing the LBWRP to achieve better performance and to increase capacity. This TM evaluates treatment processes and provides alternatives to meet future regulatory compliance.

A phased approach was recommended to better achieve the different regulatory timelines presented for each constituent for SJCWRP. This approach is also recommended for LBWRP as it allows the Sanitation Districts to adjust the plan to meet future capacity and regulatory requirements as necessary. Phases from SJCWRP included:

- Phase 1 includes optimization projects to improve performance.
- Phase 2 includes expansion projects to meet the future flows.
- Phase 3 includes treatment for effluent temperature reduction by 2031.
- Phase 4 includes adding treatment processes to meet total organic carbon (TOC) limit by 2030.
- Phase 5 includes adding treatment processes to meet per- and polyfluoroalkyl substances (PFAS) limits by 2034.
- Phase 6 includes adding treatment processes to meet biostimulations, cyanotoxins, and biological conditions (B&C&B) nutrient limits by 2041.

As discussed in TM 4, LBWRP does not need Phase 2 as the recommended Phase 1 optimization projects will restore sufficient capacity to meet future flows. Phases 4 and 5 for SJCWRP evaluated treatment for TOC and PFAS removal to meet future regulations for potable reuse via direct spreading at the Montebello Forebay. This reuse scenario does not apply to LBWRP. As discussed in TM 2 - Regulatory Review, currently LBWRP effluent has three uses: discharge to surface water, indirect potable reuse - via advanced treatment at offsite facilities, and non-potable reuse. Therefore, technologies for Phases 4 and 5 for potable reuse via direct spreading are not evaluated. However, Phases 3 and 6 for temperature and nutrients still apply for LBWRP for the portion of the flow that is discharged to Coyote Creek, a tributary to the San Gabriel River.

Table 5.1 provides a summary of the recommended projects for Phase 3 and Phase 6 based on the findings of this TM. Phases 1 and 2 were discussed in TM 4 and this TM addresses Phases 3 and 6. The technologies to meet the regulatory limits are described in detail in Section 5.2. Figure 5.1 presents the regulatory timeline developed based on input from the Sanitation Districts' Reuse and Compliance Section. Figure 5.1 includes the time anticipated to plan, design, and construct the new treatment facilities. Some of the recommended Phase 1 projects have been started by the Sanitation Districts' staff in 2023.

Table 5.1 Recommended Projects for Phases 3 and 6

Need/Regulation	Timeline	Recommendations/Technology	Project Cost, \$ millions ⁽¹⁾	Project Description
Temperature	Phase 3 - 2031	Cooling Towers or Cooling Towers with Chillers	\$2	Install 6.4 mgd cooling towers.
Nutrient Removal (B&C&B)	Phase 6 - 2041	4 Stage Bardenpho	\$10	Modify the existing step feed trains to 4-Stage Bardenpho with carbon addition.

Notes:
 Abbreviations: mgd - million gallons per day.
 (1) Cost in 2023 dollars.

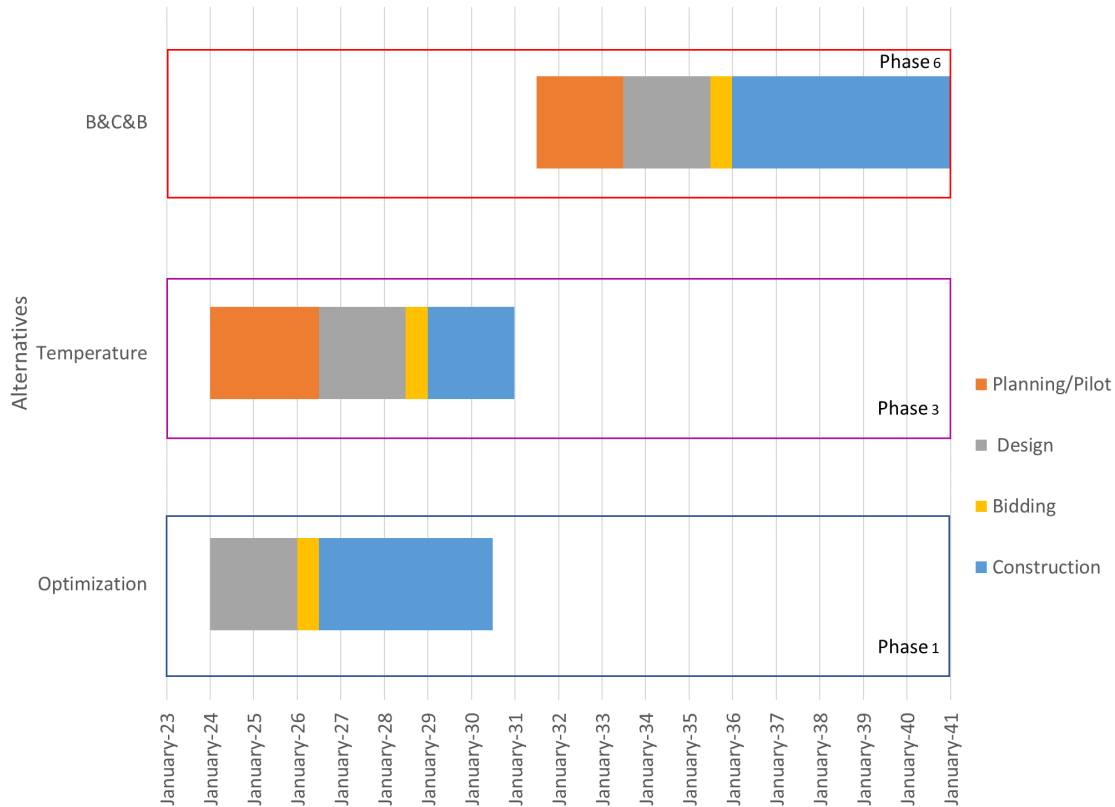


Figure 5.1 Regulatory Timeline and Phased Approach to Compliance

5.2 Screening of Treatment Options

As part of this TM, a screening of viable technologies was performed for each of the constituents of concern identified for future compliance issues. Table 5.2 presents the current effluent concentrations provided by the Sanitation Districts and anticipated future limits developed in TM 2.

Table 5.2 Current Concentration and Future Limits

Constituent	Current Effluent Concentration	Potential Future Effluent Limits ⁽¹⁾
Effluent Temperature	62 to 86 degrees F	80 degrees F
Total Nitrogen	7 mg/L	3 mg/L

Notes:

Abbreviations: F - Fahrenheit; mg/L - milligrams per liter.

(1) Concentrations are provided in TM 2.

For each of the constituents of concern, alternative treatment processes were identified and screened down to a list of viable options. The screening analysis is discussed in the following sections for the reduction of temperature and B&C&B (nutrients). The screening process for the treatment options is based on comparison of options against a set of criteria that differ slightly between temperature reduction and B&C&B as shown in Table 5.3.

Table 5.3 Screening Criteria Description

Screening Criteria ⁽¹⁾	Criteria Description for:		Metric
	Temperature	B&C&B	
Ability to Meet Regulations	Complies with near-term regulations and can be adapted to meet anticipated regulations.		Pass/Fail
Technology Maturity & Risk (at this size)⁽¹⁾	Proposed technology/approach has at least one proven installation in the US.	Proposed technology/approach has at least one installation with a capacity of 20 mgd or greater, with at least 1 year of successful operation (within the last 10 years) at 90 percent capacity. Each technology/approach is categorized by the following groups and screened accordingly: <ul style="list-style-type: none"> • Established. • Emerging. • Embryonic. 	Pass/Fail
Ease of Permitting	Not Used.	Technology has been permitted at a WWTP.	Pass/Fail
Cost	Proposed technology is cost-effective.	Not used.	Pass/Fail/Unknown
Site Constraints	Structures, equipment, etc., fit within the existing WWTP boundaries.		Pass/Fail
Independent Operations	Facilities can be fully operated by the Sanitation Districts staff (i.e., contract operations by independent entities are not required).		Pass/Fail

Notes:

Abbreviations: WWTP - wastewater treatment plant.

(1) To facilitate screening, technologies will be grouped into three categories: embryonic, emerging, and established.

5.2.1 Temperature Reduction Options

A comprehensive list of cooling alternatives were identified and the alternatives were evaluated on their ability to meet the regulations, technology implementation, site constraints, cost, and operability. The NPDES permit requires a maximum daily effluent temperature of 80 degrees F, and the discharge shall not change the river temperature by more than 5 degrees. However, there is an interim receiving water limitation that at no time shall the warm freshwater habitat designated waters be raised above 86 degrees F as a result of waste discharge. Because the WRPs cannot immediately comply with the new limitations, the Regional Water Board has allowed the Sanitation Districts the interim limit and a 10-year compliance schedule which includes time to conduct studies to evaluate the temperature impacts in the watershed. For the purposes of this evaluation, we used the effluent limit of 80 degrees F as that is likely the conservative requirement starting in 2031.

A temperature evaluation for the LBWRP was performed and the projected high temperature was estimated based on future conditions under climate change in 2050. The historical effluent temperature has ranged from 62 degrees F to 86 degrees F. The projected high temperature under climate change for 2050 is 89.5 degrees F. Therefore, comparing this projected number to the NPDES permit limit of 80 degrees F, it is assumed that new facilities will be required to reduce the effluent temperature by 9.5 degrees F before discharging into Coyote Creek. The Sanitation Districts do not have a required minimum discharge requirement to Coyote Creek. On an annual basis approximately half of the flow from LBWRP is reused, but during the summer irrigation months a majority of the effluent is reused. Figure 5.2 shows the NPDES discharges and reuse flows for the evaluation period. The NPDES discharges are shown in grey colored bars and reuse flows are shown in yellow colored bars.

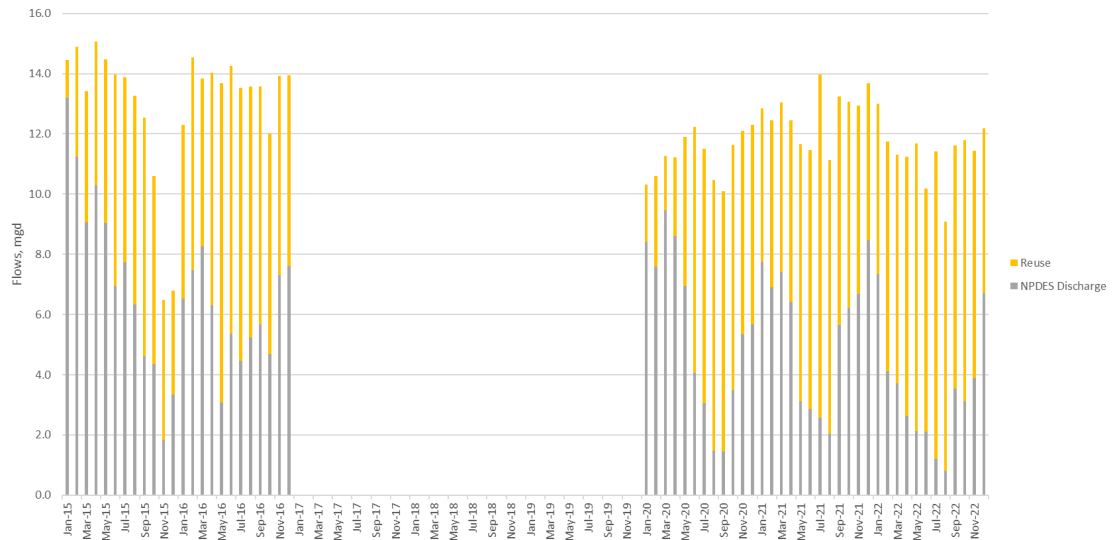


Figure 5.2 NPDES Discharges and Reuse Flows

The minimum effluent flows requiring temperature reduction were estimated by the difference between 2050 average projected flows (15.9 mgd) and contracted reuse flows (9.5 mgd). A comparison of the projected flows, temperatures and the reuse contracts found that temperature reduction would be needed for 6.4 mgd of LBWRP effluent.

