Valley Sanitary District

WATER RECLAMATION FACILITY FINAL MASTER PLAN

September 2015

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Valley Sanitary District

Water Reclamation Facility Master Plan

FINAL

September 2015

Prepared for Valley Sanitary District
Indio, California

Prepared by MWH
Pasadena, California

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ACRONYMS AND ABREVIATIONS

To conserve space and improve readability, abbreviations have been used in this report. Each abbreviation has been spelled out in the text the first time it is used. Subsequent usage of the term is usually identified by its abbreviation. The abbreviations used in this report are shown below.

°C	Degrees Celsius
°F	Degrees Fahrenheit
AC	Asphalt Concrete
ADF	Average Daily Flow
ADWF	Average Dry Weather Flow
AQMD	Air Quality Management District
ASP	Activated Sludge Process
BO	Build Out
BOD	Biochemical Oxygen Demand (5-day)
BTPS	Biological Treatment Pond System
BTU	British Thermal Unit
cBOD	Carbonaceous Biochemical Oxygen Demand (5-day)
CCT	Chlorine Contact Tank
CDPH	California Department of Public Health
CHP	Combined Heat and Power
CIP	Capital Improvement Program
COD	Chemical Oxygen Demand
CSMP	Collection System Master Plan
CT	Product of Total Chlorine Residual and Modal Contact Time
CVAG	Coachella Valley Association of Governments
CVSC	Coachella Valley Stormwater Channel
District	Valley Sanitary District
DS	Digested Sludge
EIA	United States Energy Information Administration
EPA	Environmental Protection Agency
GBT	Gravity Belt Thickener
gpcd	Gallons per Capita per Day
GIS	Geographic Information System
HRT	Hydraulic Residence Time
ICE	Internal Combustion Engine
MBTU	Million British Thermal Unit
MG	Million Gallons
mgd	Million Gallons per Day
MLSS	Mixed Liquor Suspended Solids
MPN	Most Probable Number

MWH	MWH Americas, Inc.
NPDES	National Pollutant Discharge Elimination System
NTU	Nephelometric Turbidity Unit
ORC	Organic Rankine Cycle
PDWF	Projected Dry Weather Flow
PPA	Power Purchase Agreement
PS	Primary Sludge
PWWF	Projected Wet Weather Flow
QA	Quality Assurance
QC	Quality Control
RAS	Return Activated Sludge
ROI	Return on Investment
SCADA	Supervisory Control and Data Acquisition
SCR	Selective Catalytic Reduction
SOUR	Specific Oxygen Uptake Rate
SOW	Scope of Work
SRT	Solids Residence Time
SS	Suspended Solids
SWD	Side Water Depth
TDH	Total Dynamic Head
Title 22	Portion of the California Code of Regulations defining the requirements for recycling wastewater
TKN	Total Kjeldahl Nitrogen
TS	Total Solids
TSS	Total Suspended Solid
TWAS	Total Waste Activated Sludge
USEPA	United States Environmental Protection Agency
UV	Ultra-Violet
VOC	Volatile Organic Carbon
VS	Volatile Solids
VSD	Valley Sanitary District
VSR	Volatile Solids Reduction
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WRFMP	Water Reclamation Facility Master Plan
WRF	Water Reclamation Facility
WTP	Water Treatment Plant
WWPF	Wet Weather Peaking Factor

Executive Summary

Executive Summary

This Executive Summary of the Water Reclamation Facility (WRF) Master Plan for Valley Sanitary District (VSD) provides a concise description of the objectives, background, existing and future flow projections, existing capacity and recommendations for future plant expansion. Also summarized is the recommended phasing and costs for each phase.

ES.1 OBJECTIVES

This Master Plan has been developed under Task Authorization No. 3 between VSD and MWH Americas, Inc. (MWH) dated October 17, 2014.

The key objectives of the Master Plan are to:

- Assess the capacity of existing WRF unit processes.
- Determine the feasibility of continued use of the Biological Treatment Pond System (located to the south of the main WRF near the bird sanctuary) for secondary treatment.
- Prioritize unit process expansion or improvement.
- Forecast future flows using the 2013 Sewer Master Plan.
- Forecast future wastewater constituent loads to provide a basis for treatment unit sizing
- Select and size future treatment unit processes.
- Recommend phasing of treatment process expansion and improvements.
- Provide cost estimates (Capital and Operation & Maintenance) that can be incorporated into a phased Capital Improvement Program for the WRF.

In October 2006, Lee & Ro completed a Valley Sanitary District Wastewater Treatment Plant Master Plan. In November 2013, MWH completed a Collection System Master Plan, and among other outcomes determined the build-out flow for the area.

The goal of this Master Plan is to review the current capacities of pre-2006 as well as newly installed processes initially recommended in the Lee & Ro Master Plan, and to update treatment upgrades recommendations, phasing, and anticipated costs. Water recycling and cogeneration (developing electric power from digester gas) is also considered as part of this report. The future of the Biological Treatment Pond System is also discussed.

ES.2 BACKGROUND

The WRF is located adjacent to and on the southwest bank of the Whitewater River (also referred to as the Coachella Valley Stormwater Channel - CVSC). This stream ultimately discharges to the Salton Sea 15 miles to the east of the WRF.

The service area is 96% in the City of Indio. Using the City population projections as a basis, future population projection is shown in **Table ES-1**.

Table ES-1 Projected Population and Flow for Service Area

Year	City of Indio Population Projection	Projected Average Flow, mgd
2010	76,036	
2014 (current)	**82,398**	**6.0**
2015	87,486	6.4
2020	100,387	7.3
2025	106,923	7.8
2030	113,681	8.3
2035	120,676	8.8
2040	128,097	9.4
2045	135,976	9.9
2050	144,338	10.5
Build-Out	274.000	20.0

The flow is projected based on multiplying the City of Indio projected population by the average of 73 gallons per capita per day from 2014 (6,000,000 gallons / 82,398 = 73 gallons). Parameters developed for flow projections is shown in **Table ES-2**.

Flow projections are shown in **Figure ES-1**.

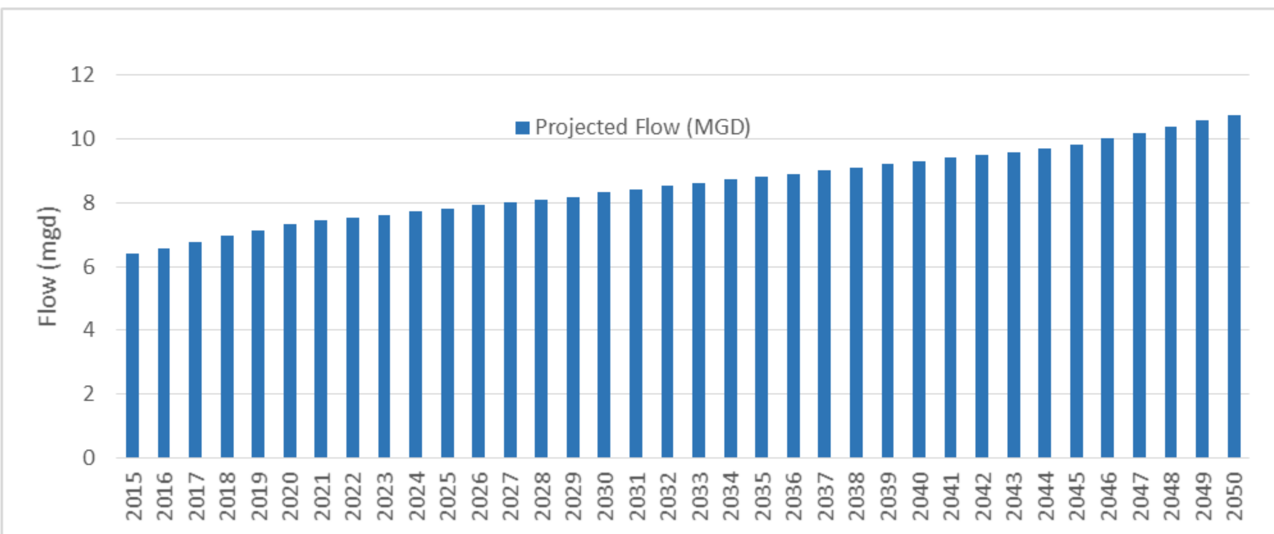


Figure ES-1 Projected flows

Table ES-2 Basis of Flow Projections

Existing Conditions		
Average daily flow	6.0	million gallons per day (mgd)
Served population	82,398	persons
Average per capita flow	73	gal/capita/day
Max Observed Wet Weather Inflow	16.5	mgd
Build Out		
Build-out Average Daily Flow	20	mgd
Build-out Peak Wet Weather Flow	44.5	mgd
Build-out Wet Weather Peaking Factor	2.2	-

Load projections for Biochemical Oxygen Demand, suspended solids and nitrogen are shown in **Table ES-3**.

Table ES-3 Existing BOD and TKN Loading

Water Quality Analysis		**50 percentile**	**90 percentile**	**99 percentile**
BOD	Concentration (mg/L)	256	313	354
TSS	Concentration (mg/L)	201	246	290
TKN	Concentration (mg/L)	49	52	53

BOD = Biochemical Oxygen Demand
TSS = Total Suspended Solids
TKN = Total Kjeldahl Nitrogen (measure of total organic nitrogen)

The 90%-ile BOD, TKN and TSS will be used for sizing the activated sludge basins, clarifiers and digesters. The 99%-ile BOD and TKN will be used for determining aeration requirements for the activated sludge plant.

ES.3 EXISTING WRF CAPACITY

The existing WRF liquid flow diagram is as shown in **Figure ES-2**. Biosolids process flow diagram is shown in **Figure ES-3**.

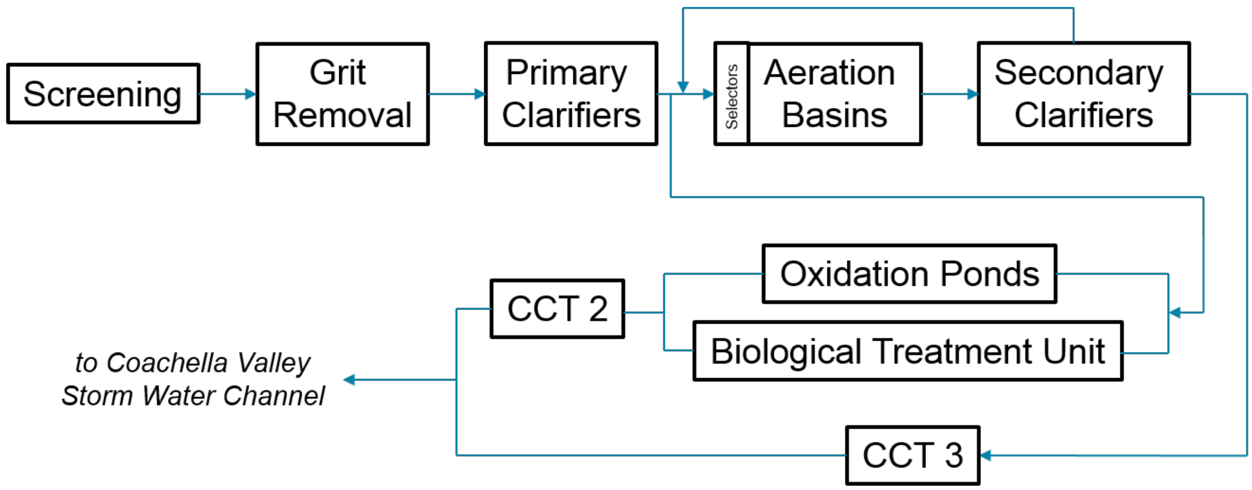


Figure ES-2 Existing Liquid Process Flow Diagram

The existing Biological Treatment Pond System (BTPS) south of Pond 3 has proven to be an ineffective secondary treatment facility. The recommendation is to decommission this facility to remove the vector attraction liability and cost of maintaining the facility.

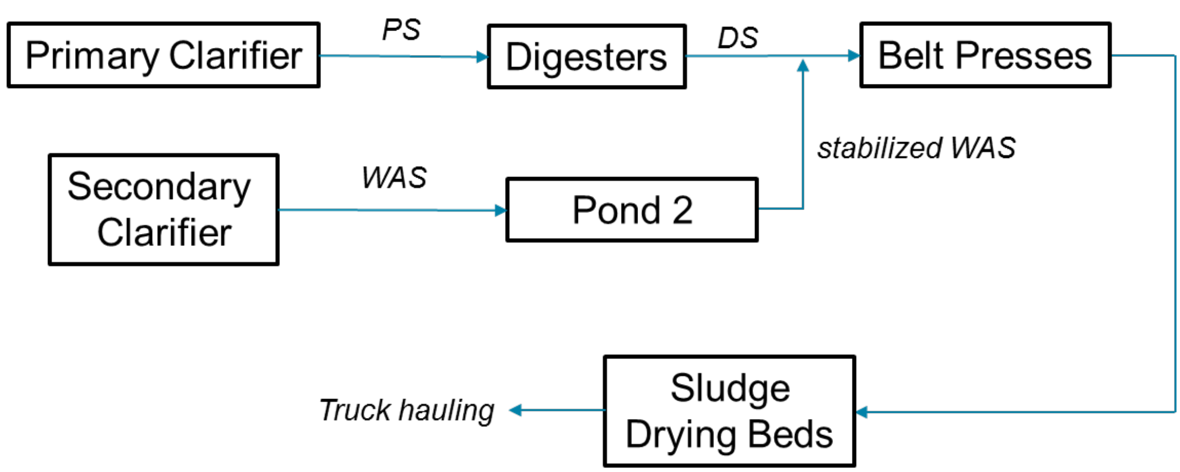


Figure ES-3 Existing Solids Process Flow Diagram

Figure ES-4 summarizes the capacity of each process in the existing WRF. As the graph shows, the most undersized processes are the grit chambers and the sludge drying beds. These will be the focus of the next phases of expansion for the WRF.

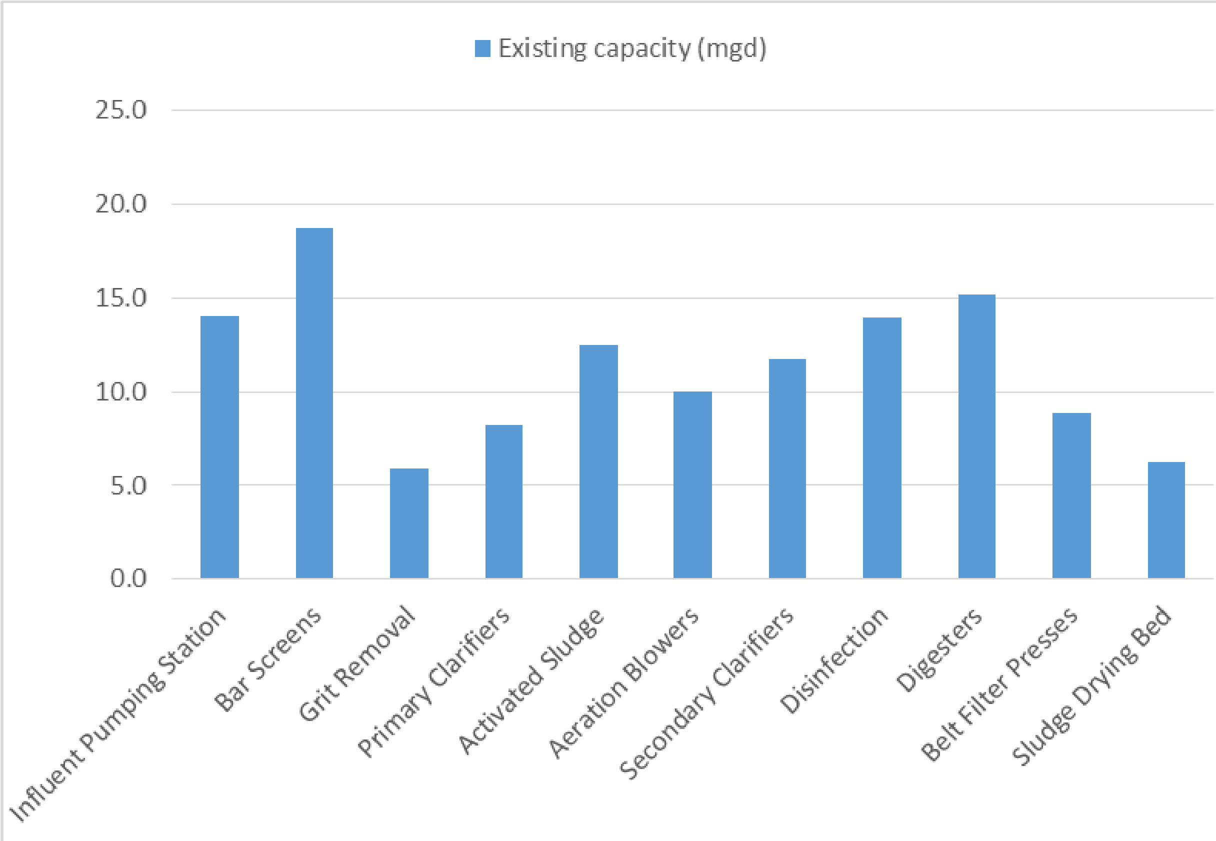


Figure ES-4 Existing WRF Process Capacity (Average Daily Flow)

ES.4 LIQUID PROCESS OPTIONS

Three liquid process options were developed, each with a different final effluent quality, but all sized for the build-out flow and loads.