The different temperature reduction technologies are listed in Table 5.4. There are several cooling alternatives that could meet the required limits; however, additional investigation is needed to determine how feasible they are for LBWRP. Reducing or eliminating discharge is the most feasible alternative to providing mechanical cooling but would require significant investment in storage to avoid discharges. Chillers and cooling towers are the most mature and proven technologies that can provide the required amount of cooling. For the cooling towers to work efficiently, the temperature difference between the effluent target temperature and ambient air temperature shall be at least 9 degrees F. This 9-degree-F differential requirement for cooling towers is met all the time for current temperatures and nearly all the time for future conditions under climate change. As the chillers require a significant amount of energy to run, cooling towers were selected as the effluent temperature reduction technology to be used for site planning and capital improvements projects (CIP) development in this TM. If the climate conditions change in the future, cooling towers could be operating at the edge of their efficiency and the cooling system may have to be retrofitted to add chillers. Cooling towers are a required part of chillers, and thus would be required either way.

The 6.4-mgd cooling tower footprint and location is shown in the layout in Section 5.3.3. Selection of the preferred temperature management option for implementation will consider the results of temperature and biological studies currently being conducted in the San Gabriel River Watershed as well as input from technical advisors and stakeholders. Implementation of the preferred option may be contingent on regulatory approval(s), such as those related to discharge permitting and basin planning.

Table 5.4 Temperature Screening Results

Classification	Examples	Screening					Justification Notes for Screening
		Ability to Meet Regulations	Technology Implementation	Site Constraint	Cost	Independent Operations	
Natural Heat Flow	Cooling with surface or groundwater using heat exchanger, Blending with deep groundwater water source, Geothermal cooling, Infiltration trenches, Aquifer storage and recovery	Fail	Unknown	Fail	Unknown	Fail	Requires groundwater permits, large space requirement, flow availability, temperature of groundwater unknown, unknown applications, cost-effectiveness unknown, cannot meet 9.5 degrees F drop.
Evaporative Cooling	Passive cooling wetlands/pond, Seasonal pond storage, Spray Pond Cooling System, Pipe in pond cooling, Hyporheic zone injection, Enhanced mixing of effluent and receiving stream	Fail	Unknown	Fail	Unknown	Fail	Large space requirement outside of plant site, unknown applications, cost effectiveness unknown, cannot meet 9.5 degrees F drop.
	Effluent discharge structure modifications	Fail	Unknown	Pass	Unknown	Fail	Unknown applications, cost effectiveness unknown, cannot meet 9.5 degrees F drop.
	Cooling Towers - Direct and Indirect	Pass	Pass	Pass	Pass	Pass	SELECTED only for CIP and Layout Planning Purposes. Known applications, proven technology.

Classification	Examples	Screening					Justification Notes for Screening
		Ability to Meet Regulations	Technology Implementation	Site Constraint	Cost	Independent Operations	
Source Control	Controls at commercial, retail, or industrial	Unknown	Unknown	Fail	Unknown	Fail	Unknown applications, unknown temperature control.
	Recovering heat in buildings or in large sewer trunks and interceptors in the service area	Fail	Unknown	Pass	Unknown	Fail	This concept is used where heat can be harvested from the sewer in cold environments. Not suitable for warm climates.
	Public outreach	Unknown	Unknown	Unknown	Unknown	Fail	Industrial and commercial sources not the cause.
Heat Pump/Energy Recovery	Energy recovery and reuse from effluent, Energy recovery and reuse in the collection system	Fail	Unknown	Pass	Unknown	Fail	Works in cold climates, unlikely to work in southern California.
	Internal heat load management (operational modifications)	Fail	Pass	Pass	Unknown	Fail	WRP treatment process is not causing 9.5 degrees F rise in temperature.
Mechanical Cooling	Chiller with closed loop cooling tower, Chillers using heat exchangers, Heat Pump	Pass	Pass	Pass	Fail	Pass	ALTERNATE. Expensive and energy intensive. Process is one of few proven to work on cooling effluent. Option if cooling towers are not feasible.

Classification	Examples	Screening					Justification Notes for Screening
		Ability to Meet Regulations	Technology Implementation	Site Constraint	Cost	Independent Operations	
Reducing Discharge	Modification of discharge location - alternative locations of the discharge, consolidation with other wastewater treatment facilities, Water Recycling measures within the facility	Unknown	Unknown	Pass	Unknown	Pass	Unknown temperature control, 100 percent recycling within the facility is not possible. No other discharge location identified.
	Reduction in the scale of the proposed discharge or activity, Seasonal or controlled discharge to minimize discharging options to minimize discharging during critical water quality	Unknown	Unknown	Pass	Pass	Pass	No viable options have been identified for reducing discharge. Unknown temperature control.
	Reclaimed water use/Land application/zero discharge	Unknown	Pass	Pass	Pass	Pass	Although LBWRP does not have minimum discharge requirements, not all of the effluent is currently reused. If LBWRP plans to reuse all of it's effluent, this option can be selected.

Classification	Examples	Screening					Justification Notes for Screening
		Ability to Meet Regulations	Technology Implementation	Site Constraint	Cost	Independent Operations	
In-plant Process Changes	Identifying processes that increase wastewater temperature, Using alternatives processes/technology for wastewater treatment, Using energy-efficient mechanical equipment to minimize losses due to friction	Fail	Fail	Pass	Unknown	Fail	WRP operation is not causing 9.5 degrees F rise in temperature.
Shading	Riparian vegetation, Shade options for River/creek	Unknown	Unknown	Fail	Unknown	Fail	Unknown temperature control, unknown applications, outside of plant site.
Others	Regulatory Solutions	Unknown	Unknown	Unknown	Unknown	Unknown	To be discussed with the regulatory agency.

5.2.2 Nutrient Removal

The future effluent limits for total nitrogen and total phosphorous are much lower than what the existing biological treatment processes can achieve. In addition to the optimization projects (Phase 1) that will improve the overall performance, alternatives adding a physical treatment process, post-secondary denitrification, activated sludge processes, intensification, or emerging technologies are required to meet future regulations. A screening of technologies and approaches to meet this potential future limit were performed based on the criteria shown in Table 5.3. Description of each technology is in Appendix 5A and screening results are in Appendix 5B. The summary of the screening results is shown as follows:

- Alternative 1 - Optimized Step Feed with Physical Treatment Process for N Removal: Reverse Osmosis is selected as it can remove up to 90 percent of total nitrogen based on the available studies.
- Alternative 2 - Optimized Step Feed with Post-Secondary Denitrification: Biologically active filter is selected as it can reduce the total nitrogen concentration to 3 mg/L.
- Alternative 3 - Activated Sludge Biological Nutrient Removal (BNR): 4 stage Bardenpho process with carbon addition is selected. The estimated treatment capacity of the Bardenpho train is approximately 20 percent less than step feed but the optimization projects can provide the additional capacity.
- Alternative 4 - Established Intensification Technology: Membrane bioreactor (MBR) is selected as it provides nutrient removal as well as additional secondary and tertiary capacity.
- Alternative 5 - Emerging Technology: Flow-Through Aerobic Granular Sludge (AGS) is selected as a promising new technology. However, it will require pilot testing to further evaluate this option as the process is not proven at full scale.

The screened technologies were evaluated as shown in Table 5.4. Each screened technology is given a Superior (+), Neutral (0), or Inferior (-) rating for each category. The overall rating for the alternative is the sum of the rating in each category. A description of each criterion follows:

- Operationally Complex: Operational considerations include flexibility, reliability, simplicity, and operator familiarity. In general, the Sanitation Districts' preference is to implement systems that provide operational flexibility while minimizing any associated cost premium. Simplicity of the system depends on the number of system components and the location and interactions of these components. Typically, simple systems result in fewer operations and maintenance (O&M) issues and lower costs. Facilities that are consistent with existing systems and processes were rated more favorably. Alternatives that incorporate systems that utilize less proven technologies or are significantly different or considerably more complex than existing systems were rated less favorably.
- Space Requirements: Implementing new technologies requires space and space available at LBWRP is limited. Some alternatives require modifications that can fit inside the existing tank, whereas some alternatives will require additional land. The alternatives that minimize the use of valuable space were rated favorably.

- Capacity per train: Water is a valuable commodity in Southern California. The LBWRP produces high-quality effluent that can be reused in a variety of ways. The alternatives that can provide more capacity per train or produce more effluent are rated favorably.
- Project Cost: In general, the most cost-effective approach is the approach that delivers the necessary level of service at the least cost. Cost effective alternatives are rated favorably.

From Table 5.4, the 4-stage Bardenpho with carbon addition alternative has the highest overall rating. Supplemental carbon dosing will be added to the second anoxic zone to allow denitrification (conversion of nitrates to nitrogen gas). Significant structural upgrades including but not limited to demolishing some of the existing internal tanks walls, building new walls for the different zones, core cutting wall for MLR pump, etc. will be required to modify the existing step feed train to 4-stage Bardenpho, but it is easier to operate than the other technologies in Table 5.4. As most of the modifications will be done inside the existing step feed train, additional space will not be required but the effluent production for the Bardenpho train will be 20 percent less than the step feed train, requiring additional optimization projects to restore the capacity to meet 2050 flow projections. Comparing the project cost with other technologies selected to meet the Phase 6 regulations, 4-stage Bardenpho is the most cost-effective option. Therefore, it is selected to meet the future nutrient removal limits.

Table 5.5 Phase 6 Screening Evaluation

Alternative	Technology	Screening Criteria				
		Operationally Complex	Space Requirements	Capacity (per train)	Project Cost	Overall Rating
Optimized Step Feed with Physical Treatment Process for N Removal	Reverse Osmosis	-	-	-	-	-4
Optimized Step Feed with Post-Secondary Denitrification	Biologically Active Filter	0	-	0	0	-1
Activated Sludge BNR	4 stage Bardenpho process with carbon addition	+	+	-	+	+2
Established Intensification Technology	MBR	-	+	+	-	0
Emerging Technology	Flow-Through AGS	Needs Piloting	Needs Piloting	Needs Piloting	Needs Piloting	Not Applicable

5.2.3 Summarizing the Selected Options

Table 5.6 summarizes all the selected technology to meet the temperature and nutrient removal regulations and the recommended phases for implementation and timing. These technologies are then combined into alternatives which are discussed in the next section.

Table 5.6 Selected Technology Summary

Regulation	Timeline	Technology
Temperature	Phase 3 - 2031	Cooling Towers or Cooling Towers With Chillers ⁽¹⁾
Nutrient Removal	Phase 6 - 2041	4-Stage Bardenpho

Notes:

(1) This selection is only for CIP and layout planning purposes but the final selection of technology is dependent upon other efforts being undertaken by the Sanitation Districts. Cost provided in this TM is for 6.4 mgd cooling towers.

5.3 Alternative Development

From TM 3 the existing capacity of the LBWRP plant is limited by the secondary process to 14.8 mgd. As discussed in TM 4, optimizing the step feed process with settleability improvements, solids retention time (SRT) control, aeration control, mixed liquor recycle, and EQ tank can provide additional capacity up to 20.4 mgd. With all the optimization projects, additional capacity will not be required to meet the 2050 projected annual average daily flow (AADF) 15.9 mgd.

As described in TM 4, there are two possible phasing options for optimization projects: Option 1 - Implementing a flow equalization (FE) tank in Phase 1 and Implementing Settleability and Stability Improvements in the later phases, and Option 2 - Implementing Settleability and Stability Improvements in Phase 1 and Implementing an FE tank in later phases. Either one of these options can be implemented in Phase 1. When 4-Stage Bardenpho is implemented to meet Phase 6 regulations, the secondary capacity of the LBWRP will be reduced by 20 percent. To return the plant to capacity, the remainder of the Phase 1 projects can be implemented at that time. Figure 5.3 shows the phased alternative with timeline. Figure 5.4 shows the site layout with the improvements required for all the phases.

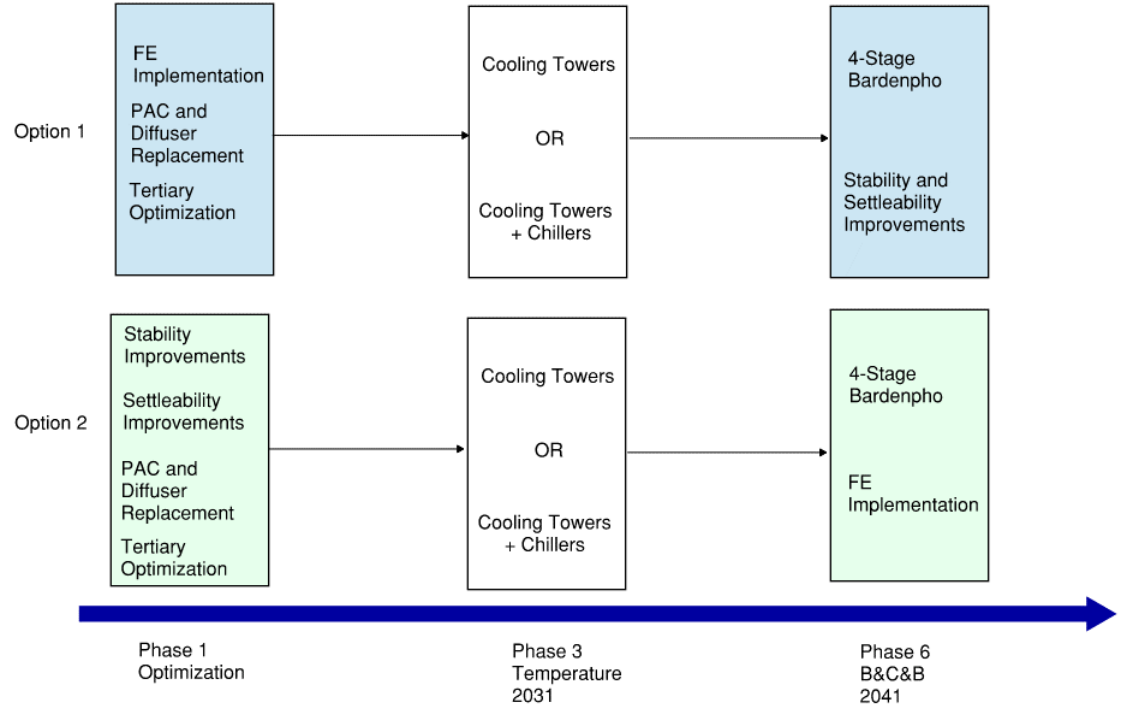


Figure 5.3 Phased Alternative with Timeline



LEGEND	
①	PAC Replacement and Stability Improvements.
②	Flow Equalization Tank (225' x 100')
③	Densification Facilities (Each 30' x 20').
④	150,000 gallon tank for filter backwash equalization.
⑤	CCT Modifications.
⑥	Cooling Tower (50' x 50').
⑦	Upgrade to 4- stage Bardenpho.

Figure 5.4 Site Layout

5.4 Cost Estimate

This section presents a comparison for the project costs, O&M costs, and life cycle costs in 2023 dollars.

A Class 5 cost estimate was prepared for each alternative. A Class 5 cost estimate has an expected accuracy range of -20 to -50 percent and +30 to +100 percent for the low and high ranges, respectively. The cost presented in Table 5.7 were developed using Carollo Engineers, Inc.'s (Carollo's) Cost Estimation database and past similar projects. Note that these costs are developed only for the purposes of alternative comparison.

The following assumptions were made to estimate the project cost:

- Subtotal Construction Cost is calculated as the sum of the following:
 - Total Direct Cost.
 - General Conditions, Bonds, Insurance = 15 percent of Total Direct Cost.
 - Sales Tax = 7.8 percent based on 50 percent of Total Direct Cost.
 - Contractor Overhead, Profit & Risk = 15 percent of total Direct Cost.
- Total Construction Cost is calculated as the sum of the following:
 - Subtotal of Construction Cost.
 - Change Order Allowance = 10 percent of Subtotal of Construction Cost.
- Total Project Cost is calculated as the sum of the following:
 - Total Construction Cost.
 - Engineering = 15 percent of total Construction Cost.
 - Legal and Admin Fees = 5 percent of total Construction Cost.
 - Environmental Permitting = 2 percent of total Construction Cost.
 - Construction Management = 7 percent of total Construction Cost.
 - Warranty = 1 percent of total Construction Cost.

The project cost for each phase is different dependent on which option is selected but the overall cost for both options will be the same when all the phases are implemented.

Table 5.7 Project Costs

Alternative	Phase 1 Optimize (in millions)	Phase 3 - Temperature ⁽²⁾ (in millions)	Phase 6 - B&B (in millions)	Total (in millions)
Option 1	\$87	\$2	\$23	\$112
Option 2	\$71	\$2	\$39	\$112

Notes:

Abbreviations: B&B - biointegrity and biostimulatory.

(1) Cost in 2023 dollars.

(2) Project Cost for 6.4 mgd cooling towers.

5.4.1 Power, Chemical, and Consumables Costs

Annual costs were developed for power, chemical, and consumables for both options. As both options have the same process improvements, the total O&M cost is the same. The assumptions made when developing these costs are shown in Table 5.8. The O&M costs are shown in Table 5.9. Labor costs are not included in the estimate.

Table 5.8 Cost Assumptions

Assumptions	Units	Cost
Electrical Cost ⁽¹⁾	\$/kilowatt-hour	\$0.2131
Chemicals		
Methanol ⁽²⁾	\$/gallon	\$2.0
Consumables		
Diffusers ⁽³⁾	\$/diffuser	\$15

Notes:

- (1) From US Energy Information Administration - March 2023.
- (2) Assumed from previous projects.
- (3) Assumed cost for diffuser replacement.

Table 5.9 Power, Chemical, and Consumables Cost, in millions

Alternative Description	Annual Power	Annual Chemical	Annual Consumables	Total
Optimization + Cooling Towers +4-Stage Bardenpho	\$1.0	\$0.8	\$0.06	\$1.9

Notes:

- (1) Cost in 2023 dollars.

5.4.2 Life Cycle Costs

Table 5.10 presents the net present value (NPV) costs for both options. Labor costs are not included in the life cycle costs.

Table 5.10 Life Cycle Cost

Alternative Description	NPV, millions ⁽¹⁾
Option 1	\$136
Option 2	\$132

Notes:

- (1) Assumptions used in NPV: Analysis Period= 2023- 2050. Escalation Rate = 3%. Discount Rate = 2%. Cost in 2023 dollars. Time frame assumed - Construction of Optimization projects 2025 - 2029, Construction of Cooling towers 2029-2030, Construction of Bardenpho 2039-2040. Optimization O&M costs are from 2030-2050, Cooling tower O&M cost are from 2031-2050, Bardenpho O&M costs are from 2041-2050.

5.5 Engineer's Recommendation and Summary of all Phases

The alternatives analysis began with identifying all the alternatives that would improve performance, add capacity, and meet future regulations. A phased approach based on the regulatory timeline was recommended to meet current and future needs.

Phase 1 optimization projects are recommended to provide capacity and improve performance. After all the optimization projects are implemented, the plant will be able to meet projected 2050 capacity and no additional capacity will be needed. As described in Options 1 and 2, optimization projects can be phased with either implementing FE tank in Phase 1 followed by settleability and stability improvements in the future or implementing settleability and stability improvements in Phase 1 and FE tank in the future. While implementing FE tank in Phase 1 is more expensive in the near term, it will provide early benefits for the entire treatment plant. It will allow storing wet weather flows and improving storm water management. This optimization

project will reduce the peak wet weather peaking factor from 2.40 to 1.85 and reduce peak energy demands. Recycled water production will also increase, and secondary and tertiary performance will improve. SJCWRP FE tank has been in operation since Fall 2020 and has observed similar benefits. The Sanitation Districts have also identified a sewer relief project for the downstream sewer.

The current summer effluent wastewater temperature at the LBWRP is higher than the maximum daily limit of 80 degrees F. Therefore, cooling towers were the selected effluent temperature reduction technology in Phase 3 after several technologies were screened based on ability to meet the regulations, technology implementation, site constraints, cost, and operability. The cooling towers will provide the required temperature reduction in the effluent for 6.4 mgd (the difference between projected flow and contracted reuse flow) before it is discharged to Coyote Creek.

To meet the B&C&B limits in Phase 6, several technologies were evaluated as shown in Section 5.2.2. When the B&C&B regulations are implemented, the existing step feed trains can be modified to 4-stage Bardenpho process. Although this modification will reduce the capacity of the secondary process by approximately 20 percent, implementing stability and settleability projects from Phase 1 along with Bardenpho process will provide adequate capacity required to meet the projected 2050 AADF.

The overall benefits for Option 1 and Option 2 after all the phases are implemented will be the same. However, implementing FE tank in Phase 1 has overall better benefits. Therefore, the recommendation for the Sanitation Districts is to proceed with Option 1 and construct the FE tank in Phase 1 followed by the cooling towers in Phase 3 and Bardenpho process with the remaining optimization projects in Phase 6. Table 5.11 summarizes all the projects recommended from Phase 1 through Phase 6.

Table 5.11 Summary of Recommended Projects

Area/Project	Brief Description of Scope	Estimated Capital Cost (\$ million) ⁽¹⁾	Estimated Annual O&M Cost (Power + Polymer Usage) (\$ million) ⁽¹⁾	Average Daily Flow Capacity Increase (mgd)	Currently on CIP? If so, current budget.	Implementation Schedule (needed w/in 5, 10, or 20 years?)	Further Evaluation Needed to Finalize Recommendation?
PHASE 1 - REHABILITATE AGING ASSETS AND OPTIMIZE PERFORMANCE							
FE							
New FE Facilities	A new FE tank with 2.5 MG volume and odor control station.	\$29	-0.024	1.6	Yes - \$20.5 million	5 years	Perform additional studies to design the tank. O&M costs are evaluated in terms of changes from the current usage and represent savings after the project will be implemented.
Secondary Optimization							
PAC Replacement	Upgrade existing PACs with new and more efficient turbo blowers operated on a single common air distribution system.	\$36	--	--	Yes - \$4.1 million	5 years	Project identification and cost estimate by the Sanitation Districts.
Diffuser and air piping Replacement ⁽²⁾	Replace the existing diffuser with higher efficiency diffusers. Replace air piping since the existing steel distribution piping within air channels exhibits some exterior corrosion.	--	-0.134	--	Yes	5 years	Project identification and cost estimate by the Sanitation Districts. Cost included in the PAC Replacement project.
MLR Pumping ⁽²⁾	Install MLR pumps to improve denitrification.	--	0.047	0.6	Yes	5 years	O&M costs are evaluated in terms of changes from the current usage. Cost included in the PAC Replacement project.
Densification and Surface Wasting	Install hydrocyclones to separate lighter biomass and denser biomass.	\$9	-0.131	2.7	No	5 - 15 years	Carollo Recommendation - Pilot hydrocyclones. O&M costs are evaluated in terms of changes from the current usage. Settleability improvements can be performed in Phase 1 or Phase 6.
DO Control and Nutrient Monitoring	Add nutrient monitoring to use the existing DO control - tracking nutrients will help both secondary and disinfection operation. Monitoring will include primary effluent ammonia, ammonia in aerobic zone, Nitrate and oxidation reduction potential in anoxic zones, Secondary effluent nitrate. Reprogram aeration system to tie to nutrient monitoring and DO - can be done with or without predictive control. Add model predictive control.	\$4	-0.037	0.7	No	5 - 15 years	Carollo Recommendation - Perform an aeration control study. O&M costs are evaluated in terms of changes from the current usage. Stability Improvements can be performed in Phase 1 or Phase 6.
SRT Control and Sludge Blanket Control	Monitor solids in aeration and maintain SRT by wasting. This can be done manually, and no additional instruments will be required. Automate SRT control by adding sludge blanket monitors and change total suspended solids monitors with programming in SCADA - in house or purchase commercially.	\$4	-0.037	0.7	No	5 - 15 years	None.
Tertiary Optimization							
Filter Backwash FE Tank	Provide 150,000-gallon backwash FE tank at LBWRP to reduce fluctuations in the flow swings and stabilize disinfection chlorine dosage.	\$4	--	0	No	5 years	Perform additional studies to design the tank.
New Chlorine Contact Tank		\$18	--	0	No	5 years	Perform additional studies to design the tank.
PHASE 3 - TEMPERATURE							
Temperature							
Effluent Cooling ⁽³⁾	Install 6.4 mgd cooling towers.	\$2	0.097	0	No	8 years	Study/Pre-design to select technology, sizing.