- Option 1: Secondary without Nitrogen Removal (same as existing)
- Option 2: Secondary with Nitrogen Removal
- Option 3: Tertiary with Filtration (for recycling)

Figure ES-5, **Figure ES-6** and **Figure ES-7** show the process flow diagram differences between the three options.

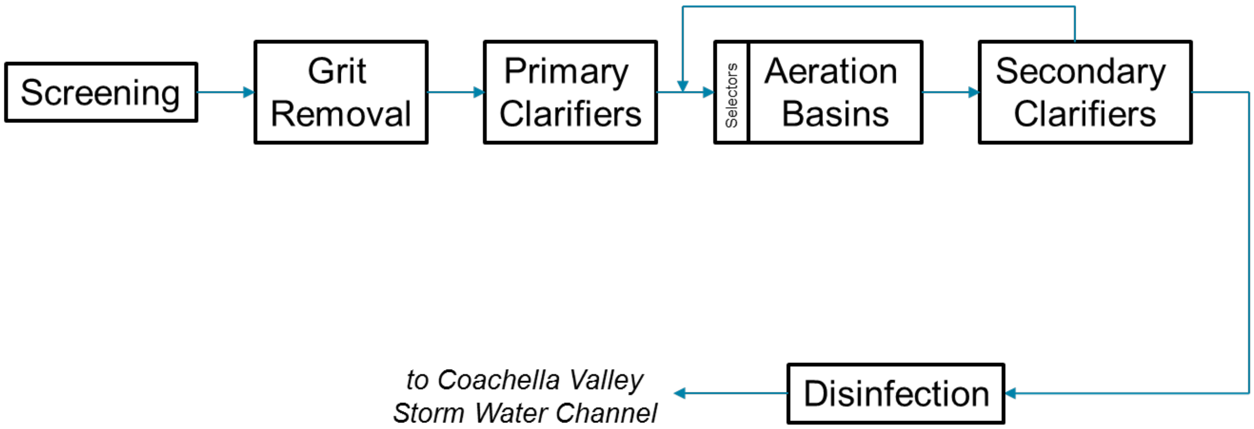


Figure ES-5 Option 1 Process Flow Diagram

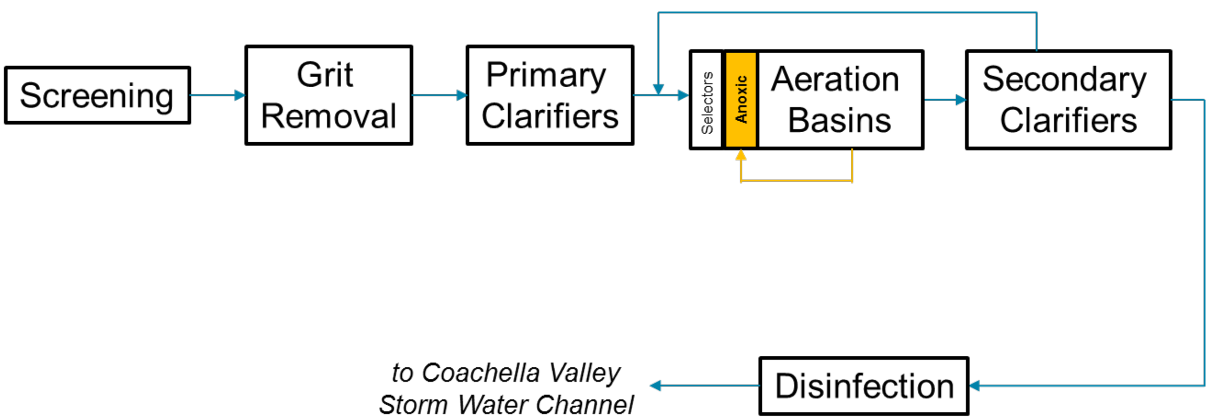


Figure ES-6 Option 2 Process Flow Diagram

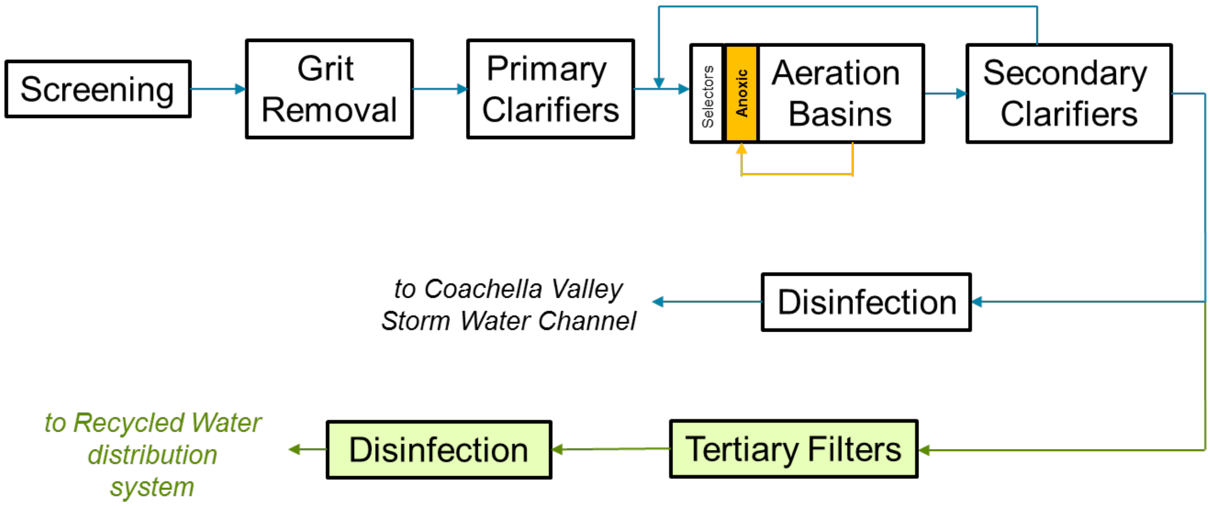


Figure ES-7 Option 3 Process Flow Diagram

The differences between Option 1 and 2 are the size of the aeration tanks (larger for Option 2), the size of aeration blowers (greater capacity for Option 2), and the addition of mixed liquor recycle (for Option 2). The headworks, primary clarifiers, (if selected) tertiary filters and all biosolids handling unit sizing are unaffected. Chlorine contact tanks size and (if selected) ultraviolet disinfection are not affected.

In the phasing plan assumptions were made that initially no changes to the effluent quality will be required (Option 1). When the plant reaches the capacity of the existing activated sludge plant (10 mgd as limited by blowers or 11.7 mgd as limited by secondary clarifiers), the new aeration tanks that replace the existing tanks are sized for nitrogen removal (Option 2).

Further, the phasing is based on providing Tertiary with Filtration (Option 3) when the plant flow approaches 8.2 mgd.

Note that filtration is not dependent on nitrogen removal and can work well with or without nitrogen removal in the secondary treatment process.

ES.5 BIOSOLIDS MANAGEMENT

Future biosolids processes are independent of which liquid treatment scheme is in use. Process selection is shown below in terms of technologies:

- Waste Activated Sludge thickening — Gravity Belt Thickener
- Biosolids Stabilization — Anaerobic Digestion
- Biosolids Dewatering — Belt Press
- Solids Drying — Solar Drying Beds

The above selections of biosolids management technologies do not differ from the existing solids processing system at the WRF with the exception of the handling of waste activated sludge (WAS). At present, WAS is discharged to Pond 2, stored and stabilized using surface aerators, then dredged to an existing belt press for dewatering. This method produces a well-stabilized sludge.

The future WAS will be thickened so that it is suitable for stabilization in the anaerobic digesters. In this manner, the digester gas production level will increase substantially once a means of thickening and digester capacity are available.

At present, well over 50% of all digester gas generated at the WRF is flared as a means of disposal. A small amount of digester gas is used in winter for heating water in the boilers for the digesters to maintain temperature.

ES.6 COGENERATION SUMMARY

General

For the WRF, cogeneration refers to generation of power from digester gas. Due to the relatively small amount of digester gas that is required for digester heating, a large fraction of the gas generated in the digesters will be available for energy generation.

The following technologies were evaluated for cogeneration at VSD:

- Internal Combustion Engines
- Microturbines
- Fuel Cells

Internal Combustion Engines are the most efficient of these three technologies. All three technologies require digester gas pretreatment, which can be complex. However, at the scale of gas production that would be available at the WRF at the build-out condition (20 mgd influent flow), the return on investment for cogeneration may be insufficient to warrant the investment.

At the current electricity cost of $0.107/kWh implementation of co-generation does not provide the required ROI of 50% to be financially feasible. Electricity costs need to rise slightly above $0.11/kWh to provide an adequate ROI.

Power Purchase Agreement

Another option for VSD would be contracting a specialized company through a Power Purchase Agreement (PPA) to purchase, install, maintain, and operate cogeneration units at the WRF. Such a company would guarantee an electrical output given a guaranteed gas production by VSD. VSD, in exchange, would buy the power produced by the cogen units. The advantages are that the capital investment, ownership, operation and maintenance of the cogen system is done by the power purchase contractor. The gas conditioning systems, in particular, can be difficult and require specialized knowledge. For small systems such as the one that could be installed at the WRF, there are many benefits of a contractor owning and operating the cogen system.

Typically, those types of agreement become beneficial when the production of the biogas production of the WRF reaches 100,000 cf/day. This type of gas production would be attained at VSD at the WRF after TWAS digestion begins and after the plant routinely receives 6 mgd.

Depending on the size of the system and the cost of power, the cogeneration electricity rate as purchased by VSD from the PPA could be as low as 80% of the utility rate, which would represent cost savings for VSD as well as biogas reuse.

A PPA would allow reusing the biogas without initial capital investment, would save power, and potentially reduce costs for VSD. In addition, the risks associated with operation of a complex gas conditioning and energy recovery system is transferred to the PPA.

Although the economies of cogeneration for an individual wastewater treatment facility may not be worthy of investment, many utilities have found that contracting with a cogeneration operator offers a viable means of benefiting from the production of digester gas.

ES.7 DESIGN CRITERIA SUMMARY

A list of all design criteria for the master planning phase are listed in **Table ES-4**. Note that the Biological Treatment Pond System may be demolished at any time.

Table ES-4 Phasing Plan

Process Unit or Parameter	Existing	Phase 2b	Phase 2c	Phase 3	Phase 4	Buildout
Design Flow (mgd)	5.9	5.9	8.2	10.0	13.3	20.0
Influent Pumps	5	5	5	5	5	6
Bar Screens ½”	2	2	2	--	--	--
Bar Screens ¼”	--	1	1	3	3	3
Aerated Grit Chamber	1	--	--	--	--	--
Vortex Grit Chamber (22-ft diameter)	--	1	1	1	2	2
Primary Clarifier (170’x20’x12’)	2	2	2	2	4	6
Aeration Tank Exist.	4	4	4	4	--	--
Aeration Tank New (281’x30’x20’)	--	--	--	--	4	6
Blowers, 4,500 cfm	3	3	3	3	--	--
Blowers, 6,000 cfm	--	--	--	--	5	7
Secondary Clarifier (95-ft diameter)	3	3	3	3	4	6
Ponds Available	2,3,N,S	2,3,N,S	3,N,S	3 (part),N	3 part	3 part
Biological Treatment Ponds	3	--	--	--	--	--
Chlorine Contact Capacity (mgd)	22.3	22.3	22.3	32.3	26.2	26.2
UV Disinfection Capacity (mgd)					13.5	20
Filters Capacity (mgd)	--	--	--	10	13.3	20
Gravity Belt Thickeners	--	2	2	2	2	3
Digesters (85-ft dia.)	1	2	2	2	3	4
Sludge Holding Tank	--	1	1	1	2	2
Belt Press (2 meter)	2	2	2	3	4	4
Solar Drying Bed Area (acres)	1.8	1.8	3	3	4	6

Figure ES-8 shows the capacity of each unit process with color code for each plant expansion phase.

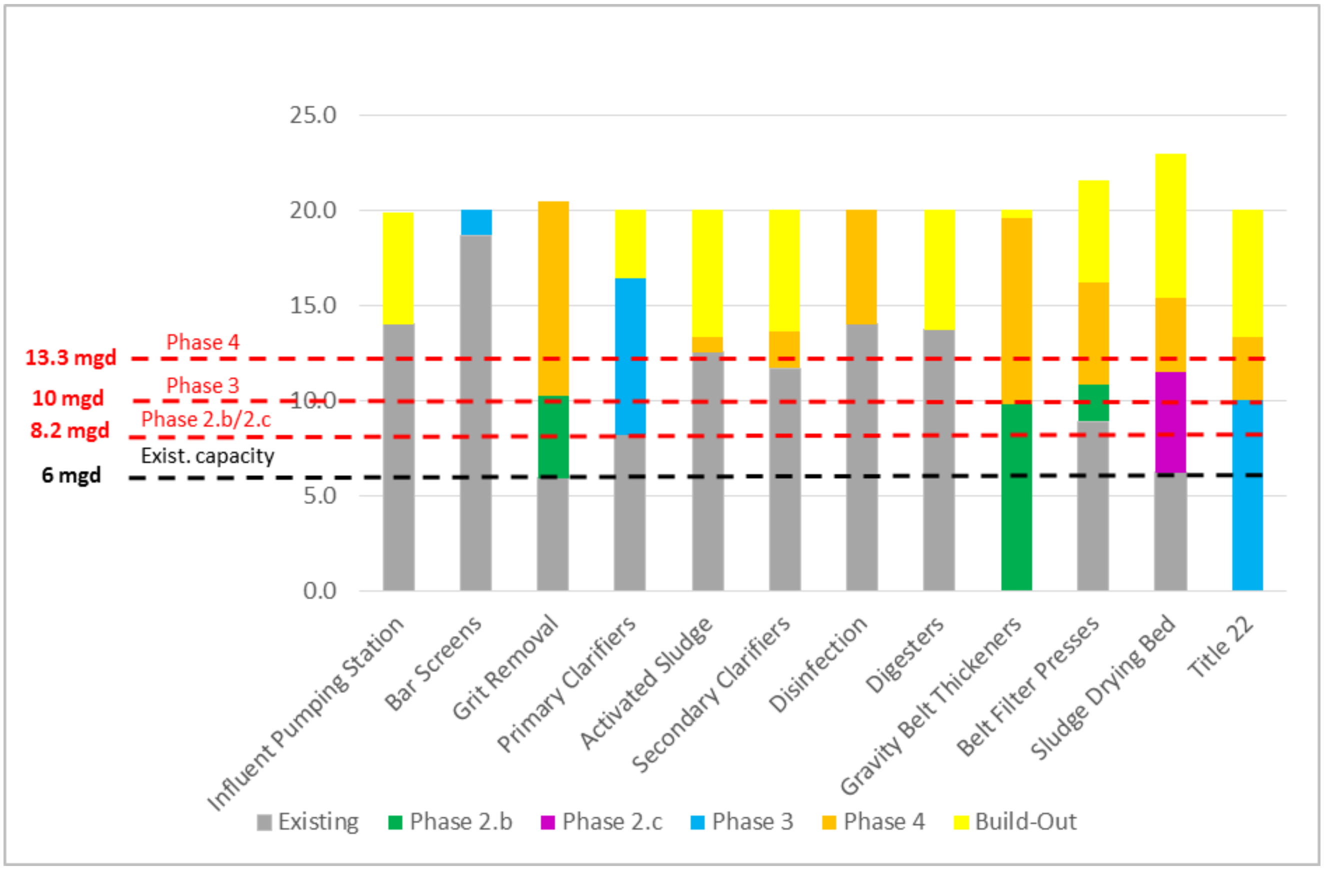


Figure ES-8 Process Treatment Capacities by Phase

Figure ES-9 shows the WRF flow projection together with the timing of each expansion phase.