Area/Project	Brief Description of Scope	Estimated Capital Cost (\$ million) ⁽¹⁾	Estimated Annual O&M Cost (Power + Polymer Usage) (\$ million) ⁽¹⁾	Average Daily Flow Capacity Increase (mgd)	Currently on CIP? If so, current budget.	Implementation Schedule (needed w/in 5, 10, or 20 years?)	Further Evaluation Needed to Finalize Recommendation?
PHASE 6 - B&B							
AWT - Nutrients							
4 Stage Bardenpho ⁽³⁾	Modify existing step feed trains to 4-Stage Bardenpho configuration with methanol addition.	\$10	2.1	0	No	15 years	Perform additional studies prior to implementation.

Notes:

Abbreviations: AWT - advanced wastewater treatment; DO - dissolved oxygen; MLR - mixed liquor recirculation; PAC - process aeration compressor; SCADA - supervisory control and data acquisition.

(1) Based on August 2023 dollars using an *Engineering News-Record* of 15179 for Los Angeles.

(2) Cost included in PAC replacement project estimate.

(3) Project based on future regulations.

Appendix 5A
SUMMARY OF NUTRIENT REMOVAL
TECHNOLOGIES

Summary of Nutrient Removal Technologies

This appendix provides descriptions of nutrient removal technologies that were screened for further evaluation. Alternatives are grouped as:

- Physical Treatment Processes.
- Post-Secondary Denitrification.
- Activated Sludge Biological Nutrient Removal (BNR).
- Established Intensification Technology.
- Emerging Technology.

5A.1 Physical Treatment Process for N Removal

Reverse osmosis (RO), ion exchange, and carbon based activated treatment (CBAT) or advanced oxidation technologies are described in this section. All of these technologies rely on physical methods for removing nitrogen from the wastewater.

5A.1.1 Reverse Osmosis

RO membrane systems are designed to remove dissolved solids from water. Feed water is pumped through a semi-permeable membrane which acts as a barrier to the dissolved solids. The required operating pressure depends on the amount of dissolved solids in the feed and the membrane properties. The process can easily achieve 90 percent removal or more of soluble nitrogen compounds. Since the production (or recovery) of an RO membrane is typically 80 percent, systems are commonly designed with two or more stages where reject from the previous stage is concentrated in subsequent phases with excess water recovered to increase production. Advantages are that it generates very pure water with no particulate solids and very low in dissolved solids. Disadvantages are the high capital and operating cost associated with feed pumping, membrane cleaning, and brine disposal. RO systems also require membrane pretreatment with micro- or ultra-filtration to minimize fouling of the RO system.

Figure 5A.1 illustrates a typical process flow diagram for a two-stage RO system.

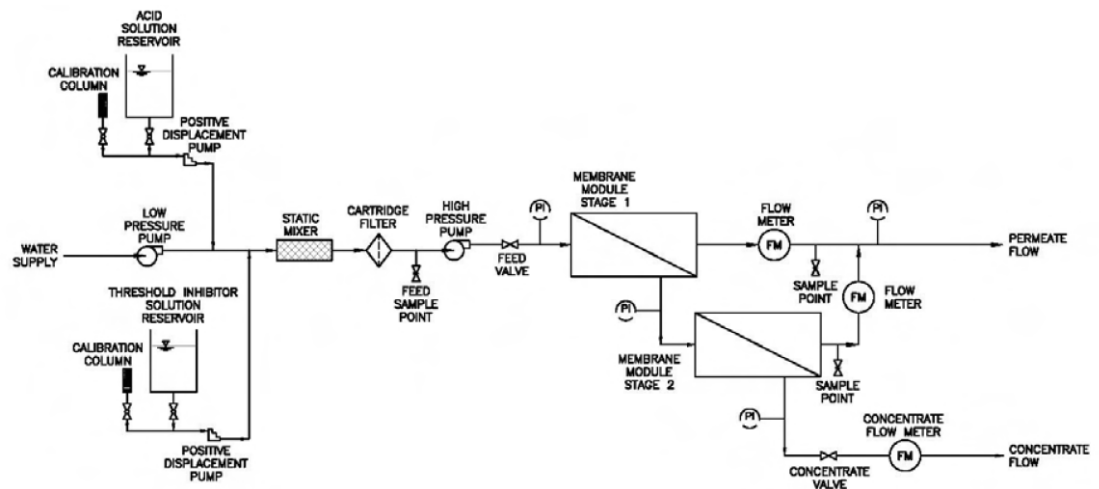


Figure 5A.1 Two-Stage RO Process Flow Diagram

5A.1.2 Ion Exchange

Ion exchange is a treatment process where dissolved ions are replaced by other more desirable ions of a similar electrical charge. The exchange usually takes place on a solid polymeric resin designed for the type of contaminant trying to be removed. Ion exchange is an effective technology for removal of nitrate from water and is frequently used in drinking water applications. There are no known applications for wastewater at the scale of San Jose Creek (SJC). A disadvantage of the process is that it would typically add chlorides as a byproduct of the treatment. This would increase effluent total dissolved solids (TDS), which is not desirable for Los Angeles County Sanitation Districts (Sanitation Districts).

Figure 5A.2 illustrates the typical treatment steps for nitrate removal using ion exchange in a drinking water application.

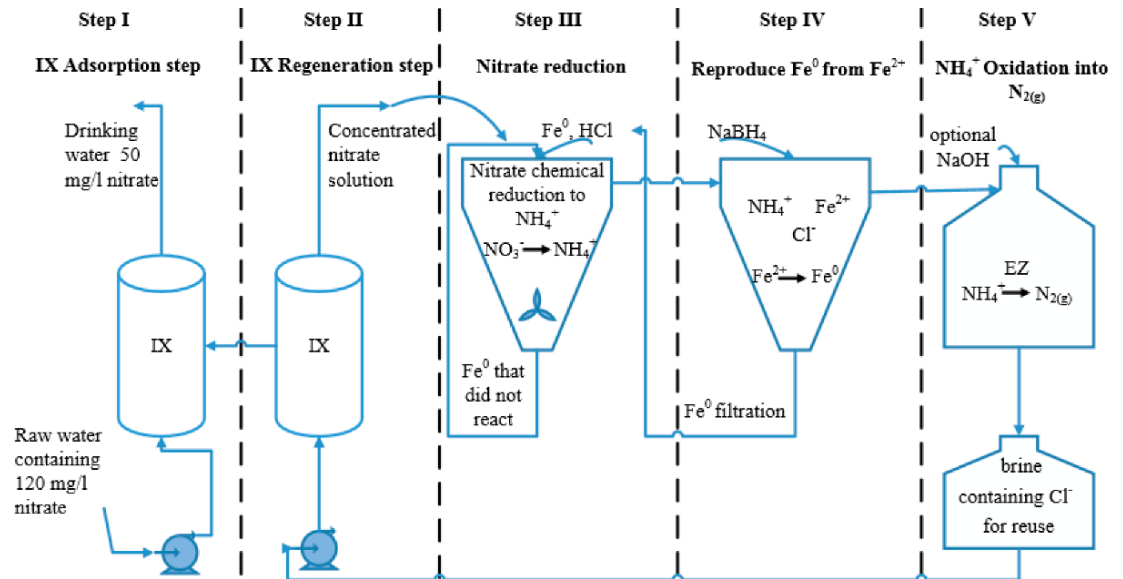


Figure 5A.2 Steps for Nitrate Removal With Ion Exchange

5A.1.3 Carbon Based Activated Treatment

CBAT consists of ozone, biologically activated carbon, and granular activated carbon. Ozone treatment in CBAT breaks down the long-chain per- and polyfluoroalkyl substances (PFAS) compounds and activated carbon treatment adsorbs the compounds thus removing them from the effluent. It can be an effective treatment method for removing organic compounds that are not easily removed with biological treatment systems. While there is benefit for using CBAT for removal of PFAS or other organic compounds, it does not remove nitrate or nitrogen.

Figure 5A.3 illustrates a process flow diagram of a typical CBAT system.

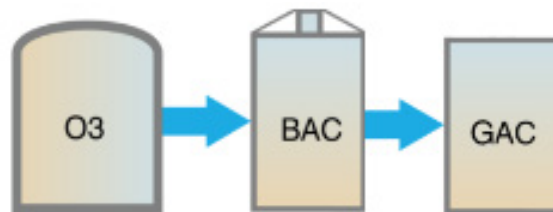


Figure 5A.3 CBAT Process Flow Diagram

5A.2 Post-Secondary Denitrification

Denitrification filters, biologically activate filters (BAFs), moving bed biofilm reactors (MBBR), and second-stage activated sludge technologies are described in this section. These technologies are based on reducing nitrate in the secondary effluent, which also will reduce the total inorganic nitrogen (TIN) in the final effluent. All post-secondary technologies require the addition of supplemental carbon and operation under anoxic conditions, which are necessary for reducing nitrate to nitrogen gas (denitrification). All technologies are capable of achieving effluent nitrate less than 1 milligram per liter (mg/L).

5A.2.1 Denitrification Filters

A denitrification filter is structurally similar to a sand filter. Media size is typically 2-3 millimeters (mm), and depth is typically 6-8 feet. The media in a denitrification filter provides a surface to grow biomass that converts nitrate to nitrogen gas. It also requires frequent backwashes to clean the media and maintain a healthy biomass. Denitrification filters are a proven technology and have the added benefit of providing filtration. Most filters can be operated in the 2-3 gallons per minute per square foot (gpm/sf) range, and some are even approved for Title 22 use in California. A disadvantage of denitrification filters are the high space requirements since they are designed to operate at 2-3 gpm/sf (instead of 5 gpm/sf for conventional filtration).

Figure 5A.4 shows illustrations of conventional and a continuously backwashed denitrification filters.

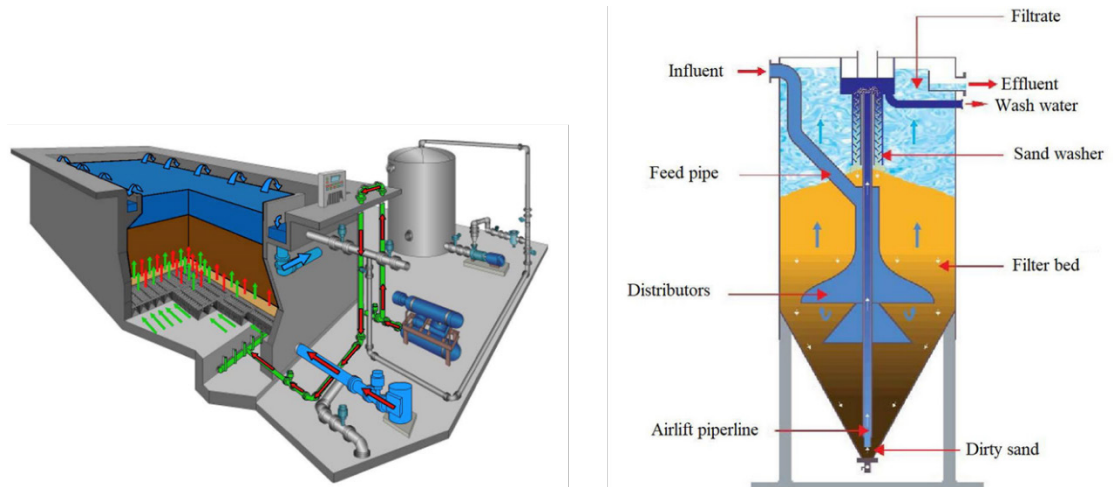


Figure 5A.4 Denitrification Filters

5A.2.2 Biologically Active Filters

The BAF is a submerged media filter. The media are typically plastic or polystyrene ranging in size from 3-5 mm and provide a surface for biofilm to grow. The media bed depth is 10-12 feet. Wastewater passes through the media in either an upflow or downflow configuration, and the media is regularly backwashed to remove excess solids that slough off. Although the effluent solids concentrations will not be as low as denitrification filters, they are capable of generating a 30/30 effluent. When operated to nitrify, air is introduced at the bottom to create aerobic conditions. When operated to denitrify, they are unaerated and require supplemental carbon (as with any post-secondary denitrification process). The main advantage of BAFs is that they are

compact when compared to denitrification filters. The main disadvantage is their designs typically require a two-story process equipment gallery, which drives up capital costs.

Figure 5A.5 is process flow schematic for nitrifying and denitrifying BAFs. For the Sanitation Districts, unaerated BAFs are being considered to reduce nitrate.

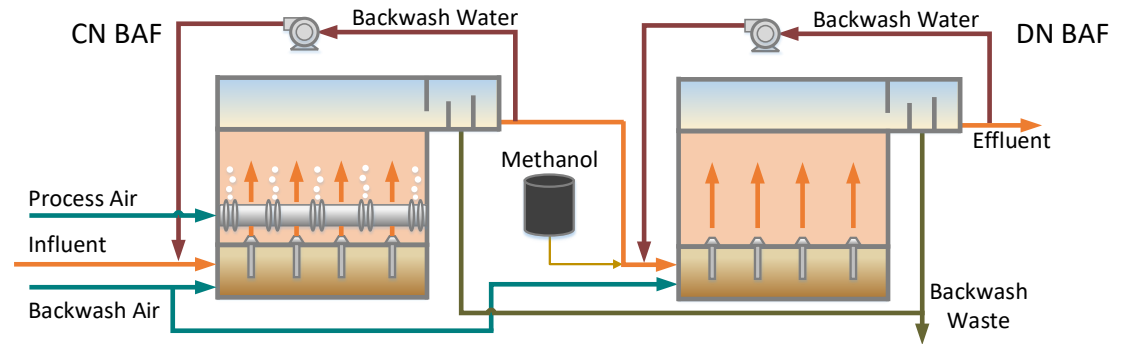


Figure 5A.5 Nitrifying and Denitrifying BAFs

5A.2.3 Moving Bed Biofilm Reactor

MBBRs are reactors with floating plastic media which provide a surface for biomass to grow. The plastic media are retained in the reactor using effluent screens, and the effluent requires solids separation to meet a 30/30 target. Typically, dissolved air flotation or high-rate filters (such as cloth media filters) are used for solids separation. This would be necessary if implemented at SJC to prevent solids overload on the tertiary filters.

Like the BAFs, the process can be operated to nitrify (with air) or denitrify (unaerated with mixing and supplemental carbon). The main advantage of this process is that it is the most compact and less complicated to operate than other post-secondary technologies as there is no need for backwashes. The disadvantage is that it requires solids separation and is not very compact when this is considered. In addition, there are fewer US installations compared to the other post-secondary technologies.

Figure 5A.6 shows MBBR Biofilm and process flow diagram.



Figure 5A.6 Nitrifying MBBR Biofilm (left) and MBBR Process Flow Diagram (right)

5A.2.4 Second-Stage Activated Sludge

Activated sludge systems can be designed for a range of influent water quality characteristics. This approach consists of an activated sludge system that is designed to further reduce the

nitrate in SJC's secondary effluent. Like any activated sludge system, there would be dedicated clarifiers for solids separation as well as return activated sludge (RAS) pumping. The reactor would be mixed and unaerated, and would require supplemental carbon like the other post-secondary technologies. An advantage of this approach is that it is a familiar technology. The main disadvantage is the large footprint associated with the clarifiers. Although the solids loading to the activated sludge system is significantly lower than the current step feed process, the clarifiers would have similar design overflow rates as the Sanitation Districts' final sedimentation tanks.

5A.3 Activated Sludge Biological Nutrient Removal

This section summarizes the different activated sludge configurations for BNR. The configurations below reflect varying levels of nitrogen and phosphorus removal.

5A.3.1 Modified Ludzack Ettinger

The Modified Ludzack Ettinger (MLE) configuration is the most widely used configuration for nitrogen removal and is capable of generating an effluent TIN ranging from 8-12 mg/L. Other than incidental uptake for biomass, there is no biological phosphorus removal occurring. In the MLE configuration, the bioreactors are divided into separate anoxic and aerobic zones. Nitrification occurs in the aerobic zone and denitrification occurs in the anoxic zone. Internal mixed liquor recirculation (IMLR) pumps bring nitrate from the aerobic zone back to the anoxic zone where denitrification occurs. RAS pumps bring the higher concentration mixed liquor suspended solids (MLSS) back to the anoxic zone. In most cases, supplemental carbon is not needed, but can be added to the anoxic zone if there are carbon limitations.

Figure 5A.7 is a process flow diagram of the MLE configuration.

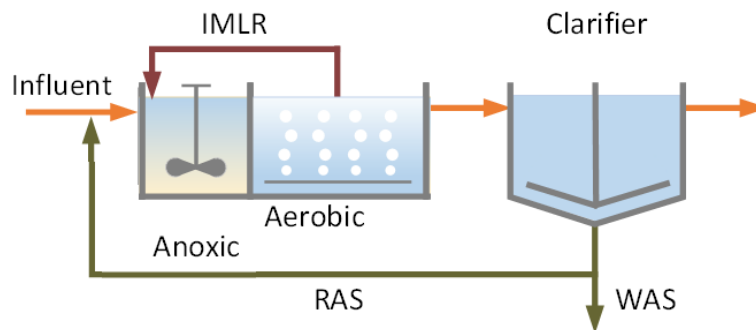


Figure 5A.7 MLE Process Flow Diagram

5A.3.2 Anaerobic, Anoxic, Oxic

The anaerobic, anoxic, oxic (A₂O) configuration is similar to MLE in that it has an anoxic and aerobic zone with IMLR recirculation. In addition, there is an anaerobic zone to promote the growth of phosphorus accumulating organisms (PAOs) for phosphorous removal. This configuration is able to achieve effluent TIN ranging from 8-12 mg/L and orthophosphate less than 1 mg/L. One of the challenges of operating a configuration designed for biological phosphorus and nitrogen removal is that sometimes there are carbon limitations and supplemental carbon or more proactive carbon management may be needed. If needed, supplemental carbon could be added to the anaerobic and/or anoxic zones.

Figure 5A.8 is a process flow diagram of the A2O configuration.

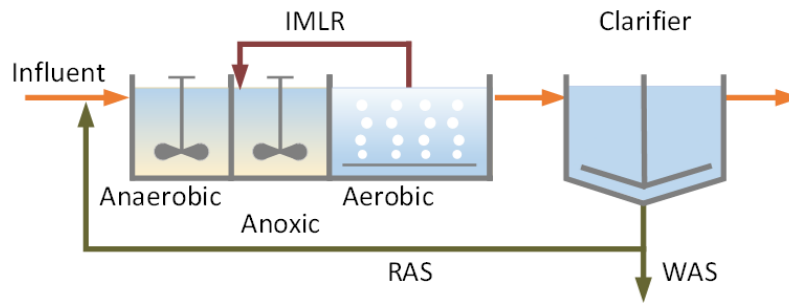


Figure 5A.8 A2O Process Flow Diagram

5A.3.3 4-Stage Bardenpho

The 4-Stage Bardenpho is similar to MLE in that it has an anoxic and aerobic zone with IMLR recirculation. In addition, there is a second anoxic zone (typically referred to as post-secondary anoxic) where supplemental carbon is added to achieve additional denitrification beyond what the MLE process would normally achieve. The final aerobic zone is there for final polishing and bioflocculation of the solids prior to entering the final clarifiers. This configuration is capable of generating an effluent TIN of 1 mg/L or less, and is considered the limit of technology (LOT) for biological treatment of nitrogen. Other than incidental uptake for biomass, there is no biological phosphorus removal occurring.

Figure 5A.9 is a process flow diagram of the 4-Stage Bardenpho configuration.

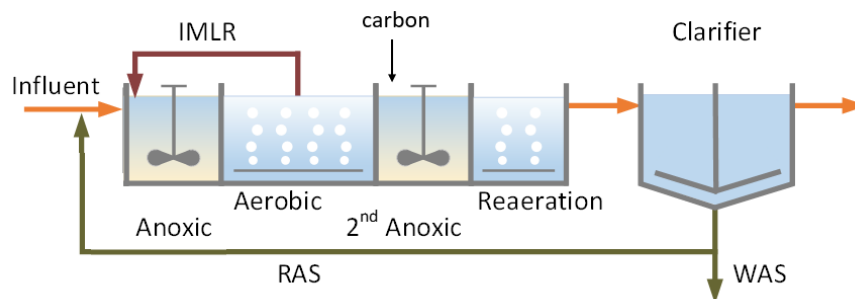


Figure 5A.9 4-Stage Bardenpho Process Flow Diagram

5A.3.4 5-Stage Bardenpho

The 5-Stage Bardenpho has all the same elements as the 4-Stage Bardenpho along with an anaerobic zone at the front. Like the A2O process, the purpose of the anaerobic zone is to promote the growth of PAOs for phosphorous removal. This configuration is considered the LOT for biological removal of nitrogen and phosphorus and is able to achieve effluent TIN and orthophosphate of less than 1 mg/L. Lower concentrations for orthophosphate can be achieved with downstream chemical addition and filtration. Like the A2O, one of the challenges of operating a configuration designed for biological phosphorus and nitrogen removal is that sometimes there are carbon limitations and supplemental carbon or more proactive carbon management may be needed.

Figure 5A.10 is a process flow diagram of the 5-Stage Bardenpho configuration.