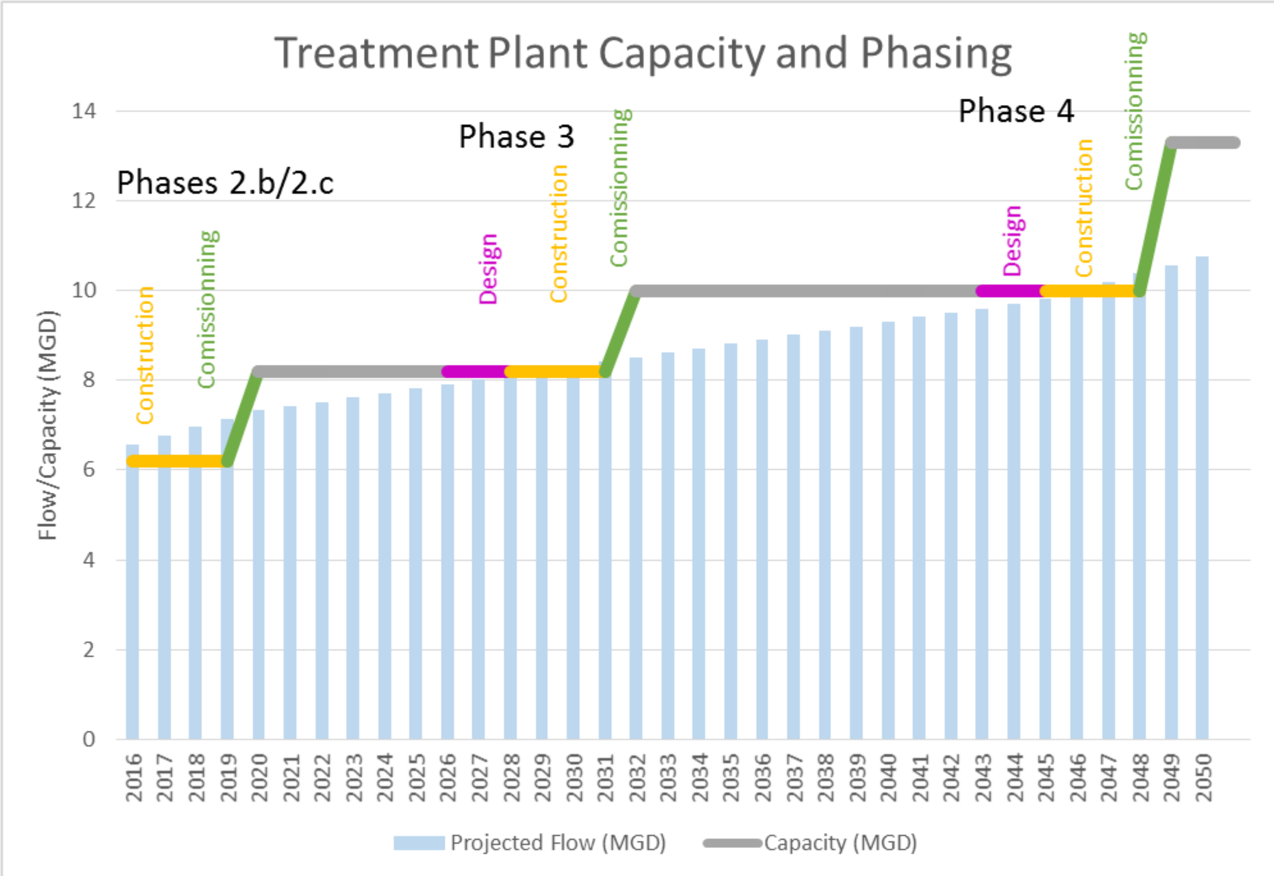


Figure ES-9 Flow Projection, Plant Capacity, and Proposed Phasing

Upgrades and costs per phase are summarized below:

<u>Phase 2b ($27.3 million)</u>

- Bar Screen, ¼-inch Spacing (1)
- Vortex Grit Chamber (1)
- Gravity Belt Thickeners (2)
- Digester (1)
- Sludge Holding Tank (1)
- Thickeners building
- BTPS decommissioning

<u>Phase 2c – 8.2 mgd ($15.7 million)</u>

- Solar Drying Bed Area (1.2 acres)
- Gas Storage Bladder
- Pond system decommissioning

Phase 3 – 10.0 mgd ($52.6 million)

- Bar Screen, ¼-inch Spacing (2)
- Filters (10 mgd)
- Chlorine Contact Capacity
- Belt Press (1)

Phase 4 – 13.3 mgd ($71.9 million)

- Vortex Grit Chamber (1)
- Primary Clarifiers (2)
- Aeration Tanks (4)
- Blowers (5)
- Secondary Clarifier (1)
- Filters (3.3 mgd)
- UV Disinfection Capacity (in existing Chlorine Contact Tank 3 – 13.5 mgd)
- Digester (1)
- Sludge Holding Tank (1)
- Belt Press (1)
- Solar Drying Bed Area (1.0 acre)

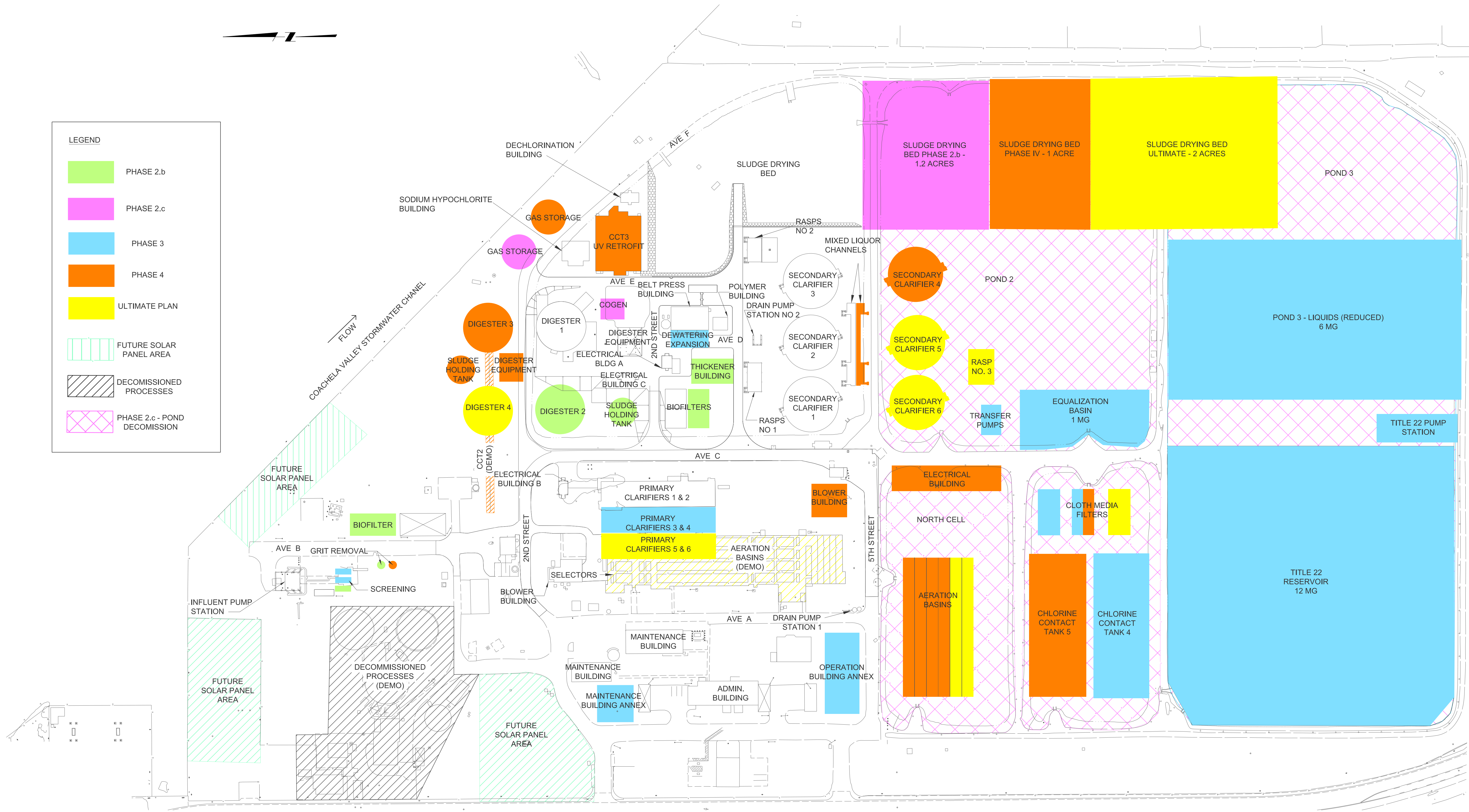
Buildout – 20.0 mgd ($47.6 million)

- Primary Clarifiers (2)
- Aeration Tanks (2)
- Secondary Clarifiers (2)
- Filters (6.3 mgd)
- Digester (1)
- Solar Drying Bed Area (2.0 acres)

ES.8 WRF SITE PLAN AND PHASING

The proposed site plan for the WRF and the phasing of improvements is shown on **Figure ES-10**. WRF expansion and improvements can fit on the existing north section of the WRF property, leaving the southerly portion now occupied by the BTPS for other uses.

Since the existing system for handling WAS involves use of one of the Ponds, and that same pond area will be required for the first phase of Solar Drying Bed addition, this places a constraint on construction sequencing. Before additional solar drying beds can be constructed, a new means of thickening and stabilizing WAS must be implemented. For this reason, two phases are required to bring the plant process units fully to 8.2 mgd at average flow (Phases 2b and 2c).



REV	DATE	BY	DESCRIPTION	SCALE 1"=80'	WARNING 0 ½ 1 IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE	DESIGNED S CALVET DRAWN S CALVET CHECKED	MWH	VALLEY SANITATION DISTRICT Figure ES-10 Site Plan and Phasing	WATER RECLAMATION FACILITY MASTER PLAN	SHEET JobNumber

ES.9 BIOLOGICAL TREATMENT POND SYSTEM DECOMMISSIONING

The existing Biological Treatment Pond System (BTPS) serves no wastewater treatment purpose. The plan for decommissioning the BTPS includes several steps and options.

All three ponds may be fully decommissioned, or a partial decommissioning may be done leaving one or two ponds in place. Partial decommissioning will not eliminate the annual cost of maintaining the ponds, but will reduce the cost.

However, the recommended action is to completely decommission all three of the BTPS ponds.

ES.10 CAPITAL AND OPERATION & MAINTENANCE COST SUMMARY

Table ES-5 below summarizes for each phase of WRF expansion the capital and annual O&M cost. Capital costs include construction, contingency, engineering and administration. Staffing requirements for Treatment Plant O&M is also estimated for each phase.

Note that Annual O&M Cost does not include an accounting for depreciation. Nor do the O&M costs include Laboratory, Collection System or Administration. The table includes the current plant O&M figure (excluding depreciation).

Table ES-5
Summary of Capital Improvement Program

Phase	Constr. Year	Flow Capacity, mgd	Project Cost WRF Only (2015 $M)	Project Costs Tertiary Only (2015 $M)	Total capital cost (2015 $M)	O&M Staffing	Annual O&M Cost (2015 $M)	Annual O&M per MG
Current	--	6.2	--	--	--	13	$3.22	$1,420
2b	2016	6.2	$27.0	--	$27.0	13	$3.22	$1,420
2c	2017	8.2	$15.7	--	$15.7	15	$4.09	$1,370
3	2027	10.0	$18.0	$34.6	$52.6	17	$5.57	$1,530
4	2045	13.3	$57.6	$14.3	$71.9	21	$7.26	$1,500
Build-out	--	20.0	$39.8	$7.8	$47.6	26	$10.0	$1,370

Section 1 - Introduction

Section 1
Introduction

This introductory section of the Water Reclamation Facility (WRF) Master Plan for Valley Sanitary District (VSD) provides a brief description of the project background, the scope of work, and a description of the report organization.

1.1 AUTHORIZATION

This Master Plan has been developed under Task Authorization No. 3 between VSD and MWH Americas, Inc. (MWH) dated October 17, 2014. All work under this Task Order is governed by the provisions of the Master Services Agreement for Environmental Engineering and Planning Consulting Services between VSD and MWH, dated April 19, 2012.

1.2 PROJECT BACKGROUND

The WRF is located adjacent to and on the southwest bank of the Whitewater River (also referred to as the Coachella Valley Stormwater Channel - CVSC). This stream ultimately discharges to the Salton Sea 15 miles to the east of the WRF. The WRF serves primarily the City of Indio (96% of the service area).[1] The remaining 4% of the service area includes small parts of the City of La Quinta and City of Coachella as well as small areas of unincorporated Riverside County. The City of Indio estimated year 2014 population was 82,398 and was used as a surrogate for the overall service area population for planning purposes.

In October 2006, Lee & Ro completed a Valley Sanitary District Wastewater Treatment Plant Master Plan. The intent was to phase future treatment upgrades given the observed growth at the time, and give an estimated cost for the recommended upgrades. In November 2013, MWH completed a Collection System Master Plan, and among other outcomes determined the build-out flow for the area.

The goal of this Master Plan is to review the current capacities of newly installed processes initially recommended in the Lee & Ro Master Plan, and to update treatment upgrades recommendations, phasing, and anticipated costs. Water recycling (Title 22) is also considered as part of this report.

1.3 EXISTING FACILITIES

The Water Reclamation Facility was originally built in the 1920s. The liquid treatment processes include headworks, primary sedimentation, aeration basins, secondary clarifiers, and chlorine contact tanks for disinfection. The solids handling facilities include one digester for Primary Sludge stabilization, a pond system partially used for Waste Activated Sludge digestion, and Belt Filter Presses for solid dewatering. Solids are then dried in sludge drying beds before being hauled off site. A detailed Plant history is provided in **Section 3**.

[1] Note that substantial portions of low-density land use within the Indio city limits are not served by any sewer system.

1.4 OBJECTIVE AND SCOPE OF WORK

The key objectives of the Master Plan are to:

- Assess the capacity of existing WRF unit processes.
- Determine the feasibility of continued use of the BTPS.
- Prioritize unit process expansion or improvement.
- Forecast future flows using the 2013 Sewer Master Plan.
- Forecast future wastewater constituent loads to provide a basis for treatment unit sizing
- Select and size future treatment unit processes.
- Recommend phasing of treatment process expansion and improvements.
- Provide cost estimates (capital and O&M) that can be incorporated into a phased Capital Improvement Program for the WRF.

The scope of work for this Master Plan consists of the following tasks:

- Task 1: Provide Project Management, Communication and Meetings.
- Task 2: Collect and Analyze Plant Operating Data and Record Drawings.
- Task 3: Determine Build-Out Capacity Requirements
- Task 4: Develop Performance Assessment of Biological Treatment Pond System
- Task 5: Develop Effluent Reuse Options (including treatment requirements)
- Task 6: Evaluate Biosolids Management Options
- Task 7: Evaluate Alternatives and Prepare Recommendation
- Task 8: Prepare Draft and Final Reports for the Master Plan

1.5 DATA SOURCES

In preparation of this Master Plan, VSD staff provided operational monthly reports, descriptions of unit process and equipment effectiveness, record drawings and supplemental laboratory date. Multiple meetings were held and extended interaction with VSD staff occured throughout the master planning process to obtain a thorough understanding of the District’s requirements.

1.6 WATER RECLAMATION FACILITY MASTER PLAN ORGANIZATION

This Master Plan is divided into eight sections plus an Executive Summary, similar to the tasks performed in the scope of work. **Section 2** provides a description of the VSD service area with population, flow and load projections. **Section 3** discusses the capacity of the existing WRF by unit process. **Section 4** describes Option 1 (secondary treatment without nitrogen removal). **Section 5** describes Option 2 (Secondary treatment with nitrogen removal, discharge to Whitewater River). **Section 6** describes Option 3 (Tertiary treatment for recycled water pursuant to the State of California Water Resources Control Board Title 22 requirements). **Section 7** discusses biosolids management, including thickening, stabilization, dewatering and drying / storage. Cogeneration options will also be discussed. **Section 8** presents the site master plan phasing and implementation and the Capital Improvement Program (CIP) along with anticipated annual O&M costs.\

Section 2 – Service Area Description and Flow Projections

Section 2
Service Area Description and Flow Projections

This section describes the Valley Sanitary District's (VSD) existing service area. A discussion of population, land use, climate, and geography within the service area is presented, as well as wastewater flow and contaminant loading projections.

2.1 EXISTING GEOGRAPHICAL DESCRIPTION

The size of the service area is approximately 19.9 square miles. VSD serves the city of Indio which borders the cities of Coachella, Bermuda Dunes, and La Quinta. The District Service area sits at an average elevation of 18 feet (ft.) above sea level with a high elevation of 142 ft. above sea level and a low elevation of 54 ft. below sea level. VSD and the City of Indio are bordered by three mountain ranges which contribute to its warm climate. VSD is approximately 20 miles south-east of the city of Palm Springs, 15 miles north-west of the Salton Sea, and 134 miles east of the city of Los Angeles. The VSD sewer service area boundary is shown on **Figure 2-1**.

2.1.1 Climate

VSD is located in a desert region where temperatures typically range between 60 to 90 degrees Fahrenheit (°F) as shown in **Table 2-1**. The warmest month of the year is July with an average maximum temperature of about 107.3 (°F), while December is the coldest month of the year with an average minimum temperature of 44.2 (°F). VSD's climate is affected by its proximity to the three mountain ranges that surround the area, which keep temperatures warmer throughout the year. Humidity is relatively low during high temperatures.

Annual precipitation data from the last twenty years (i.e., 1994 to 2013) is presented in **Table 2-2**. VSD experiences an average of approximately 3.47 inches of rainfall each year (based on annual precipitation data from 1912 to 2012), although annual precipitations have been lower than that over the 20 past years as seen in **Table 2-2**. Average monthly precipitation that occurs in the area is shown in **Table 2-3** and is based on 100 years of data. Although a low total rainfall is observed, the intensity of the few rainfall events significantly impacts flows to the WRF.

As a result of the warm climate, a wastewater temperature range was observed from 69 °F to 91 °F during the period between September 2012 and September 2014.