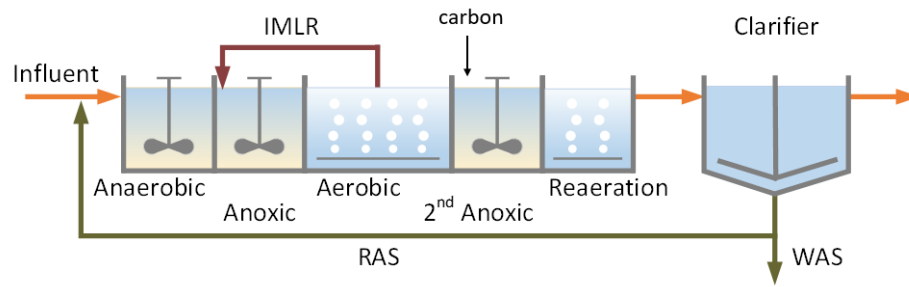


Figure 5A.10 5-Stage Bardenpho Process Flow Diagram

5A.3.5 Step Feed

The step feed configuration consists of a series of anoxic zones where influent is introduced followed by aerobic zones. The nitrate generated in the aerobic zones (from nitrification) is denitrified in the downstream anoxic zone where the influent feed supplies carbon. RAS flow is returned to the first anoxic zone, and it is typically higher than other BNR processes to maximize how much nitrate is introduced into the first anoxic zone. Effluent nitrate from step feed processes are quite variable ranging from 5-15 mg/L, and depend on the number of influent feed locations. SJC's step feed processes has three feed locations and effluent nitrate ranges from 5-8 mg/L. Some advantages of step feed are there is no IMLR circulation, and the reactor volume requirements are lower compared to other BNR configurations. Disadvantages are that the process is sometimes susceptible to ammonia breakthrough, and it is not capable of meeting LOT levels of nitrogen removal (unless other modifications are made). In addition, achieving biological phosphorus and phosphorus removal is more challenging in a step feed configuration.

Figure 5A.11 is a process flow diagram of the step feed configuration with three influent feed locations.

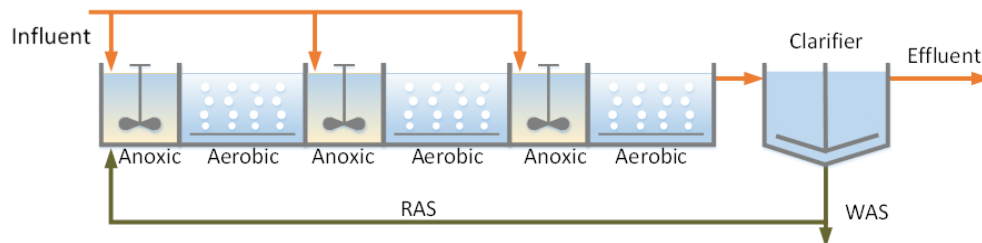


Figure 5A.11 Step Feed Process Flow Diagram

5A.3.6 Simultaneous Nitrification and Denitrification

Instead of compartmentalizing the reactor with dedicated anaerobic, anoxic, or aerobic zones like the other activated sludge configurations, this process relies on operating at low dissolved oxygen (DO) concentrations and creating oxygen gradients in the basin or in the floc. When operating at low DO concentrations in the bulk liquid, the outer layers of the floc are aerobic and nitrify, while the inner layers are anoxic and denitrify. This is possible in any activated sludge configuration and is most commonly practiced in complete mix or oxidation ditch reactor configurations. This process has been shown to perform as good or sometimes better than an

MLE process, and generally is more efficient in using carbon in the wastewater for denitrification. Some oxidation ditch facilities operating at long solids retention times (SRTs) (greater than 20 days) have reported effluent nitrate data as low as 2-3 mg/L. Despite these benefits, some of the disadvantages of the low DO operation are that the process is operating closer to ammonia breakthrough and there is a higher risk of generating poor settling sludge, which means there is a greater risk of process upset. In addition, the low DO operation also makes the process susceptible to biological phosphorus growth, which could cause struvite formation in the solids handling facilities.

Figure 5A.12 is a diagram of an example simultaneous nitrification and denitrification (SND) control strategy, where DO and ammonia are monitored to determine the air flow setpoint.

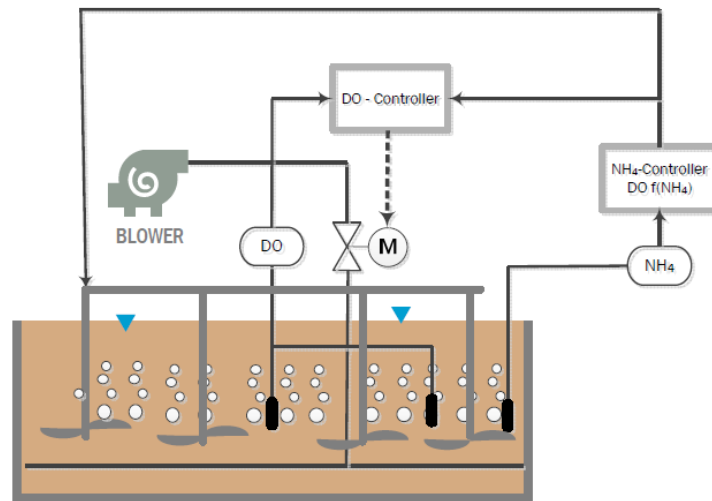


Figure 5A.12 SND Control Diagram

5A.4 Established Intensification Technology

Intensification is the approach of achieving more capacity within a unit of reactor volume than a conventional activated sludge process. This is accomplished by operating with more biomass inventory (i.e., pounds of biomass) compared to a conventional activated sludge reactor. Different approaches for operating with more biomass and intensifying are described further in this section. It should be noted that any of the activated sludge configurations described above could be utilized with any of the intensification approaches described here. This means that the performance is similar to activated sludge and is dependent on the configuration.

5A.4.1 Ballasted Activated Sludge

Ballasted activated sludge (BAS) is the approach of adding ballast to the reactor to enhance settling, which allows the reactor to be operated at higher MLSS concentrations compared to activated sludge without overloading secondary clarifiers. By operating at a higher MLSS in the reactors, more biomass can be maintained in the reactor, resulting in more treatment capacity (and intensification). A common material used for ballast is magnetite. To minimize the loss of magnetite, recovery equipment is added to the reactors to enhance settling. Flocs with enmeshed magnetite are denser and settle more quickly than a typical floc. The main advantage of BAS is that the reactor footprint is more compact than activated sludge (30-40 percent reduction). Disadvantages are that the process requires a lot of mechanical equipment to recover

the magnetite, and there is an ongoing cost for makeup magnetite as 100 percent recovery is not possible. In addition, because the mixed liquor is at higher concentrations and more dense, more aeration and mixing energy is needed compared to conventional activated sludge.

Figure 5A.13 is a process flow diagram of a BAS system with an MLE configuration. As mentioned previously, the process configuration could be any of those described for activated sludge.

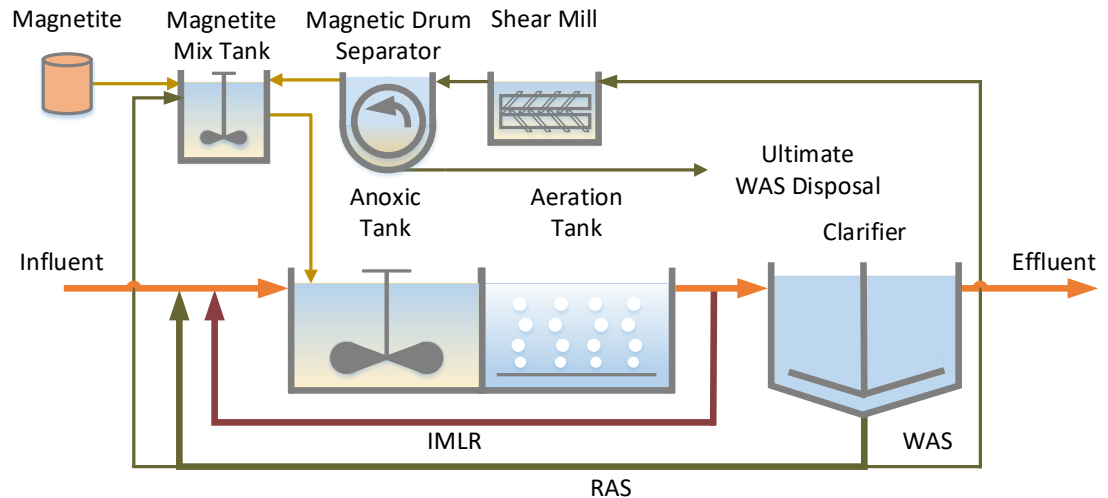


Figure 5A.13 BAS Process Flow Diagram (MLE Configuration)

5A.4.2 Integrated Fixed Film Activated Sludge

Integrated fixed film activated sludge (IFAS) is considered a hybrid system that utilizes the suspended growth biomass of an activated sludge system and the biofilm that grows on media in a bioreactor. The media can be fixed or floating, but the most common in use today is floating media. Retention screens keep the floating media in the bioreactor while mixed liquor passes to the secondary clarifiers for solids separation. Since a portion of the required biological inventory can be provided with the attached growth biomass, less suspended growth biomass and bioreactor volume is needed than in a conventional activated sludge process.

The main advantage of IFAS is that the reactor footprint is more compact than activated sludge. In cold climates, there could be a 30-40 percent reduction in volume as the biofilm is more effective in growing nitrifying biomass than suspended growth biomass which is susceptible to washout at cold temperatures. At temperatures experienced at the Sanitation Districts, the benefit is not as significant and would be closer to a 10-15 percent reduction in bioreactor volume. The main disadvantage is that the process requires more process aeration than conventional activated sludge because the bulk liquid DO concentration needs to be elevated (2-4 mg/L) to sufficiently penetrate the biofilm. In addition, systems with floating media are not able to use fine bubble diffusers, which further increase air demand.

Figure 5A.14 is a process flow diagram of an IFAS system with an MLE configuration. As mentioned previously, the process configuration could be any of those described for activated sludge.

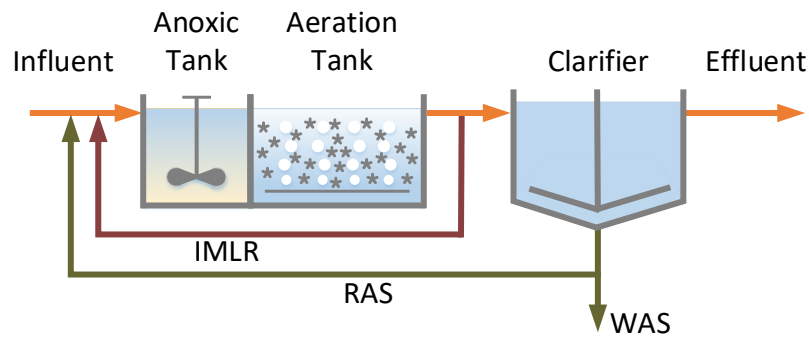


Figure 5A.14 IFAS Process

5A.4.3 Membrane Bioreactor

Membrane bioreactor (MBR) systems are bioreactors followed by membrane filters for solids separation. Permeate pumps provide suction to filter the mixed liquor through micro- or ultra-filter membranes. With porous openings ranging from 0.01-0.4 micrometer in diameter, the filters would physically separate the suspended solids from the liquid to remove particulate biological oxygen demand, solids, nitrogen, and phosphorus in the mixed liquor. The membrane types are generally categorized by hollow-fiber or a flat-plate configuration. The flat-plate configurations are able to handle higher MLSS concentrations, but require larger membrane tanks.

The main advantage of this approach is that the reactor footprint is usually 50 percent of what would be required for activated sludge. Space for membrane filters and tanks is usually less than would be required for secondary clarifiers in an activated sludge process. As a result, MBRs have the most compact footprint of all the intensification technologies described here. In addition, MBRs provide the additional benefit of providing tertiary quality water (i.e., Title 22 California), which is a higher quality than any of the other alternatives described. Disadvantages are the need for fine screening of the influent, the high capital cost, and higher aeration and pumping costs. In addition to process aeration, air scour for membrane cleaning is not insignificant along with the high RAS flow rates that are required (200-400 percent of Q).

Figure 5A.15 is a process flow diagram of a basic MBR in an MLE configuration. As mentioned previously, the process configuration could be any of those described for activated sludge.

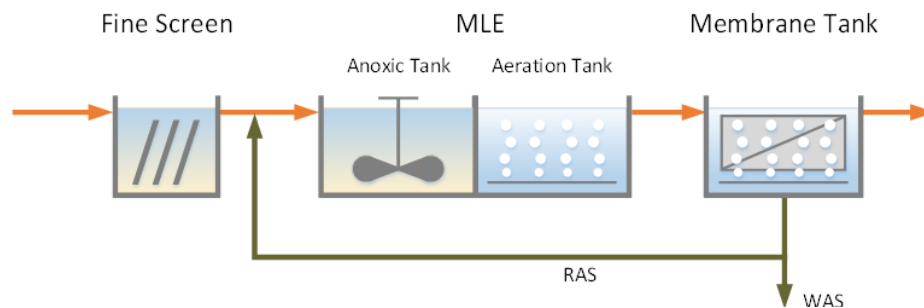


Figure 5A.15 MBR Process Flow Diagram

5A.5 Emerging Technology

Emerging technologies are continuously being developed in an effort to improve upon the performance, reduce cost, or increase the capacity that can be treated using established technologies.

5A.5.1 Aerobic Granular Sludge

Aerobic granular sludge (AGS) has been developed over the last 10-15 years as a nutrient removal alternative with a smaller footprint than activated sludge BNR systems. AGS incorporates the same bacteria (i.e., heterotrophs, ammonia and nitrite oxidizing bacteria, and phosphorus accumulating organisms) found in activated sludge, except the biomass grows as heterogeneous granules rather than as individual flocs.

Activated sludge processes use partitions within reactors and recirculation flows to create the right environment to grow the different types of bacteria in a nutrient removal system. AGS utilizes sequencing batch reactors (SBRs) instead. Typical cycle times for the reactor range from 3-8 hours and include a mix/aeration period followed by a settle period. During the settle period, sludge is first wasted and then influent is fed to the bottom of the reactor. The feed cycle displaces the settled effluent over effluent weirs at the top.

By controlling when and where sludge is wasted from the reactor, slowly settling flocculent biomass is selectively wasted from the system while the more rapidly settling, denser biomass (or granules) are retained. By selecting for rapidly setting biomass, the bioreactor can be operated at much higher effective mixed liquor concentrations and SRT compared to activated sludge. Typical design criteria are a 20-40-day SRT and an effective MLSS of 8,000 mg/L. The bioreactor volume for an AGS process is similar to a conventional activated sludge BNR process, but secondary clarifiers are not needed since settling is achieved in the AGS bioreactors.

The main benefit of this technology is the more compact footprint. The main disadvantage is that it requires all new tank construction and cannot easily be retrofitted into existing activated sludge reactors.

Figure 5A.16 is an illustration of a typical granule cross-section and the different operating modes in a typical cycle.

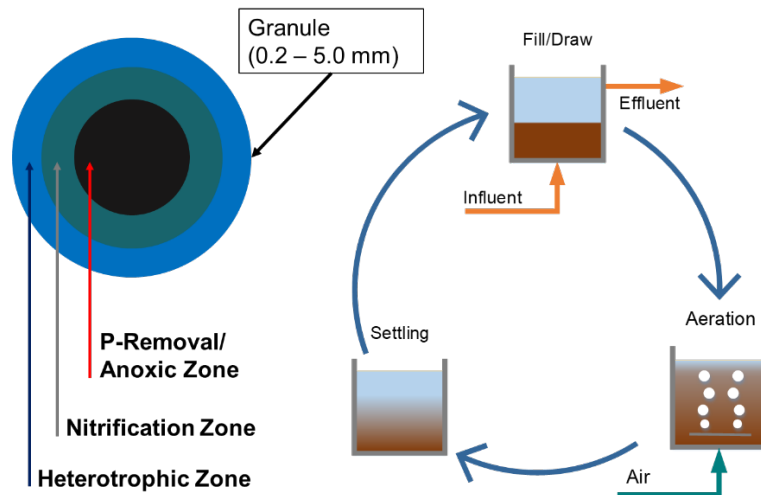


Figure 5A.16 AGS

5A.5.2 Flow Through Aerobic Granular Sludge

This is a variation of the traditional AGS system which is based on intermittent feed to SBRs. In a flow through system, wastewater would be fed to the reactors continuously like conventional activated sludge and other conventional processes. A benefit of this approach is that it can more easily be retrofit into existing aeration tanks. There is ongoing research in the industry as well as pilot opportunities, however, a disadvantage is there are no known full-scale systems currently in operation.

5A.6 Membrane Aerated Biofilm Reactor

Membrane aerated biofilm reactor is a technology where membrane modules are installed in the anoxic zones of an activated sludge process. DO is supplied through the membrane module, which provides a surface to grow biofilm under aerobic conditions. Aerobic biofilm that is grown in the anoxic zones increases the system's overall biomass, nitrification, and overall treatment capacity. The main advantages are similar to those for IFAS in that the reactor footprint is more compact than activated sludge. In cold weather applications, there could be a reduction of 20-30 percent reduction of the reactor volume. At temperatures experienced at the Sanitation Districts, the benefit is not as significant and would be closer to a 5-15 percent reduction in bioreactor volume. The main disadvantage is that the process is not very mature and design approaches are not as well established yet. The Sanitation Districts has piloted this technology at one of their facilities.

Figure 5A.17 is an illustration of the membrane aerated biofilm module.

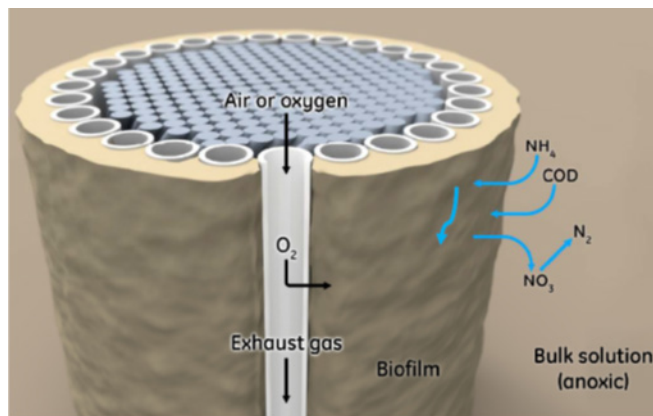


Figure 5A.17 Membrane Aerated Biofilm Module

5A.7 A/B Process

The A/B process is a two-stage process where the A-stage is comprised of a very short hydraulic retention time and SRT activated sludge process that is designed to maximize carbon removal. The configuration of the B-stage would depend on treatment objectives, and would typically be a longer SRT activated sludge process designed for ammonia or total nitrogen removal. It has also been paired with emerging or experimental efforts to implement nitrite shunt or mainstream deammonification. An advantage of this process is it is designed to maximize carbon removal in the A-stage, which is beneficial at facilities with anaerobic digestion and energy recovery. Disadvantages are that it is not very common and operating two stages

increases overall complexity and required space. Some of that space can be recovered if primary clarifiers are leveraged for use in the A-Stage.

Figure 5A.18 is an example process flow diagram of the A/B process.

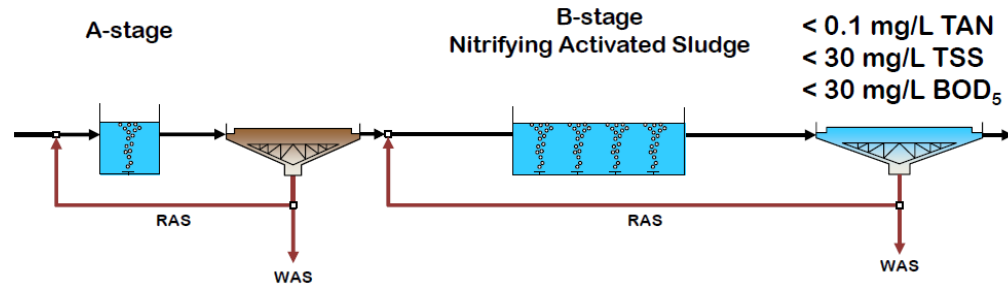


Figure 5A.18 A/B Process

5A.8 Nitrite Shunt

Nitrite shunt is an activated sludge process where ammonia is first partially oxidized (or nitrified) to nitrite instead of complete oxidation to nitrate. The nitrites are then converted to nitrogen gas under anoxic conditions. The main benefit of nitrite shunt is that it uses 25 percent less oxygen for nitrification and 40 percent less carbon for denitrification. This is because the ammonia oxidation step is stopped at nitrite. The key to successful nitrite shunt operation is the ability to suppress nitrite oxidizing bacteria (NOB) while promoting the growth of ammonia oxidizing bacteria (AOB). This is typically accomplished with robust nutrient monitoring, low DO operation and ammonia-based aeration control, and operating at low SRTs close to the washout of AOBs.

Figure 5A.19 is an illustration of the molecular pathway of conventional nitrification and denitrification, nitrite shunt, and deammonification.

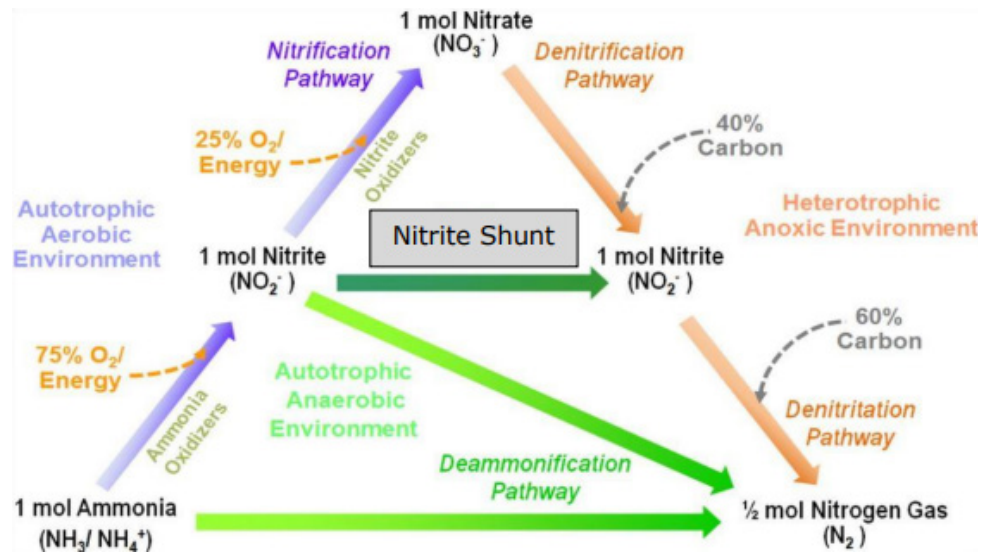


Figure 5A.19 Nitrogen Removal Pathways

5A.9 Mainstream Deammonification

The first step in deammonification is the same as nitrite shunt. In deammonification, approximately half the ammonia is partially nitrified to nitrite. What is unique to deammonification is that the remaining ammonia is then removed through anaerobic ammonium oxidation. This pathway is illustrated in Figure 5A.19 above. The main benefit (like nitrite shunt) is there is reduced aeration and chemical usage compared to conventional nitrogen removal pathways. Similar to nitrite shunt, the key is being able to suppress the NOBs for the process to work. There is growing experience with deammonification in sidestream treatment applications. Mainstream deammonification has been tested at full scale, most notably at the Strass Wastewater Treatment Plant (WWTP) in Austria. At that facility, the deammonification step was incorporated into the "B-Stage" of an "A/B" process. A goal of that facility was to maximize energy and resource recovery. The results from full-scale operation have been variable, and results of research at other facilities are similar. While there is great benefit to successfully implementing mainstream deammonification full-scale, it has not yet demonstrated reliable performance at full-scale.