Table 2-1
Average Monthly Temperatures[2]

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Max °F	71.9	75.3	81.3	87.5	95.7	103.1	107.3	106.6	102.0	91.9	79.6	71.0	**89.5**
Mean °F	58.3	61.6	68.1	74.1	81.7	88.6	93.8	93.4	88.0	77.8	65.7	57.6	**75.8**
Min °F	44.6	48.0	54.8	60.7	67.7	74.2	80.3	80.3	74.0	63.7	51.8	44.2	**62.1**

Table 2-2
Annual Precipitation[3]

Year	Rainfall (inches)
1994	**1.57**
1995	**4.39**
1996	**1.19**
1997	**1.64**
1998	**Non Detect**
1999	**1.11**
2000	**0.59**
2001	**1.04**
2002	**0.98**
2003	**1.63**
2004	**2.87**
2005	**1.15**
2006	**Non Detect**
2007	**Non Detect**
2008	**Non Detect**
2009	**1.12**
2010	**5.08**
2011	**1.48**
2012	**1.83**
2013	**2.42**
Average	**1.51**
Mean	**1.19**

[2] Source: National Oceanic and Atmospheric Administration National Data Center Climatological Normals Data Tables for Station USC00044259 (Indio Fire Station).

[3] Source: U.S. Historical Climatology Network, data from station 044259, INDIO FIRE STATION, California

Table 2-3
Average Monthly Precipitation[4]

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Ave. Rain-fall (in)	0.55	0.63	0.43	0.04	0.08	0.00	0.04	0.55	0.04	0.28	0.20	0.63	3.47

Discrepancies between rainfall records were resolved by taking the more conservative figure. For solar drying, for example, the more conservative figure is 3.47 inches per year.

2.1.2 Land Use

The City of Indio is largely open space which encompasses 52.5 percent of the land based on the zoning land-use Geographic Information System (GIS) information from the City. **Figure 2-2** shows existing land use in the VSD sewershed.

Using the existing land use, MWH consolidated the use categories into eight simplified categories for ease of developing flow projections. The eight categories are: Commercial, Industrial, Mixed Use, Residential (high, medium, and low density), Open, and Public. **Table 2-4** shows the breakdown of generalized land use categories and the percentage of area each category occupies in the existing VSD service area. The generalized land use for the existing VSD sewershed is mapped on **Figure 2-3**.

Based on the land use, approximately 20 percent of the VSD service area is residential low (i.e., low-density residential), 8 percent is residential medium (i.e., medium-density residential such as townhomes, multi-family homes, condominiums, mobile homes), and 7.7 percent is residential high (i.e., high-density residential such as apartment buildings). The commercial land use category comprises 4.8 percent of VSD. Industrial land use makes up 3.3 percent of the VSD service area.

[4] http://www.usclimatedata.com/climate/indio/california/united-states/usca0512

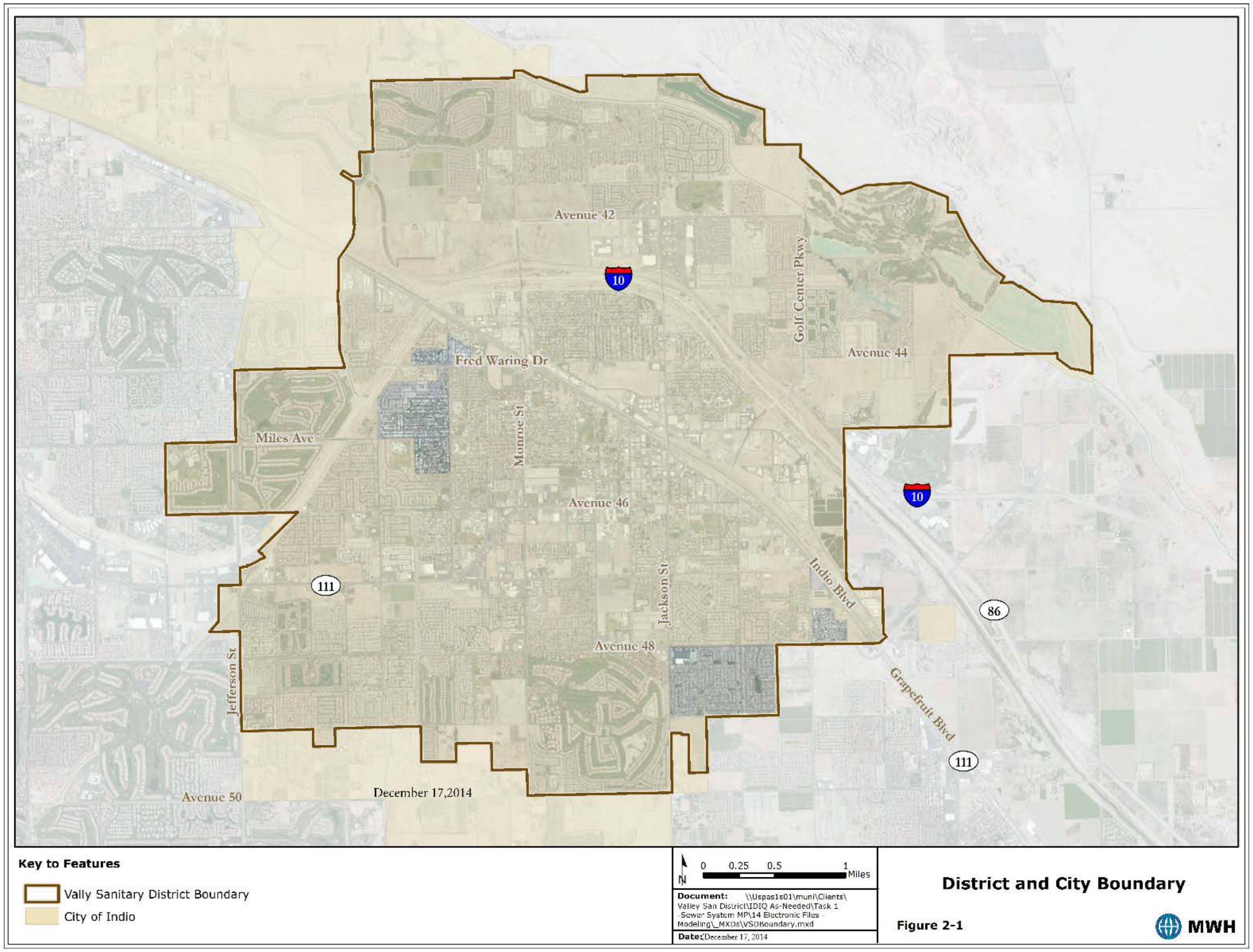


Figure 2-1
Valley Sanitary District Sewershed and City Boundary

Table 2-4
Existing Land Use

Land Use	Area (acres)	Area (sq. mi.)	Percentage of Total Area of VSD (%)
Commercial	617	0.96	4.8
Industrial	425	0.66	3.3
Mixed Use	119	0.19	0.9
Open	6,763	10.57	52.5
Public	359	0.56	2.8
Residential High	987	1.54	7.7
Residential Low	2,582	4.04	20
Residential Medium	1,030	1.61	8.0
Total	**12,882**	**20.13**	**100.0**

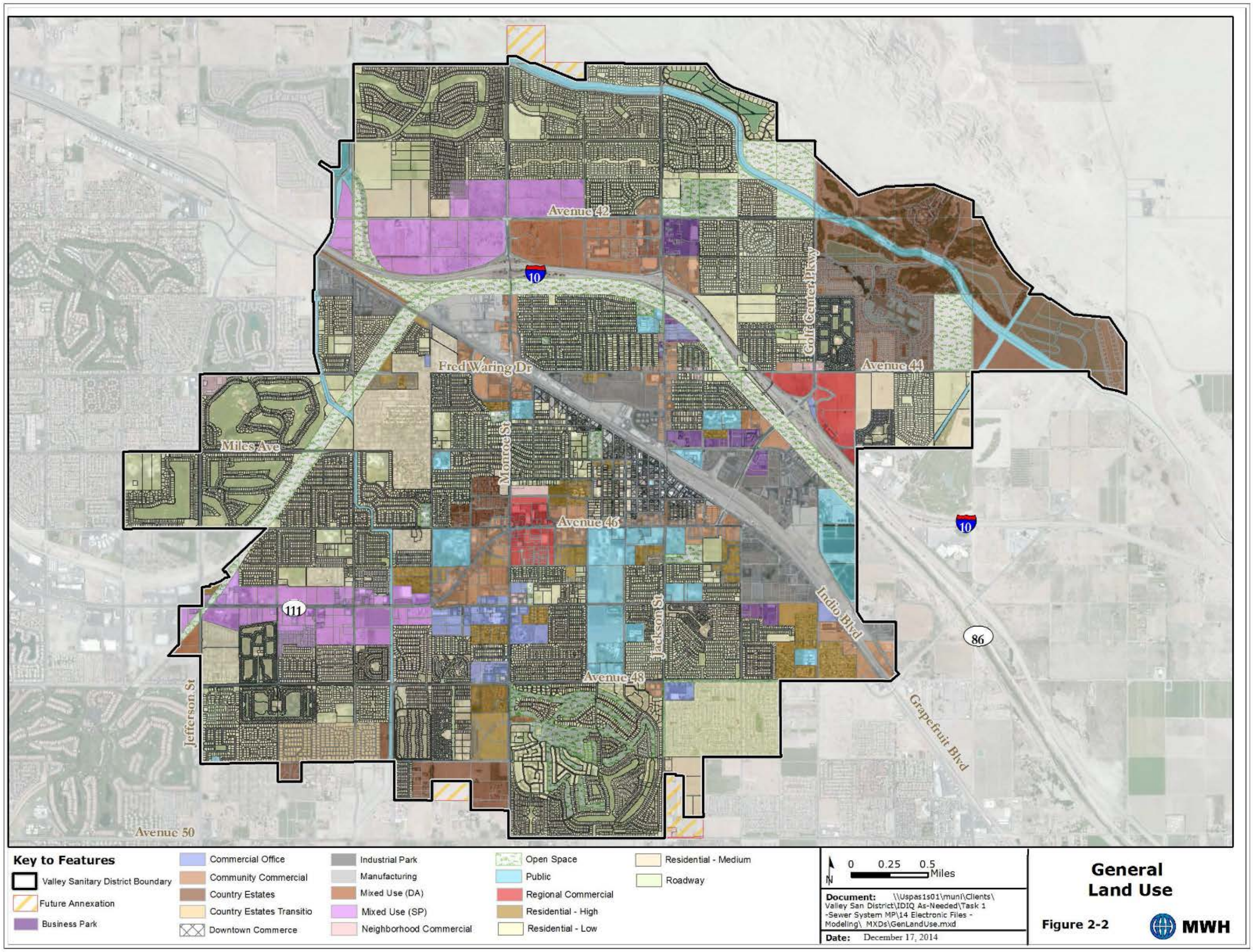


Figure 2-2
Existing Land Use

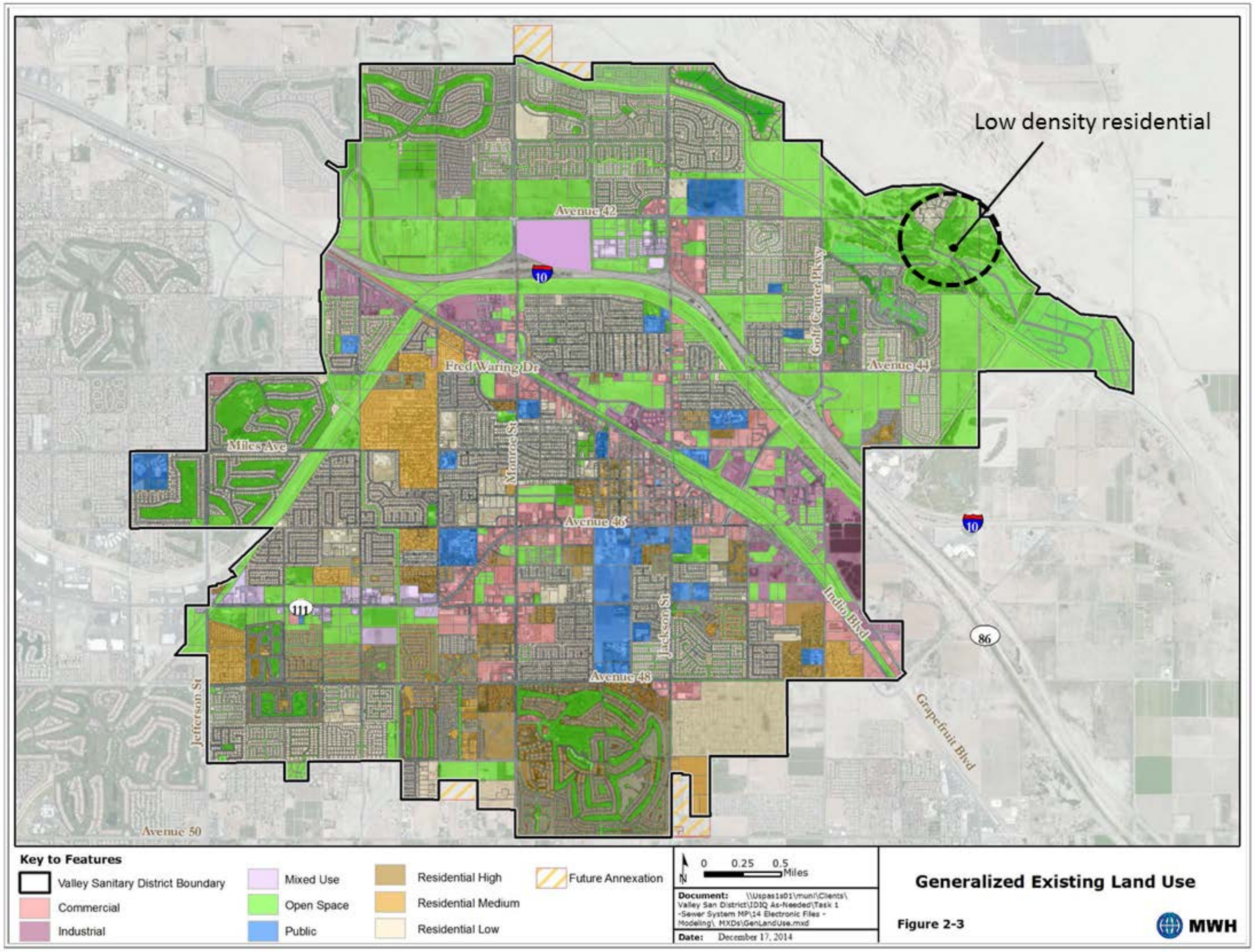


Figure 2-3
Generalized Existing Land Use

2.1.3 Population

Population projections are based on 2012 Coachella Valley Association of Governments (CVAG) data for the City of Indio. Since projections are not available for unincorporated areas within the VSD service area, this area is assumed to have a similar growth rate as the City of Indio. Population projection data were evaluated from year 2010 through 2035 in five-year increments, as shown in **Table 2-5**. Population within the VSD service area is expected to increase almost 60 percent from year 2010 to 2035. Based on these projections MWH estimates that the 2014 population served by VSD at the time of this analysis was approximately 82,398. For the purposes of this Water Reclamation Facility Master Plan, 82,398 people is used as the current, served population.

Table 2-5
Existing and Projected Population for the City of Indio

Year	City of Indio
2010	76,036
2014 (current)	**82,398**
2015	87,486
2020	100,387
2025	106,923
2030	113,681
2035	120,676
2040	128,097
2045	135,976
2050	144,338

Source: 2012 Coachella Valley Association of Governments until 2035. 2035 to 2050 population values estimated based on 2030 to 2035 growth rate. Population projections from the Coachella Valley Association of Governments have been derived from economic growth prediction by the California Department of Finance and if the economy varies from those projections, so will the population served and the flows to the WRF.

2.2 PROJECTED FUTURE CONDITIONS

Population projections developed are used to project future flows. In the MWH 2013 Collection System Master Plan (CSMP), build-out conditions were developed. Planned development information gathered from VSD staff was used to project population at 5-year intervals. Sewage flows were predicted based on population, but the build-out flow was predicted based on land use and individual development information. **Table 2-6** summarizes land use for the build-out scenario.

Table 2-6
Build-out Land Use

Land Use	Area (acres)	Area (sq. mi.)	Percentage of Total Area of VSD (%)
Commercial	1,063	1.66	8.25
Industrial	542	0.85	4.21
Mixed Use	777	1.21	6.03
Open	3,574	5.58	27.74
Public	457	0.71	3.54
Residential High	4,437	6.93	34.45
Residential Low	1275	1.99	9.90
Residential Medium	758	1.18	5.88
Total	**12,882**	**20.13**	**100**

2.2.1 Flow Projections

Existing flow data for VSD's WRF was provided to MWH for a period from September 2012 to September 2014. From that historical data, an average dry weather flow (ADWF) was calculated to be 6.01 mgd. Based on the current served population of 82,398 people (discussed in Section 2.2.3), this gives an average per capita wastewater flow of 73 gallons per capita per day (gpcd).

A peak wet weather flow was observed on August 19, 2013 from the hours of 6:00 pm to 8:00 pm. The flow exceeded the Plant influent flowmeter circle chart maximum of 22 mgd. A visual extrapolation of the flow chart was used to estimate a peak flow of 24 mgd. Based on a calculated ADWF of 7.48 between 6:00 and 8:00 pm that week (excluding the day of the rain event), a maximum observed wet weather inflow into the system (flow above dry weather flow, presumed to be due to direct surface flow into the sewer) of 16.52 mgd was estimated, as described on **Figure 2-4**.

Assuming that the wet weather inflow will remain constant in the future, this theoretical inflow was used for build-out projections, adding to the peak dry weather flow to determine peak wet weather flow as shown on **Figure 2-4 next page**. This inflow was abnormally high for a sewer system that size, and was probably due to sewer manholes covers being opened for construction, or for drainage (illegally) at that time.

The estimated inflow is not expected to increase proportionately with dry weather flow. The high flow event provided a benchmark for determining peak inflow. While the peak inflow observed on 8/19/2013 has a low probability of recurrence, prudence dictates that this value of inflow, 16.5 mgd, be considered for build-out peak weather flows.

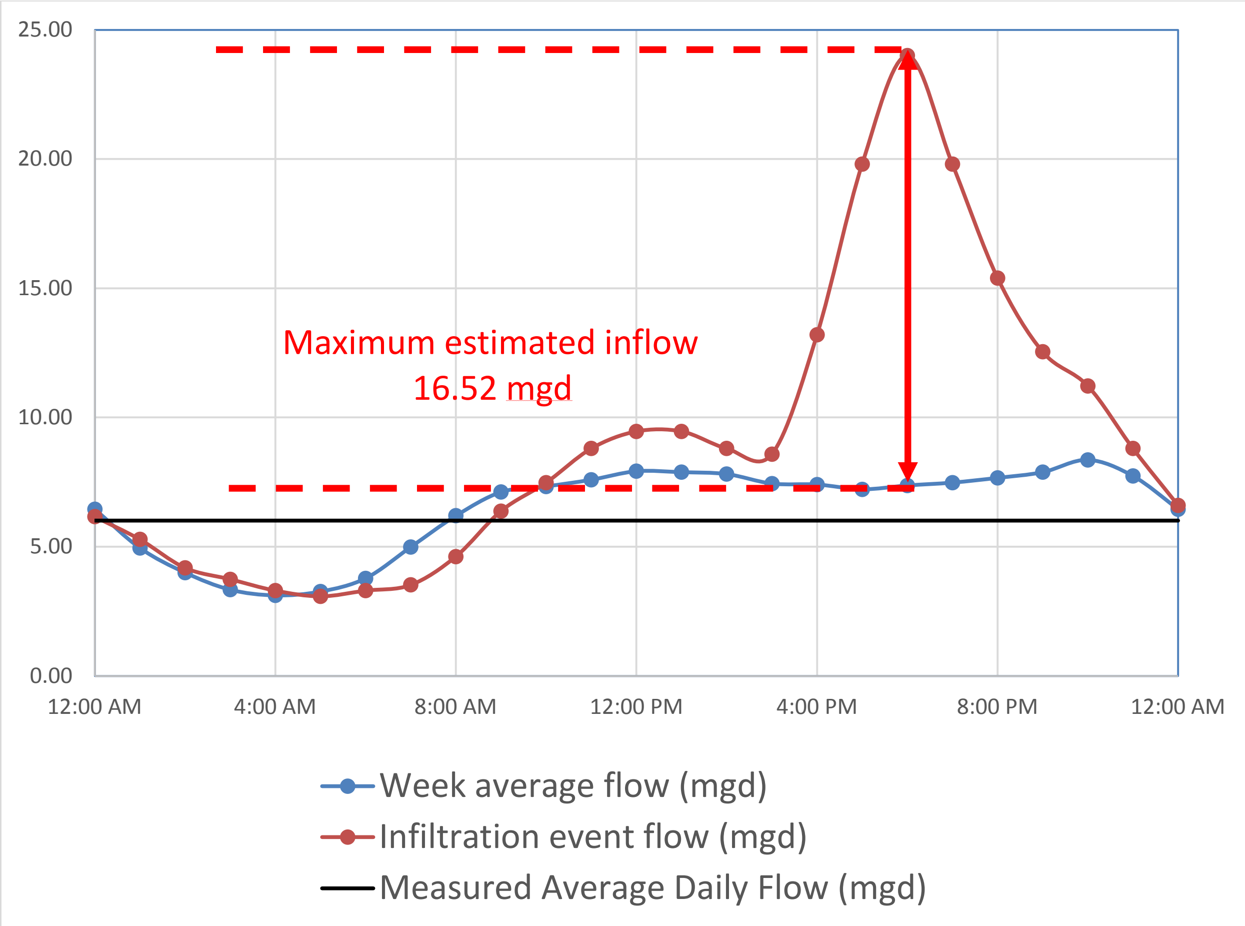


Figure 2-4 Inflow Event Flow Analysis on 08/19/2014

Using the projected flows outlined in the 2013 CSMP, a build-out average daily flow (ADF) of 20 mgd was used for the purpose of this Water Reclamation Facility Master Plan. A dry weather peaking factor of 1.35 was calculated from the historical data by calculating an average peak flow of 8.1 from the VSD supplied data and comparing it to the ADWF mentioned previously. The peaking factor was then applied to the future projected flow to get a future projected peak dry weather flow of 27.1 mgd at build-out. In addition to the 27 mgd of peak projected flow, the maximum observed wet weather inflow was added to this peak flow to calculate a maximum expected wet weather, build-out flow for the WRF. This final flow projection used for the WRFMP is 43.5 mgd, and is the Build-out Wet Weather Peak Flow for the collection system.

In addition to flow from the collection system, recycle / return flows from the WRF processes will be returned to the trunk sewer that parallels Van Buren Road and will be pumped through the Influent Pump Station and the entire facility. That amount is estimated at this planning stage to be 1 mgd, yielding 44.5 mgd peak flow through the WRF.

This analysis yields a Wet Weather Peaking Factor (WWPF) of 2.22, rounded to 2.2. **Table 2-7** gives a summary of these flow projections.

Table 2-7
Basis of Flow Projections

Existing Conditions		
ADF	6.01	mgd
Dry weather peaking factor	1.35	-
Served population	82,398	persons
Average per capita flow	73	gpcd
Wet weather observed maximum peak (Occurred between 6 pm and 8pm)	24	mgd
ADF (between 6 pm and 8 pm)	7.5	mgd
Max Observed Wet Weather Inflow	16.5	mgd
Build Out		
Build-out ADF	20	mgd
Build-out Dry Weather Peaking factor	1.35	-
Build-out Dry Weather Peak Flow	27	mgd
Build-out Peak Wet Weather Flow	44.5	mgd
Build-out Wet Weather Peaking Factor	2.2	-

2.2.2 Load Projections

Influent water quality data was analyzed for the same period as flow data (September 2012 to September 2014). Current 5-day biochemical oxygen demand (BOD_5) concentrations and loadings were determined from 5-day carbonaceous BOD (cBOD)[5] data collected daily at the WRF and literature review. For the balance of this report, when the abbreviations "BOD" and "cBOD" are used, they refer to the 5-day BOD or cBOD unless specifically denoted otherwise. Reported average BOD/cBOD ratios range from 1.1 to 1.35 (Muirhead, et al., 2006), and an average value of 1.2 was used in this report. Weekly TSS

[5] Carbonaceous BOD is determined from a sewage sample by running a normal BOD analysis, but with an added nitrification inhibitor. This isolates the nitrogenous BOD from the carbonaceous BOD. The reporting of cBOD is done at VSD because the facility does not have an ammonia or organic nitrogen limit in its NPDES permit.

(Total Suspended Solids), and Monthly Total Kjeldhal Nitrogen (TKN) measurement data were also analyzed. **Table 2-8** summarizes current WRF influent water quality.

Table 2-8
Summary of Current Influent Water Quality Parameters

cBOD[1]	Average - mg/L	218
	Max week – mg/L	295
BOD[2]	Average - mg/L	261
	Max week - mg/L	354
TSS	Average – mg/L	200
TKN	Average - mg/L	46

[1]inhibited BOD results

[2]BOD estimated using a 1.2 BOD/cBOD ratio

MWH also performed statistical analysis to determine the 50, 90, and 99th percentile concentrations and loadings for BOD, TSS, and TKN. The 50% percentile value is the value at which 50% of the total recorded values fall below. Similarly, the 90% value is the value at which 90% of the other values fall below, and so on. **Table 2-9** presents the result of this analysis for both BOD, TSS, and TKN. **Figure 2-5**, **Figure 2-6**, **Figure 2-7**, **Figure 2-8**, **Figure 2-9**, and **Figure 2-10** show the raw sewage distribution of BOD, TSS, and TKN graphically.

Table 2-9
Existing BOD and TKN Loading

Water Quality Analysis		**50 % percentile**	**90% percentile**	**99% percentile**
BOD	Concentration (mg/L)	256	313	354
	Loading (lb/day)	12,751	15,285	23,209
TSS	Concentration (mg/L)	201	246	290
	Loading (lb/day)	9,914	12,426	15,029
TKN	Concentration (mg/L)	49	52	53
	Loading (lb/day)	2,362	2,700	2,800