Figure 5A.20 is a simplified process flow diagram of the Strass WWTP in Austria, where mainstream deammonification was the B-Stage of an A/B process. The A-Stage in that figure is labeled as "C-redirection" where "C" stands for carbon.

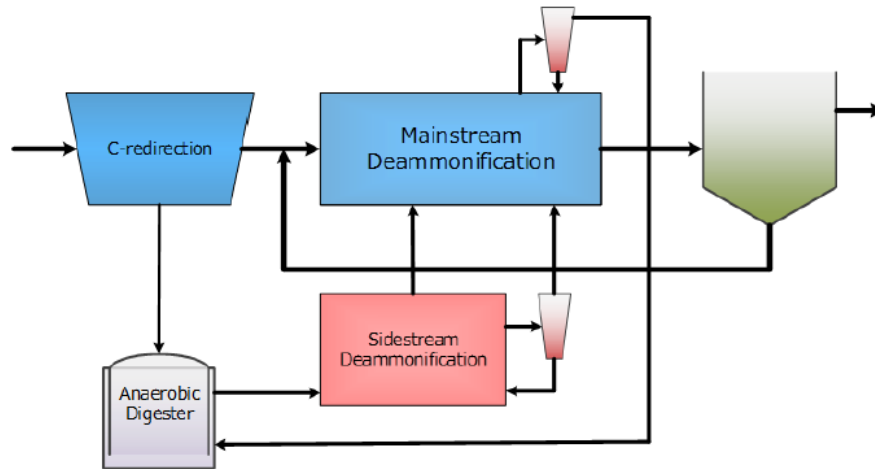


Figure 5A.20 Simplified Mainstream Deammonification Process Flow Diagram

5A.10 Anaerobic Membrane Bioreactor

The anaerobic MBR process combines an anaerobic bioreactor followed by membrane filtration. The primary advantage of anaerobic treatment is that there is no aeration required, which is the largest expense of traditional secondary treatment systems. In addition, it is possible to recover the methane to beneficially reuse. The disadvantage is that anaerobic treatment is generally not cost-effective unless wastewater strength is considerably higher than municipal raw sewage. This technology is better suited to higher-strength industrial wastewaters. In addition, while anaerobic treatment followed by membrane filtration may generate an effluent low in solids, additional treatment will still be needed to remove nutrients.

Figure 5A.21 is a schematic of a pilot system currently being tested at Silicon Valley Clean Water Agency in California.

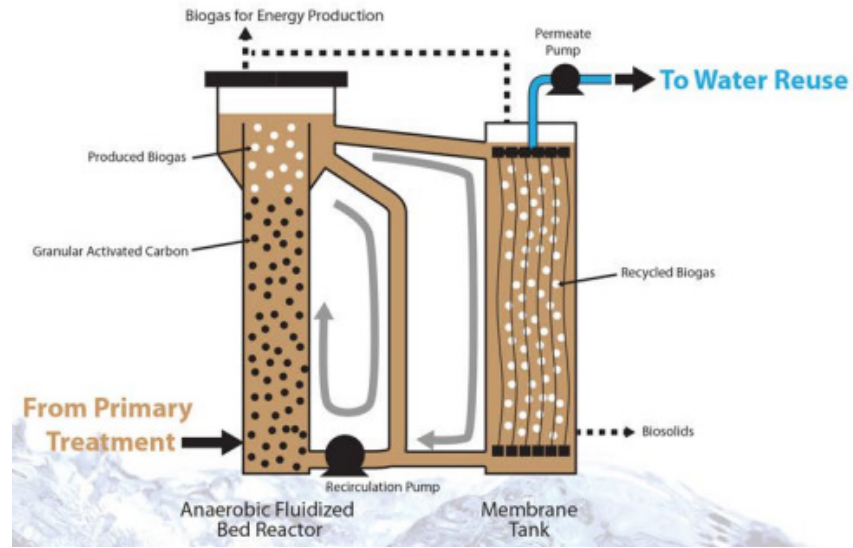


Figure 5A.21 Schematic of Pilot Anaerobic MBR

Appendix 5B

SCREENING RESULTS

Screening Results

Alternative	Technology/Configuration	Technology Maturity and Risk	Screening Criteria					Justification Notes for Screening	Selected for Further Evaluation of Alternative	Justification Notes for Selection for Further Evaluation of Alternative	
			Ability to Meet Regulations	Technology Maturity & Risk	Ease of Permitting	Site Constraints	Independent Operations				Conclusion (Pass/Fail)
Alternative 1 - Optimized Step Feed with Physical Treatment Process for N Removal											
	Reverse Osmosis	Established	Pass	Pass	Pass	Pass	Pass	Pass		Selected	Most suitable technology for scale of operation. Will require membrane pretreatment with MF/UF, or MBR effluent.
	Ion Exchange	Established	Pass	Fail	Pass	Pass	Pass	Fail	Although it would provide N removal, there are no operating facilities at this size.		
	CBAT or Advanced Oxidation	Established	Fail	Pass	Pass	Pass	Pass	Fail	Does not achieve N removal.		
Alternative 2 - Optimized Step Feed with Post-Secondary Denitrification											
	Denitrification Filters	Established	Pass	Pass	Pass	Pass	Pass	Pass		Alternate	Most common technology and will also provide opportunity to perform chemical P polishing with filtration.
	Biologically Active Filter (BAF)	Established	Pass	Pass	Pass	Pass	Pass	Pass		Selected	Smallest footprint
	Moving Bed Biofilm Reactor (MBBR)	Established	Pass	Pass	Pass	Fail	Pass	Fail	Not carrying forward as it would still require filtration (or alternate solids separation) to reduce solids loading on tertiary filters.		
	Activated Sludge	Established	Pass	Pass	Pass	Fail	Pass	Fail	Large footprint for clarification and more complex operation than other technologies in this group. Would also still require filtration for meeting Level 3 P target.		
Activated Sludge BNR											
	Modified Ludzack Ettinger (MLE)	Established	Fail	Pass	Pass	Fail	Pass	Fail	Would not achieve LOT for N or P. Would require significant changes to Step Feed process and require additional post-secondary facilities to meet LOT for N or P.		
	Anaerobic, Anoxic, Oxic (A2O)	Established	Fail	Pass	Pass	Fail	Pass	Fail	Would not achieve LOT for N or P. Would require significant changes to Step Feed process and require additional post-secondary facilities to meet LOT for N or P.		

Alternative	Technology/Configuration	Technology Maturity and Risk	Screening Criteria						Justification Notes for Screening	Selected for Further Evaluation of Alternative	Justification Notes for Selection for Further Evaluation of Alternative
			Ability to Meet Regulations	Technology Maturity & Risk	Ease of Permitting	Site Constraints	Independent Operations	Conclusion (Pass/Fail)			
	4-Stage Bardenpho (With supplemental carbon)	Established	Pass	Pass	Pass	Fail	Pass	Fail		Selected	Would achieve LOT for N and can be retrofitted in the existing tank volume.
	5-Stage Bardenpho (With supplemental carbon)	Established	Pass	Pass	Pass	Fail	Pass	Fail	Would achieve LOT for N and P. Would require significant change to existing Step Feed Process and would not fit on site. Additionally, LOT for P is not required for LBWRP currently.		
	Simultaneous Nitrification Denitrification	Emerging	Fail	Fail	Fail	Pass	Pass	Fail	Complex operation, and more difficult to control than other processes. Could be implemented as part of an operational strategy with any of the alternatives. In addition, very susceptible to poor settleability due to risk of low DO filament growth.		
Alternative 4 - Established Intensification Technology											
	Ballasted Activated Sludge (BAS)	Established	Pass	Fail	Pass	Pass	Pass	Fail	In addition to technology maturity, lots of mechanical equipment that are points of failure and risk.		
	Integrated Fixed Film Activated Sludge (IFAS)	Established	Pass	Pass	Pass	Fail	Pass	Fail	Marginal benefit compared to step feed with respect to site constraints. Would not fit on site.		
	Membrane Bioreactor (MBR)	Established	Pass	Pass	Pass	Pass	Pass	Pass		Selected	Significant industry experience, smallest footprint, and can easily incorporate P removal. In addition, MBR effluent is higher quality than any other intensification technologies and provides suitable pretreatment for RO, if implemented in the future (to meet other potential regulations).
Alternative 5 - Emerging Technology											
	Aerobic Granular Sludge (AGS)	Emerging	Pass	Pass	Pass	Fail	Pass	Fail	Although the technology may be sufficiently mature, would require new reactors in SBR configuration. No space on site to construct new reactors.		

Alternative	Technology/Configuration	Technology Maturity and Risk	Screening Criteria					Justification Notes for Screening	Selected for Further Evaluation of Alternative	Justification Notes for Selection for Further Evaluation of Alternative	
			Ability to Meet Regulations	Technology Maturity & Risk	Ease of Permitting	Site Constraints	Independent Operations				Conclusion (Pass/Fail)
	Flow-Through Aerobic Granular Sludge (AGS)	Emerging	Pass	Fail	Pass	Pass	Pass	Fail	Although the AGS technology may be sufficiently mature and there is sufficient space to implement the technology, it would require pilot and testing and there are no full-scale facilities in operation currently.	Selected as future emerging technology (not to be used for cost estimating)	If the District is interested in evaluating an emerging technology, this technology would be suitable for consideration. It could be retrofit within the existing treatment units and offers significant benefit to operations and capacity limitations.
	Membrane Aerated Biofilm Reactor (MABR)	Emerging	Fail	Fail	Pass	Fail	Pass	Fail	This is an emerging technology that requires operation at low SRTs and needs piloting to confirm process performance. It doesn't achieve LOT with the existing Step feed process configuration.		
	A/B Process	Emerging	Pass	Fail	Pass	Fail	Pass	Fail	Existing treatment units would be modified as A-stage and B-stage. A-stage would be a short HRT and SRT process, and B-stage could be a longer SRT activated sludge, nitrite shunt, or mainstream deammonification). If B-stage is activated sludge, not recommended to carry forward as would require significant modifications to existing process (while in operation), and process has large footprint for B-stage clarification and additional operational complexity of not having sufficient carbon for B-stage, and potential for poor settleability. If B-stage is nitrite shunt or deammonification, not recommended to carry forward due to insufficient successful full-scale operating experience for nitrite shunt or deammonification.		

Alternative	Technology/Configuration	Technology Maturity and Risk	Screening Criteria					Justification Notes for Screening	Selected for Further Evaluation of Alternative	Justification Notes for Selection for Further Evaluation of Alternative
			Ability to Meet Regulations	Technology Maturity & Risk	Ease of Permitting	Site Constraints	Independent Operations			
	Mainstream Deammonification	Emerging	Pass	Fail	Pass	Pass	Pass	Fail	At least 4 facilities that have tried to demonstrate full scale (Strass Austria, Changqi Singapore, Ejby Mlle Denmark, and AlexRenew USA) and at least 4 facilities have piloted (Blue Plains and Chesapeake-Elizabeth USA, Veolia Water in France and Sweden). WERF Studies have concluded technology is "viable," but results at previous facilities have been "lacking."	
	Nitrite Shunt	Emerging	Pass	Fail	Pass	Pass	Pass	Fail	Some success at full-scale facilities, (St Petersburg SW, Madison Nine Springs, Rochester NH, New Zealand), but not recommended for further evaluation as industry experience is less than others in this category (like AGS). Complex process controls and increased risk of poor settleability due to low DO operation.	
	Anaerobic MBR	Emerging	Fail	Fail	Pass	Pass	Pass	Fail	Insufficient full-scale operating experience and unable to meet Limit of Technology unless additional processes are added.	