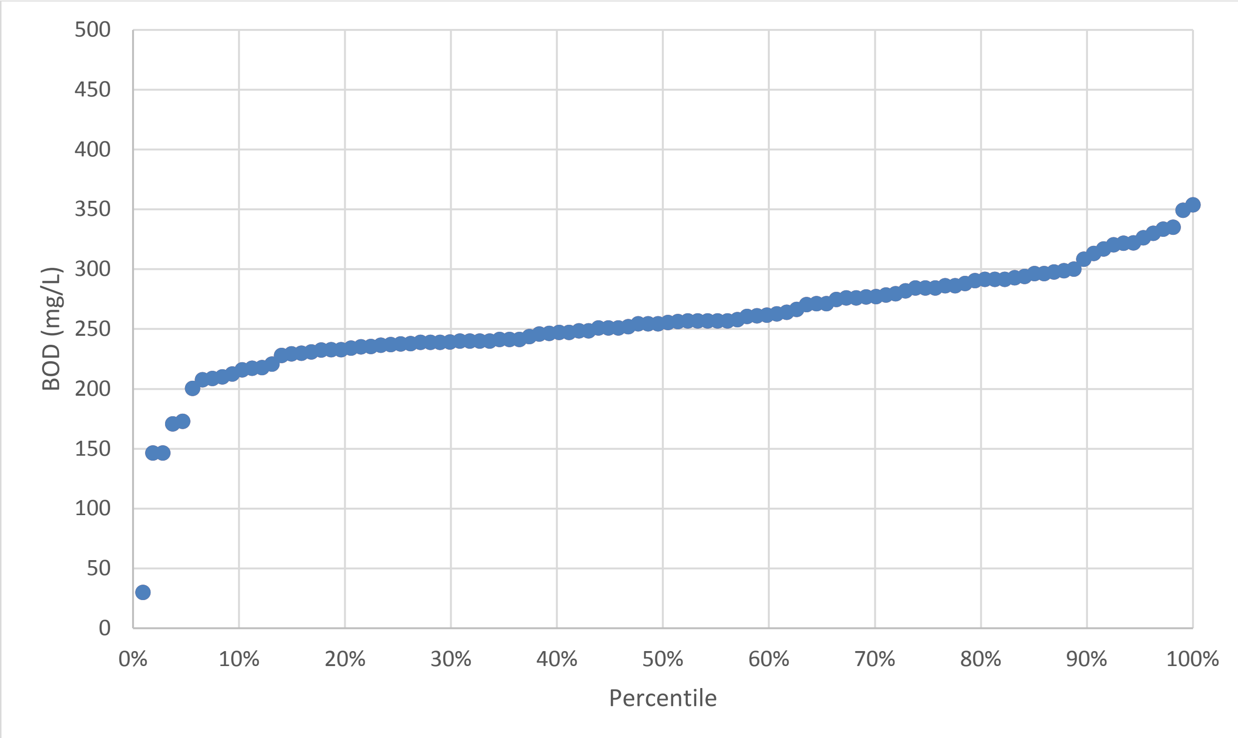


Figure 2-5 BOD Concentration Percentiles

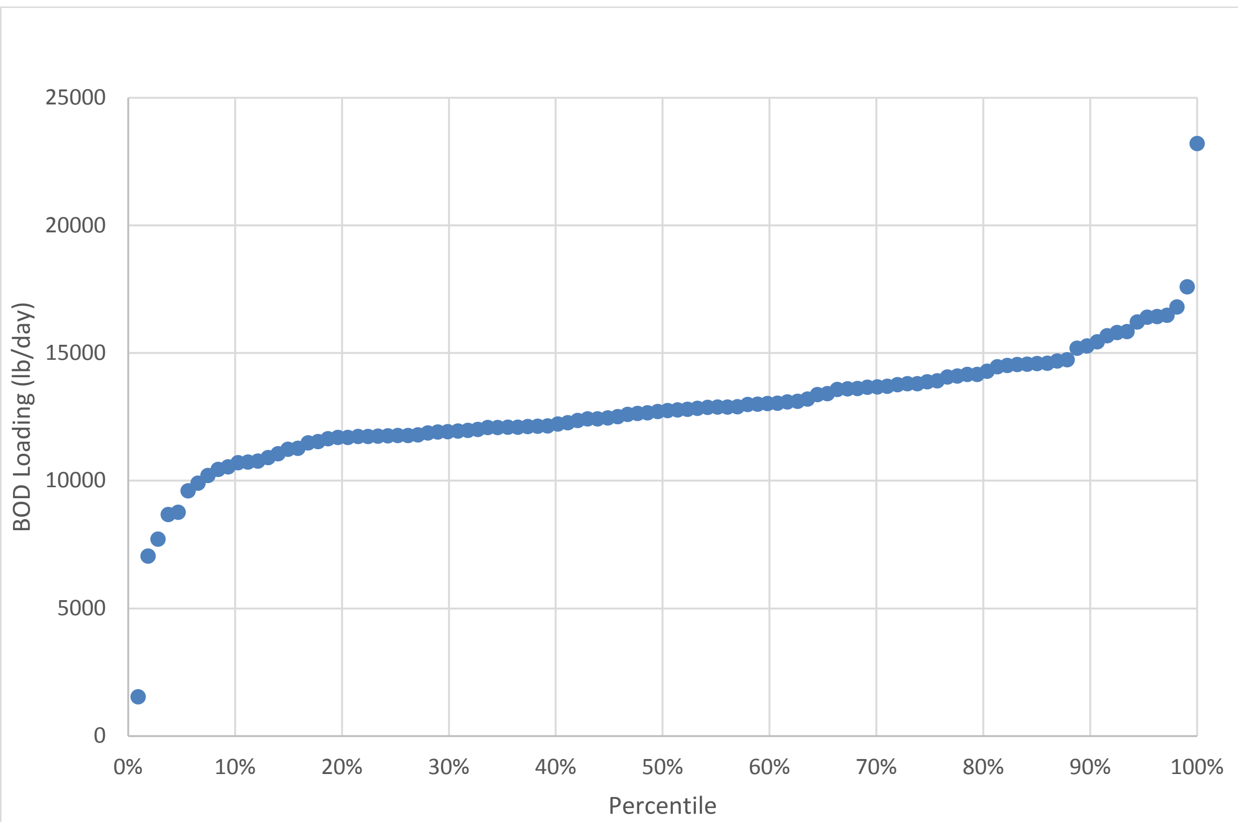


Figure 2-6 BOD Loading Percentiles

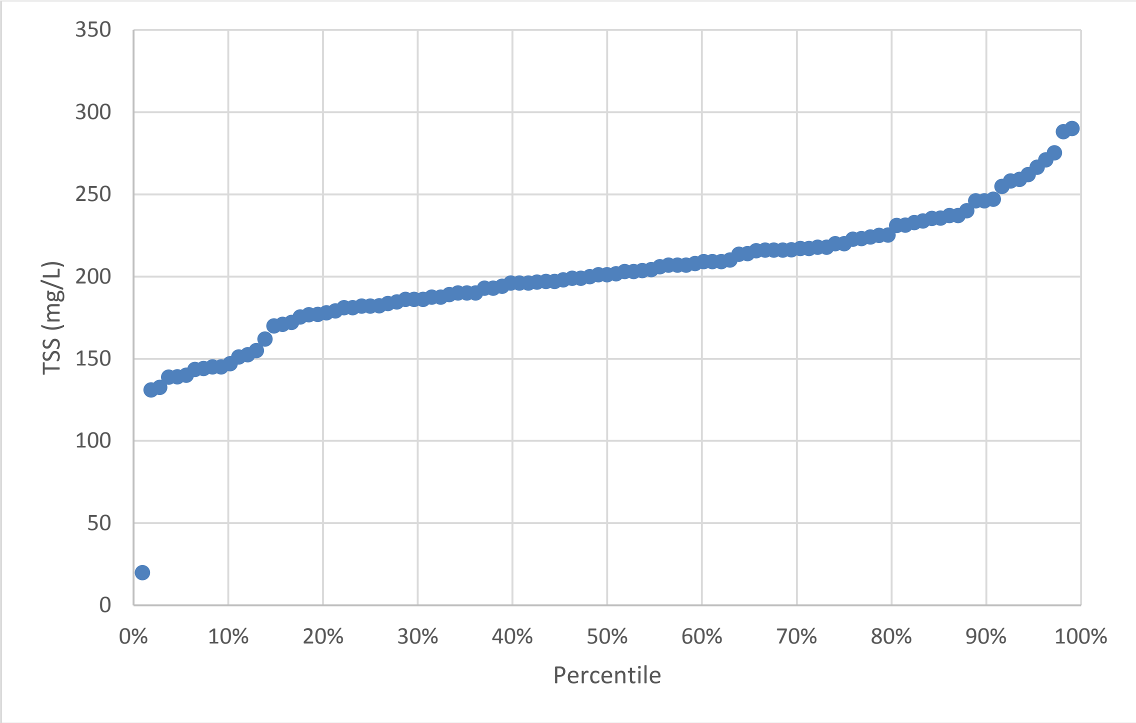


Figure 2-7 TSS Concentration Percentiles

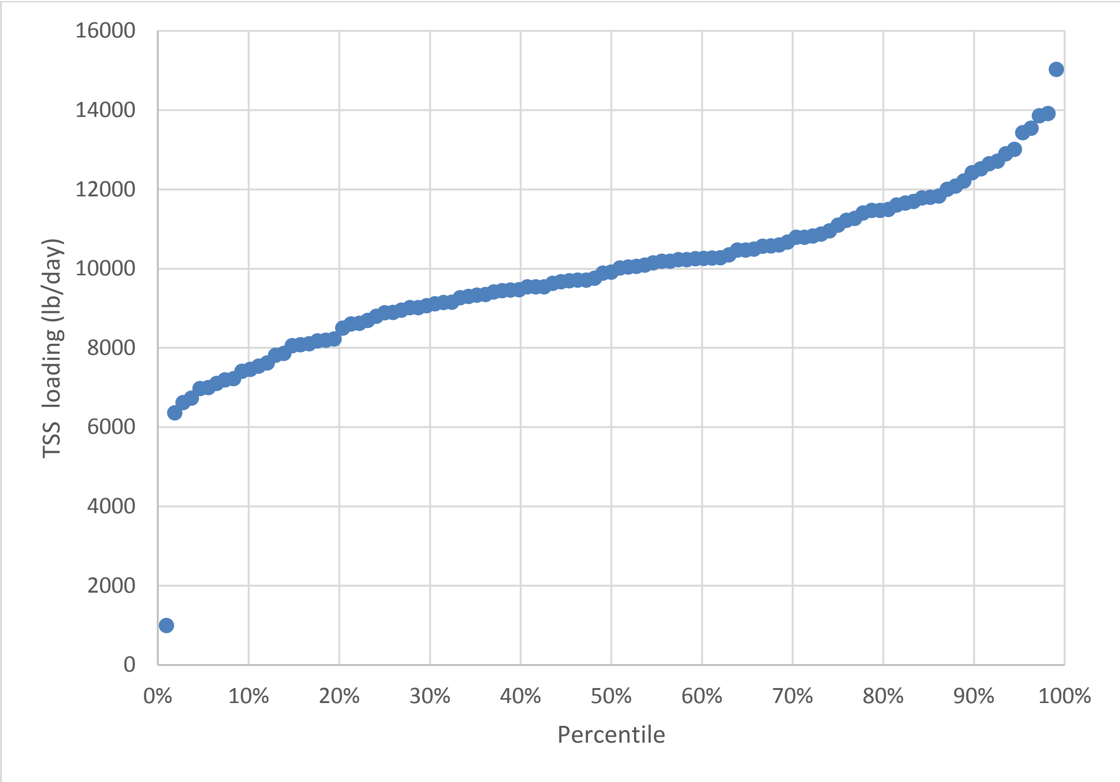


Figure 2-8 TSS Loading Percentiles

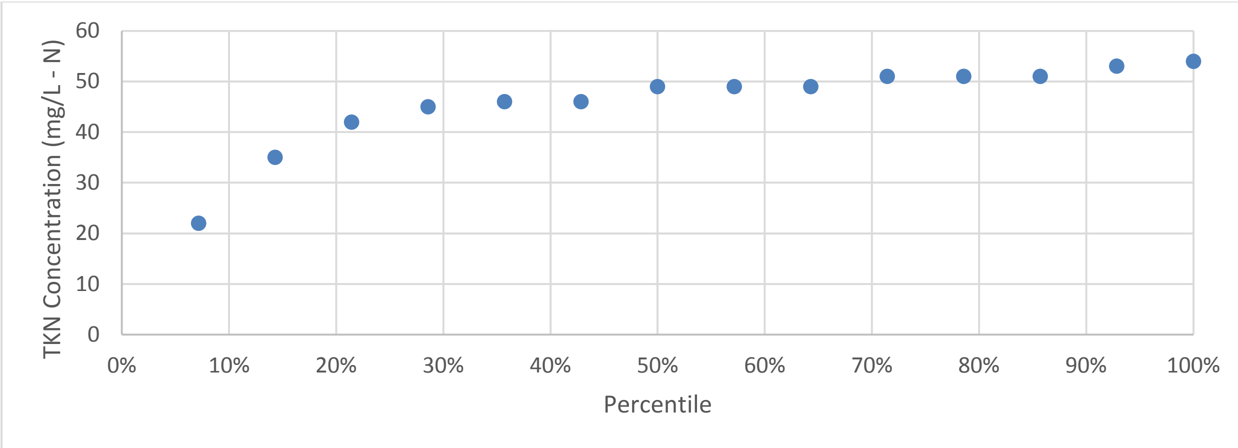


Figure 2-9 TKN Concentration Percentiles

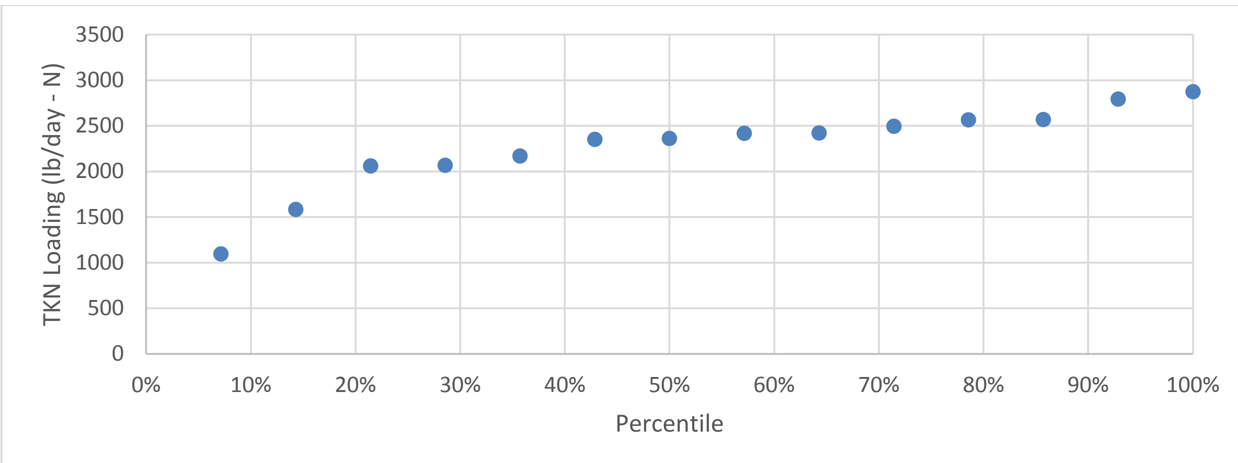


Figure 2-10 TKN Loading Percentiles

In order to determine the projected loading of BOD and TKN for the system, population and flow projection developed previously in this section were applied to the measured loading. **Table 2-10** presents these projections for the 90th percentile TKN and 99th percentile BOD. These statistical measurements were used as they provide a conservative estimate for VSD of what might be expected at the WRF in the future.

Table 2-10
Projected Flow, BOD and TKN Values

Year	Population	ADWF (mgd)	PDWF (mgd)	PWWF (mgd)	90th Percentile TKN (lb/day)	90th Percentile TSS (lb/day)	99th Percentile BOD (lb/day)
2015	87,486	6.4	8.6	25.2	2,884	13,273	18,802
2020	100,387	7.3	9.9	26.4	3,309	18,421	21,574
2025	106,923	7.8	10.5	27.1	3,525	19,620	22,979
2030	113,981	8.3	11.2	27.8	3,758	20,915	24,496
2035	120,676	8.8	11.9	28.4	3,978	22,143	25,934
2040	127,371	9.3	12.6	29.1	4,200	23,444	27,373
2050	143,214	10.4	14.54	31.1	4,853	27,013	31,637
Build-out	274,000*	20	27	44	9,031	50,272	58,878

*Population at build-out was estimated from the build-out flow (20 mgd) obtained from the 2013 Master Plan and applying 73 gpcd to calculate population.

For Design, 90th percentile BOD, TSS, and TKN are typically used. For the preliminary process capacity evaluation and sizing presented in the subsequent section, concentrations of 313 mg/L, 246 mg/L, and 52 mg/L are used for BOD, TSS, and TKN respectively.

Projected flows per year are depicted on **Figure 2-11**.

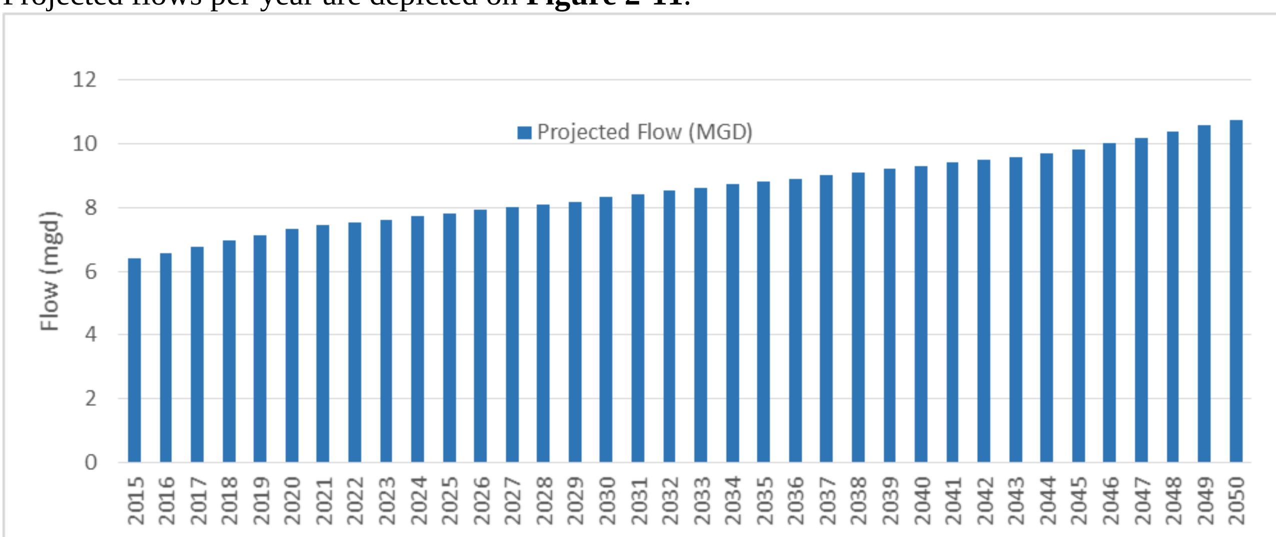


Figure 2-11 Projected Average Dry Weather Flows to Year 2050

Section 3 – Water Reclamation Facility Capacity

Section 3
Water Reclamation Facility Capacity

In this section the history of the WWTP is provided, the existing permit requirements are summarized, and the current treatment capacity of each of the WRF process units is evaluated. Since different flow conditions (average flow or peak flow) may limit the treatment capacity of each process unit, a peaking factor of 2.3 (2.27, rounded) was used to evaluate the average treatment capacity of units where size was controlled by peak flows. This value was conservatively selected based on a maximum wet weather flow of 13.6 mgd observed in September 2012 at the WRF and compared to the average flow in the data period of 6.0 mgd. The August 2013 flow of 24 mgd, used previously to calculate peaking factor at build-out, was deliberately excluded for evaluation of current capacity.

Detailed calculations by process unit are presented in **Appendix B.**

3.1 TREATMENT PLANT HISTORY AND BACKGROUND

Original Plant Prior to 1970

The WRF began in the 1920s with facultative treatment ponds. In 1950 a trickling filter was added. In 1958 three facultative oxidation ponds were added. Secondary treated effluent was chlorinated in a two-pass chlorine contact tank for disinfection of the effluent prior to discharge to the adjacent Whitewater River.

Activated Sludge Plant 1973

The WRF was substantially expanded in 1973 to include the following facilities:

- Dry-pit influent pump station
- Mechanical bar screen
- Grit removal
- Rectangular concrete structure divided into parallel tanks and processes
- Biosolids stabilization using wet-air oxidation (Zimpro)
- Vacuum filter for biosolids dewatering

The rectangular concrete structure included primary clarifiers, activated sludge aeration tanks, rectangular secondary clarifiers and a chlorine contact tank (CCT). A channel with gate and flow measuring flume was provided to permit a portion of the primary effluent to be diverted to the pond treatment system (in parallel with the activated sludge plant). Five ponds were available for a combination of secondary treatment.

Subsequent to startup, the Zimpro system was found to work well for primary sludge but not for waste activated sludge (WAS). This biological sludge was instead pumped to Pond 1 for storage and stabilization. Surface aerators were added to Pond 1 to facilitate the stabilization of WAS and the high-strength filtrate that was discharged from the vacuum filters downstream of the Zimpro wet air oxidation process. The potent foul air from the Zimpro was also bubbled into Pond 1.

Aerated Grit Chamber 1983
In 1983 an aerated grit chamber was added upstream of the primary clarifiers, together with a soil absorption bed (biofilter) for odor control. At that time, plastic media was added to the existing trickling filter.

Headworks Replacement 1998
In 1998 the Headworks Replacement project at the WRF provided the following facilities:

- 54-inch diameter trunk sewer
- 6-bay submersible Influent Lift Station
- Bar Screen Facility
- Biofilter

The design was intended to provide firm capacity to convey, pump and screen up to 40 mgd peak flow, thought to represent a buildout annual average flow of 18 mgd. Of six pump bays, four were outfitted with submersible non-clog type pumps, designed to provide firm capacity to pump 20 mgd instantaneous peak flow, with space to add more and larger pumps to provide the buildout peak flow. A magnetic flow meter with recorder was provided to measure plant influent flow.

The screening facility was built with three channels. Two mechanical climber screens and a manual rack screen for emergency were provided in the three four-foot wide channels. A 48-inch pipe connected to the existing 30-inch pipe to carry screened wastewater to the existing aerated grit chamber. Other treatment units remained in service.

Biological Treatment Pond System 2000
In 2000, the District constructed a Biological Treatment Pond System (BTPS), designed to treat 1 mgd of primary effluent, using a constructed wetland concept for secondary treatment. The system consists of three constructed ponds with liners, planted with bulrushes and other selected vegetation throughout each pond. The system was designed to consume and convert wastewater organic content, similar to an activated sludge process. Periodic harvesting of the vegetation provided ultimate removal of nutrients.

To provide primary effluent for the BTPS, sewage was pumped to the 1950s-vintage circular primary clarifiers, making use of this previously abandoned but intact facility, and primary effluent routed to the BTPS. Treated effluent from the BTPS was routed to the two-pass CCT No. 2 along with effluent from the aerated pond system.

A separate report on environmental and regulatory aspects of the BTPS was prepared by L&L Environmental and is included in **Appendix A**. L&L Environmental conducted a Section 404, preliminary jurisdictional wetland delineation of the BTPS. The purpose of the jurisdictional delineation was to quantify the portion of the facility that may be subject to jurisdiction of the U.S. Army Corps of Engineers, the California Department of Fish and Wildlife or the Regional Water Quality Control Board. The L&L report concluded that there are no jurisdictional "Waters of the U.S." or State jurisdictional Streambeds / Wetlands that apply to the BTPS.

Activated Sludge and Sludge Dewatering Facilities Upgrade 2008
In 2007 – 2008 the existing activated sludge plant was substantially modified to add the following facilities:

- Two Influent Pumps to the existing Influent Pump Station
- Three blowers to the existing Blower Building
- Three circular secondary clarifiers
- Belt Press Building with two belt presses for dewatering
- Chlorine Contact Tank No. 3 and 54” effluent pipe
- New 54” outfall
- Sodium Hypochlorite Building

The project also converted the primary clarifiers to anoxic selectors. This eliminated primary clarification at the WRF for the near term, and removed the need to operate the Zimpro wet air oxidation process.

Primary Sedimentation and Sludge Digestion Facilities 2014
In 2013/2014 resumption of primary clarification and addition of anaerobic digestion were completed by the addition of:

- Two rectangular primary clarifier tanks
- One 85-ft diameter anaerobic digester
- Gas scrubber
- Boiler
- Digester gas flare

3.2 EXISTING PERMIT REQUIREMENTS

The WRF discharge water quality and flows are regulated by the California Regional Water Quality Control Board Order R7-2015-0002, NPDES No. CA0104477. This Order expires on May 31, 2020, and limits the activated sludge effluent, the oxidation ponds/biological treatment unit system effluent, and the combination of those.

3.2.1 Activated Sludge Plant Effluent Limitations

The activated sludge plant effluent limitations are summarized in Table 3-1.

In addition to Table 3-1 requirements, the average monthly percent removal of cBOD and Total Suspended Solid (TSS) through the activated sludge plant shall not be less that 85%.

Table 3-1 Summary of Effluent Limitations Activated Sludge Plant

Parameter	Units	Average Monthly	Average Weekly
Flow	mgd	10	-
cBOD	mg/L	25	40
	lbs/day	2,085	3,336
TSS	mg/L	30	45
	lbs/day	2,502	3,753

3.2.2 Oxidation ponds and Biological Treatment Unit Effluent Limitations

The oxidation ponds and biological treatment unit effluent limitations are summarized in **Table 3-2.**

Table 3-2 Summary of Effluent Limitations Oxidation Ponds/Biological Treatment

Parameter	Units	Average Monthly	Average Weekly
Flow, Oxidation Pond	mgd	2.5	-
Flow, Biological Treatment Pond System	mgd	1.0	-
cBOD	mg/L	40	60
	lbs/day	1,168	1,751
TSS	mg/L	61	91
	lbs/day	1,780	2,656

In addition, the average monthly percent removal of cBOD and Total Suspended Solid (TSS) through the oxidation ponds and biological treatment unit shall not be less that 65%.

3.2.3 Combined Discharge Effluent Limitations

The limitations of the combination of the activated sludge plant and the oxidation ponds/BTPS are summarized in **Table 3-3**.

Table 3-3 Summary of Effluent Limitations Combined Discharge

Parameter	Units	Average Monthly	Average Weekly	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum
Residual Chlorine	mg/L	0.01	-	-	-	0.02
	lbs/day	1.1	-	-	-	2.3
Oil and Grease	mg/L	-	-	25	-	-
	lbs/day	-	-	2,815	-	-
pH	standard units	-	-	-	6.0	9.0
Copper, Total Recoverable	µg/L	10.1	-	17.4	-	-
	lbs/day	1.1	-	2.0	-	-
Heptachlor	µg/L	0.00021	-	0.00042	-	-
	lbs/day	0.000024	-	0.000047	-	-

3.2.4 Disinfection

The Order limits the geometrical mean of the bacterial density based on a minimum of no less than five samples equally spaced over a 30-day period. The Order limits both E. Coli. and Fecal Coliforms:

- For E. Coli., the geometric mean shall not exceed a Most Probable Number (MPN) of 126 per 100 millimeters, and none of the sample taken shall exceed a density of 400 MPN/100mL.
- For Fecal Coliforms the geometric mean shall not exceed a Most Probable Number (MPN) of 200 per 100 millimeters, and no more than 10% of sample taken shall exceed a density of 400 MPN/100mL.

The limitations presented were used as an evaluation basis when determining each unit process treatment capacity in Section 3. However, these will expire in June 2020 and may be subject to changes that would impact processes used, as well as their size. Sections 4, 5, and 6 present three probable permit and discharge scenarios at build-out, and recommend sizes and process required to comply with their associated limitations at build-out flow.

3.3 LIQUID HANDLING SYSTEM

The WRF liquid handling system comprises influent pumps, screening, grit removal, primary sedimentation basins, secondary treatment, oxidation ponds, a biological treatment unit, and disinfection. Secondary treatment is achieved by an activated sludge treatment process including aeration basin with selectors, and secondary clarifiers. Disinfection is achieved with chlorine through a

Chlorine Contact Tank (CCT3). Disinfected water is dechlorinated with sodium bi-sulfate before being discharged to the CVSC. Liquid can also be diverted through oxidation ponds. These act as a parallel secondary treatment system with the activated sludge system, providing peak shaving capabilities. A second Chlorine Contact Tank, CCT2, allows disinfection of pond effluent before discharging to the CVSC. A process flow diagram depicts the liquid handling system in **Figure 3-1**.

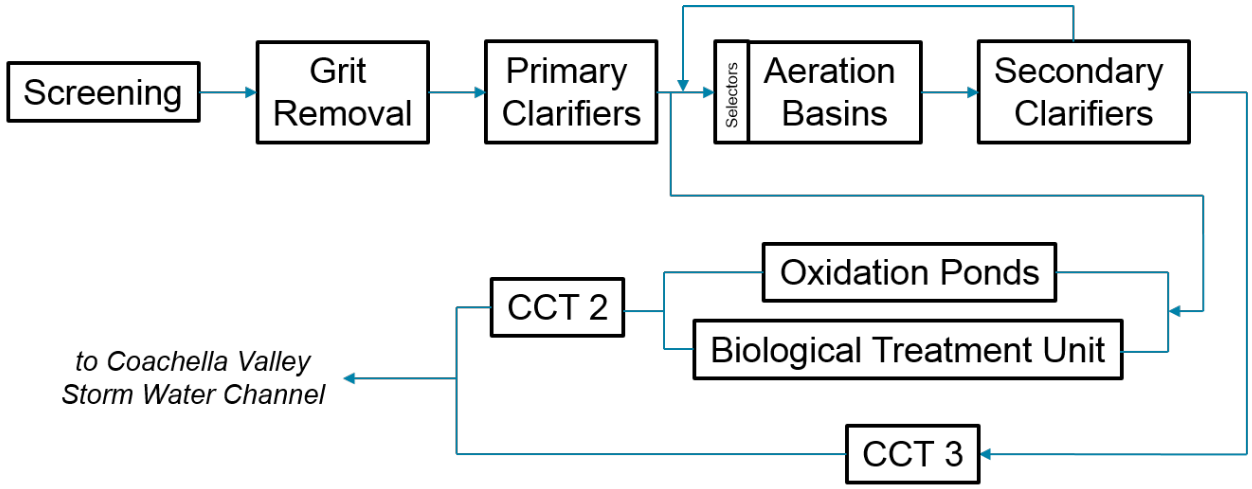


Figure 3-1 Liquid Handling Process Flow Diagram

3.3.1 Influent Pumps and Headworks

3.3.1.1 Influent Pumps

Peak flow events limit influent piping capacity. The pump station must have the capacity to pump the entire peak flow with its largest pump out of service.

The influent pump station is located on the Northwest side of the WRF. The pump station has six pump bays, three on each side of a central channel. Five submersible pumps are currently installed (one pump bay is empty), with 5,600 gpm capacity each. With five pumps on-line and one off-line, the influent pump station has a firm peak flow capacity of 32 mgd, or 14 mgd equivalent of average flow using a 2.3 peaking factor.[6] The influent pump station is shown on **Figure 3-2** Influent Pump Station

[6] Note that 2.2 is the future condition peaking factor, but 2.3 was employed for current WRF operation and unit process capacity assessment. As flows increase, a modest drop in peaking factor is expected.

Figure 3-2 Influent Pump Station

3.3.1.2 Bar Screens

Bar screens remove objects and rags from the raw wastewater, preventing damage to downstream processes. The capacity of the bar screens, like the influent pumps, is limited by peak flow events.

The Screening Facility has three channels as shown on **Figure 3-3**. Two channels are fitted with mechanical climber screens. A third channel is fitted with a manual rack for temporary bypass of one of the climber screens. Therefore, two of the three channels are assumed to be in service at peak flow events.

Assumptions used to calculate the existing bar screen capacity are listed below:

- Bar Spacing (actual) 1/2 inch
- Bar Thickness (actual) 3/8 inch
- Channel Width 4.0 feet
- Available Freeboard Upstream of Screen 1.0 ft
- Number of Screens in Service 2
- Screen Blockage (Fouling Factor) 40%

- Downstream Maximum Water Surface 476.85[7]

Each screen's capacity is limited in the near term not by the maximum available head but by practical considerations such as limiting the velocity of water through the bars. When the head loss through the bar screens exceeds six inches, excessive pass-through of malleable solids may occur. While this will not cause overflow, it will cause a loss of effectiveness during the short term periods when high flows occur. The limit of flow through each screen currently is 13.6 mgd at six inch loss. That translates to 27 mgd peak and 11.7 mgd average flow.

Figure 3-3 Screening Facility (Bar Screens)

For short periods higher flows that cause the head loss through the screens to exceed six inches can be tolerated. Each screen can pass 21.5 mgd with a head loss of 13 inches. Two screens in service can handle 43 mgd. However, a third mechanical screen should be installed when average flows exceed 10 mgd.

[7] Maximum downstream water surface elevation was taken from the hydraulic profile sheet G6 of the Primary Sedimentation and Sludge Digestion Facilities (Phase 2A) drawings dated June 2011. Since the WTP grade line is at an elevation of about 33 ft below sea level, it is suspected that 500 ft below sea level is used as an elevation baseline.

WRF staff observe that the existing ½-inch screen spacing allows more solids to pass through than desired. Future screens should incorporate finer openings to the degree that head is available without surcharging the screening facility or encroaching on the 1.0 foot freeboard.

3.3.1.3 Grit Removal

Grit is defined as granular material such as sand or similar material that can settle more easily than most organic solids in wastewater. Removal is important because grit will otherwise settle in the downstream primary clarifiers and ultimately be transported to the anaerobic digesters. Grit settling in the bottom of digesters will reduce their capacity and require down time and considerable expense for removal.

The existing grit chamber has been operated 24 hours per day since the 2012 expansion. As a safety measure, an overflow was installed to by-pass to the Influent Pump Station.

The single aerated grit removal tank downstream of the bar screens has dimensions of 25 x 12.5 x 12 ft side water depth (SWD). This provides treatment capacity of 5.9 mgd at average flow assuming a 3-minute typical hydraulic residence time at peak flow with 2.3 peaking factor.

3.3.2 Primary Clarifiers

Primary clarification provides a low-cost means of removing organic and inorganic material form the waste stream. The process is physical, consisting of quiescent settling and flotation (scum removal).Two rectangular primary sedimentation basins (170 ft x 20 ft x 12 ft SWD) are located directly south of the headworks. One basin is depicted on **Figure 3-4**. Each basin is equipped with one 150 gpm scum pump and one 250 gpm sludge pump. Both primary clarifiers can be assumed to be in service for the design flow condition when determining capacity.[8]

Ferric chloride is added at the aerated grit chamber to enhance the effectiveness of primary clarification and to reduce hydrogen sulfide in the digester gas that is derived from primary sludge.

Primary clarifier capacity is limited by the overflow rate, defined as the flow rate in gallons per day divided by the surface area of the clarifier tank. Typical primary clarifier design would have a range of 800 to 1,200 gal/day/ ft^2 at average flow and 2,000 to 3,000 gal/day/ ft^2 at peak flow.

[8] EPA reliability guidelines are that with one primary clarifier out of service, the plant should be capable of successfully treating 50% of design flow.

Figure 3-4 Primary Clarifier

The primary clarifiers at the WRF have demonstrated satisfactory performance at 1,600 gal/day/ ft^2 when calculated for average flows, perhaps due to a relatively low dry weather peaking factor under normal circumstances. This could allow the primary clarifiers to operate successfully at a higher flow range on a regular basis, but could result in poor performance when unexpectedly high flows occur.

For the purposes of setting a capacity for the existing primary clarifiers, surface overflow rates at the top of the normal design range were selected: 1,200 gal/day/ft^2 at average flow and 3,000 gal/day/ ft^2 at peak flow. The existing primary clarifiers capacity is limited by average flow conditions (not peak flow, due to the 2.3 peaking factor). The average flow capacity calculated would be 8.2 mgd.

It is worth noting that the average BOD removal from the new primary clarifiers for their first year of operation averaged 23%. It is possible that operating at a lower overflow rate could improve future BOD removal efficiency. Literature values indicate that with a detention time of 1.5 hours at average flow, the primary clarifiers should achieve a BOD removal efficiency of approximately 30%.[9] The primary clarifiers if both are in service at the rated flow of 8.2 mgd would provide a 1.8 hour detention time. For this reason, in subsequent discussions we assume a conservative 25% BOD removal in the primaries.

3.3.3 Secondary Treatment

3.3.3.1 Activated Sludge Process

The activated sludge process removes over 90% of the BOD and suspended solids that remains after the primary clarifiers. The process employs aeration and settling. Settled solids containing organic-material-consuming organisms are returned to the aeration tank to provide degradation and consumption of

[9] George Tchobanoglous, et. al. (Metcalf & Eddy), *Wastewater Engineering Treatment and Reuse*, Figure 5-46, pg. 405.

organic matter in the wastewater. A portion of the biosolids “grown” in the activated sludge process is removed for subsequent solids handling.

Figure 3-5 Activated Sludge Selectors and Aeration Basins

Four activated sludge process (ASP) basins are located south of the headworks and west of the primary clarifiers. All four units are considered when determining capacity.[10] Each basin includes a selector (anoxic zone), and an aeration basin as shown of **Figure 3-5.**

The minimum water temperature during the winter was assumed as 20 °C with operation parameters of 4 days solids retention time (SRT) and maximum 2,500 mg/L mixed liquor suspended solids (MLSS). It should be noted here that the SRT assumes that the MLSS is contained in the selectors, aeration basins, and the clarifiers.

The influent BOD was conservatively estimated using the plant influent 90th percentile value and 30% BOD removal in the primary clarifiers. The Primary Clarifiers currently remove a little less than 30% of influent BOD (around 25% removal), but the clarifiers are also operated on the high end of the allowable range of recommended surface loading rate. Under normal operations, 30% BOD removal is considered conservative.

[10] EPA reliability guidelines indicate that with one aeration tank out of service, the plant should be capable of successfully treating 75% of the design flow. With one aeration tank out of service, the capacity calculation can therefore consider all four units in service (one unit out of service leaves 75% in service).

Based on these parameters, and based on the net yield currently observed at the plant (1.0 lb of TSS/lb of BOD removed), the aeration basins are currently limited to a capacity of 12.5 mgd for the current configuration that does not include nitrogen removal.

If the existing ASP basins were retrofitted for nitrogen removal, the ASP would be limited to a capacity of 7.7 mgd, using a typical higher MLSS of 3,000 mg/L and a 10 day HRT. This retrofit would require a mixer in the anoxic zones for each basin, and 4 total internal recycle pumps, one for each basin, at 4,700 gpm and 20 hp for each pump.

Aeration blowers currently installed at the WRF are summarized below:

- Number of Aeration Blowers 3
- Type Single Stage Centrifugal
- Capacity, Each 4,500 cfm
- Pressure 8.2 psig
- Motor Horsepower, Each 200

With one blower off-line, the blowers can accommodate an ASP capacity of 10 mgd. With all blowers on-line, they can accommodate an ASP capacity of 15 mgd.

3.3.3.2 RAS Pumps

The return activated sludge (RAS) pumps provide the means to return biomass to the aeration tanks for degradation and consumption of organic matter. Of five RAS pumps three are dedicated to secondary clarifier no. 1 and 2, and two are dedicated to secondary clarifier no. 3 as shown on **Figure 3-6**. The pumps are assumed to operate in a 3+2 configuration, and therefore the capacity calculations are evaluated based on 3 duty pumps for the three clarifiers. Each pump has a capacity of 2,500 gpm with a 30 hp motor. At a typical RAS recycle ratio of 1.0, the RAS pumps are limited to a plant flow of 10.8 mgd. However, it would be acceptable to run the ASP at lower RAS recycle ratio, and a flow similar to the maximum capacity of the ASP, 12.2 mgd, is reasonable.

Figure 3-6 Clarifier No. 3 RAS Pumps

3.3.3.3 Secondary Clarifiers

Secondary clarification allows the biomass that consumes organic material in the wastewater to be separated from the clarified water. The clarified water (secondary effluent) can then be prepared for disposal or recycle.

Three circular secondary clarifiers are located on the eastern part of the WRF. One clarifier is depicted on **Figure 3-7**. Using typical hydraulic loadings of 550 gal/day/ft^2 at average flow and 1300 gal/day/ft^2 at peak flow, the secondary clarifiers hydraulic capacity is limited to 11.7 mgd of average flow. Using typical solid loadings of 1 lb/ft^2/hr at average flow and 1.6 lb/ft^2/hr at peak flow, and a MLSS concentration of 2,500 mg/L, the clarifiers solid capacity is limited to 12.2 mgd.

Figure 3-7 Secondary Clarifier

3.3.3.4 Pond Treatment System

The pond system at the WRF provides two functions: peak shaving and waste activated sludge storage and stabilization. The ponds operate in parallel with the activated sludge system for their secondary treatment function.

Four ponds of the original five ponds remain in service for treatment of primary effluent and for secondary WAS solids stabilization and storage. The ponds are interconnected for ease of series or parallel treatment configuration. Most of the ponds are aerated to a limited degree. For the purposes of establishing capacity, the parameters for aerated lagoons were used.

The key design parameter for aerated lagoons is detention time, with a design range of 10 to 30 days.[11] Areas with higher ambient temperatures, such as VSD, can be expected to perform well at the lower end of the detention time range. However, the ponds are only partially aerated. For the purposes of evaluating capacity of the ponds, a detention time at average flow of 20 days was selected.

[11] United States Environmental Protection Agency, *Wastewater Technology Fact Sheet: Aerated, Partial Mix Lagoons*, September 2002.

Figure 3-8 Pond 2 and Surface Aerator

A motorized gate at the primary effluent channel allows a fraction of the incoming flow to be diverted to the pond treatment system. This system acts as a peak shaving device. The gate diverts primary effluent through a Parshall flume to allow continuous monitoring of the rate of flow to the pond treatment system.

The Pond Treatment System is permitted to discharge up to 2.5 mgd of primary effluent for secondary treatment prior to discharge to the Whitewater River, in accordance with the existing National Pollutant Discharge Elimination System (NPDES) permit. The system can also accept higher flows during shorter periods of time (scalping of high load). The ponds, their surface areas and volumes are listed in **Table 3-4**.

Table 3-4 Pond Treatment System

Pond	Surface Area (acres)	Volume (MG)	Water Depth (ft)	Surface Aerators
2	6.0	20	9.5	6
3*	12.0	36	9.5	1
North Cell	2.0	6	9.5	2
South Cell	2.0	6	9.5	2

*One additional aerator could be installed in Pond 3 if needed.

Assuming a Hydraulic Residence Time (HRT) of 20 days, the ponds can handle a maximum flow of 3.4 mgd, assuming that Pond 3 would be outfitted with surface aerators. If Pond 2 is de-rated due to its service as a waste activated sludge storage and stabilization lagoon, the current NPDES permit rating of 2.5 mgd is reasonable. Pond 2 is shown on **Figure 3-8**.

Currently, the Ponds are mainly used for WAS storage, peak shaving, and drain lines (from Drain Pump Stations 1 and 2) discharge. WAS is stored and stabilized in Pond 2, and Pond 3, North Cell, and South Cell are used for peak shaving and drainage.

3.3.3.5 Biological Treatment Pond System

The Biological Treatment Pond System (BTPS) was designed to treat 1 mgd of primary effluent. Older circular primary clarifiers that had been abandoned in the early 1970s were rehabilitated to provide primary treatment upstream of the BTPS, thus providing a parallel treatment train to the main primary / secondary treatment system in place at that time. Within 3 years of operation, primary effluent discharge into the BTPS was discontinued due to odors and plugging issues. Instead Oxidation Pond effluent was fed to the BTPS. An additional problem with vector attraction was particularly noted due to the BTPS providing habitat for the mosquito that carries the West Nile Virus.

The system consists of three cells (A, B and C) with capacities as given in **Table 3-5** with bulrushes and other selected vegetation growing throughout each pond. Each pond's bottom consists of a layer of soil underlain by an impervious liner.

The vegetation is designed to allow microorganisms and plants to consume biodegradable waste from the Oxidation Pond effluent that is routed through the system.[12]

Table 3-5 Biological Treatment Pond System

System Parameter	Value	Comment
Number of Ponds	3	A, B and C
Total Pond Area	15.2 acres	
Cell A	4 acres	
Cell B	4.9 acres	
Cell C	6.3 acres	

The BTPS naturally stimulates the growth of the bulrushes which must periodically be harvested. Small amounts of vegetative material that is lost through natural attrition and the harvesting process fall to the pond bottom and form a decaying vegetative layer. This layer can interfere with the normally aerobic treatment process that the BTPS was designed to promote. In June 2014, the recirculation of the BTPS effluent to the oxidation pond system had been instituted in an attempt to improve the effluent quality so that it could meet the NPDES permit limitations. There is currently no discharge from the BTPS.

The BTPS has not been used for treatment of primary effluent since approximately 2004, due to odor complaints, plugging, and concerns raised by the Coachella Vector Control Agency regarding vectors

[12] Portions of the description of the Biological Treatment Pond System were derived from a report entitled *Jurisdictional Delineation with Least Environmentally Damaging Practical Alternative for the Valley Sanitary District Facility* prepared by L&L Environmental, Inc., May 2015.

that may spread West Nile Virus from the un-disinfected primary-treated wastewater in the ponds.[13] Surface spraying sprinklers were added to disturb the water surface and discourage mosquito breeding.

The required harvesting of the flora in the BTPS is a labor intensive task that must be performed regularly to keep the system working. Overall, the BTPS requires approximately $400,000 annually to operate and maintain.

Since about 2010, the effluent from the BTPS was found to violate the cBOD discharge permit limitation during the summer, as shown in **Figure 3-9**, due to the decaying vegetative layer on the bottom of the ponds.

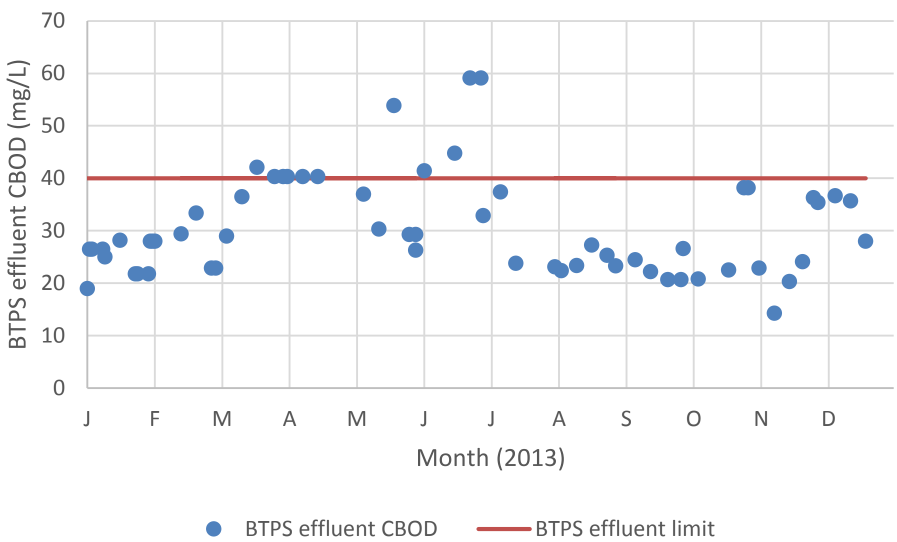


Figure 3-9 – BTPS Effluent cBOD in 2013

In effect, the BTPS offers no value in terms of actual treatment capacity, due to its inability to treat any effluent without undue public health risk due to vectors and public nuisance due to odors. The BTPS is a significant liability in terms of its annual cost to the District.

3.3.4 Disinfection

3.3.4.1 Chlorine Contact Tank 3

Disinfection at the WRF is achieved using sodium hypochlorite and chlorine contact basins. The Chlorine Contact Tank No. 3 (CCT3) provides disinfection to the liquid stream coming from the Secondary Treatment, as shown on **Figure 3-1**, and is located north of the secondary clarifiers. CCT3 is made out of two independent basins that can be operated in parallel as shown on **Figure 3-10.**

[13] Whereas the primary clarifiers, aeration tanks and secondary clarifiers in the main treatment facility have exposed water surfaces, the water surface is sufficiently disturbed to prevent mosquito breeding. However, the BTPS has large areas of quiescent water that can allow mosquito breeding if not carefully managed.

Assuming a 30-minute modal contact time and a baffling factor of 0.8, CCT3 has a treatment capacity of 16.1 mgd of average flow. At peak flow, it is acceptable to reduce the modal contact time to 15 minutes due to pathogen dilution since no minimum contact time is required by the current NPDES permit. Using a baffling factor of 0.8 as well, CCT3 has a treatment capacity of 32.2 mgd at peak flow. Using a peaking factor of 2.3, this is equivalent to 14 mgd of capacity at average flow.

Figure 3-10 Chlorine Contact Tank 3

3.3.4.2 CCT2

The CCT2 is located on the north east part of the WRF and provides disinfection to the oxidation ponds' effluent, as shown on **Figure 3-11**. Assuming a 30-minute modal contact time and a baffling factor of 0.80, CCT2 has a treatment capacity of 6.2 mgd of average flow. Using 15 minutes of modal contact time for peak flows, CCT 2 has a peak flow capacity of 12.4 mgd.

Disinfection calculations are presented in detail in **Appendix B Disinfection**, and CCT2 is depicted on **Figure 3-11**

Figure 3-11 Chlorine Contact Tank 2

3.4 SOLIDS HANDLING SYSTEMS

The WRF solid handling system processes primary sludge (PS) from the primary clarifiers, and waste activated sludge (WAS) from the ASP. The PS and the WAS are currently pumped respectively to the digester and the Pond 2 for stabilization and solids reduction. The digested sludge (DS) from the digester is then pumped to the belt filter presses, and to the sludge drying bed. Dredges are used to move the stabilized WAS from the oxidation Pond 2 to the belt presses. The sludge is dried for up to 16 to 18 months before being hauled for ultimate disposal. A process flow diagram depicts the solid handling system on **Figure 3-12** .

Primary solids averaged 8,806 dry lb/day at an average plant flow of 5.61 mgd (1,570 lb/MG). The average total solids in primary sludge in 2014 was 5.0%, somewhat better than average for primary sludge. Primary sludge averaged 3,770 gal/MG. Primary solids averaged nearly 87% volatile, indicating a high potential for solids reduction via anaerobic digestion.

WAS solids were not analyzed in monthly reports. WAS was assumed to be generated at the rate of 1.0 lb TS/lb of BOD removed and to be 80% volatile. Assuming 40% volatile reduction in the ponds gives 1,400 lb/MG of stabilized (reduced from raw) WAS generated at 90%-ile loading. This high volatile reduction assumption is further discussed in 3.4.1.2 Pond 2 and North/South Cell.

The percent solids of stabilized WAS as it is dredged from Pond 2 is estimated by VSD staff to be 2.5%.

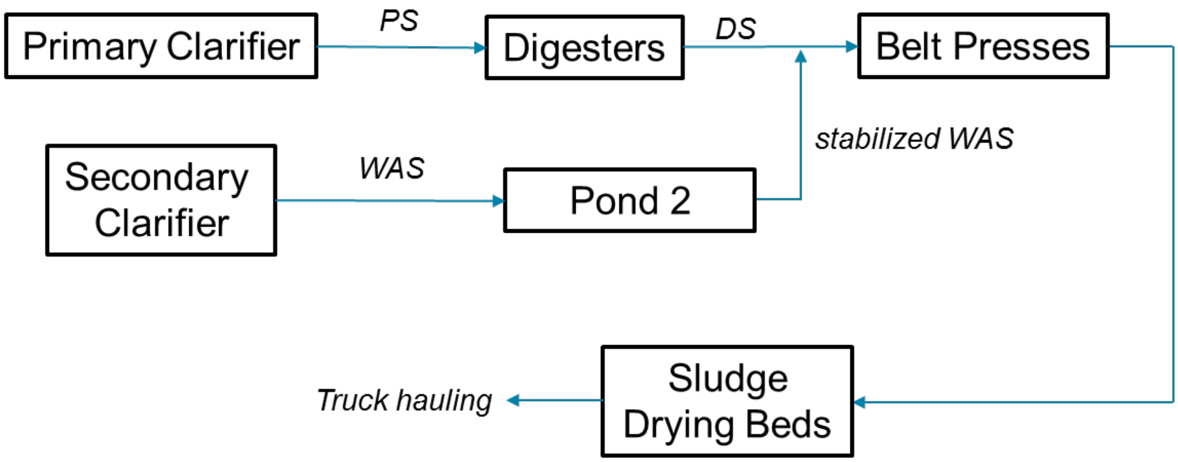


Figure 3-12 Solids Handling Process Flow Diagram

3.4.1 Solids Stabilization

3.4.1.1 Digester

The single anaerobic digester is located on the north-western side of the WRF, and is currently fed with PS only. The digester is 85 feet in diameter and has a side water depth of 27 feet. It is shown on **Figure 3-13**. Based on six months of operating data in 2014, the digester achieves 73% volatile solids reduction. For projecting future digester performance, 60% volatile solids reduction was assumed, since the digester will be more heavily loaded in the future, with consequent reduced solids retention time.

Volatile solids loading is an important parameter for digester capacity determination. For primary solids only the VS loading to the digester is estimated to be 85% of 1,570 lb VS/MG.

Assuming a typical Volatile Suspended Solid (VSS) loading of 150 lb/1,000 ft^3/day,[14] and a typical Solid Residence Time (SRT) of 20 days,[15] the digester capacity is limited to the equivalent of 15.2 mgd of average flow assuming that primary sludge only is digested. SRT is the limiting factor in capacity calculation. Calculations are presented in detail in **Appendix B Digester.**

[14] Tchobanoglous, Table 14-27, pg. 1513. Table provides a range of values from 100 to 300 lb VSS/1,000 cf/day.
[15] The 503 Biosolids regulations require a minimum of 15 days SRT for anaerobic digesters to achieve Class B Pathogen Reduction at 35 degrees C. 20 days was selected to provide a greater margin.

Figure 3-13 Digester

Digested primary sludge averages 3,770 gal/MG and 770 lb/MG.

Although the nominal capacity of the digester for primary solids was calculated as noted, the single digester has no backup when digester maintenance is needed. Although primary sludge could be discharged in the short term to Pond 2, this would be likely to cause odors that could be objectionable and would require re-seeding the digester at substantial cost. The alternative disposal of PS to Pond 2 when the digester requires maintenance that will take longer than a few days is not a long-term solution. A minimum of two digesters is recommended so that a backup unit is available when required.

3.4.1.2 Pond 2 and North/South Cell

Pond 2 is located south of the secondary clarifiers, and the North and South cells are located south of the primary sedimentation basins and west of Pond 2. Pond 2 has surface aerators and a dredging system.

Pond 2 (at present) is used as an aerobic digester for WAS. Assuming a 40-day solid residence time, the ponds are able to digest the WAS generated by 19.2 mgd of flow through the ASP given that stabilized WAS is consistently dredged out of the ponds and sent to the belt presses. However, the North and South cells are not currently used for sludge digestion due to odor issues.

An assessment was made of the effectiveness of stabilization that is achieved in the ponds where WAS is stored and stabilized. The specific oxygen uptake rate (SOUR) of sludge samples was used for this assessment.

The 503 regulations[16] consider that aerobically stabilized biosolids can meet vector attraction reduction limits if the SOUR is at or less than 1.5 mg O_2/gram/hour when tested at 20 degrees C.[17] The VSD laboratory performed SOUR testing on two random samples of WAS from Pond 2 in February 2015. The results indicated SOUR values of 0.19 and 0.55 mg O_2/gram/hour. Based on these limited test results, it appears that the stabilization of WAS in the ponds is effective.

Stabilized WAS is generated at the rate of approximately 1,400 lb/MG. As noted above, the concentration of stabilized WAS going to dewatering is estimated to be in the 2.5% range.

3.4.2 Solids Dewatering and Drying

3.4.2.1 Belt Presses

Dewatering of stabilized biosolids removes the majority of the water and prepares the biosolids for on-site drying and hauling. Belt presses employ belt wash water, which must be recycled back to the plant along with water removed. Polymer is fed to the belt press feed to enhance solids capture and improve cake percent solids.

Two belt presses (each 2-meter) are located in the belt press building between the secondary clarifiers and the digester. One belt press is shown on **Figure 3-14**. Normally one belt press is employed for digested sludge from the digester and the other is employed for dewatering of dredged stabilized WAS from Pond 2. The belt presses operate nine days per two week period, approximately 9 hours per day. This limitation was used to set belt press capacity, with no back up press. The logic is that when one belt press is out of service, the remaining belt press can be operated two shifts per day or WAS can be allowed to accumulate in Pond 2 for a period of several days or weeks without dredging.

Belt press capacity can be hydraulically or solids limited. The hydraulic limit is 120 gpm/meter and the solids limit is 750 lb dry solids per hour per meter of belt press. The cutoff is around 1.3% feed solids – above that solids concentration, the belt press is solids limited. The VSD digested sludge averages 1.4%; WAS solids is assumed to be 2.5%. On that basis, the belt presses are solids limited.

[16] Federal Code of Regulations, Title 40 (Protection of the Environment), Chapter I (Environmental Protection Agency), Subchapter O (Sewage Sludge), Part 503 (Standards for the Use or Disposal of Sewage Sludge).
[17] U.S. Environmental Protection Agency, *A Plain English Guide to the EPA Part 503 Biosolids Rule*, Chapter 5 – Pathogen and Vector Attraction Reduction Requirements, Option 4, September 1994, pg. 123.

Figure 3-14 Belt Press

The following parameters were employed to determine belt press capacity.

• Digester VS Reduction	60% (primary sludge only)
• Digested Solids	770 lb/MG (based on plant records for primary sludge)
• Pond VS Reduction	40% (WAS only, assumed)
• Stabilized WAS Solids	1,400 lb/MG
• Total Solids to Dewatering	2,170 lb/MG
• Belt Press Capacity	750 lb/hr/meter
• No. Presses	2
• Press Size	2 meter
• Belt Press Run Time	45 hours per week

The result is a belt press capacity of 8.9 mgd average daily flow.

3.4.2.2 Sludge Drying Bed

The sludge drying bed is located east of the secondary clarifiers. Paved with asphalt concrete (AC), the total area is 74,000 sf or 1.7 acres. It is shown on **Figure 3-15**. It was estimated that with 3.47 inches per year of precipitation, an average pan evaporation rate of 105 inches/year,[18] and a dry sludge production of 2,021 lb/MG, 0.29 acres per mgd are necessary for drying the solids. The drying bed capacity is then 5.9 mgd, which is very close to the 6 mgd of ADF currently observed at the WRF.

[18] Web Site: Pan evaporation, Indio Date Garden, Mean Monthly, Seasonal and Annual Pan Evaporation for the United States, 1982.

Figure 3-15 Sludge Drying Bed

The calculated sludge drying bed capacity matches WRF staff observations that the bottleneck in the solids handling system is the drying beds at this time. Adding drying bed area should be among the first upgrades to be designed and built.

3.4.2.3 Truck Hauling

Solids are kept on-site for 16 to 18 months, and then hauled to Yuma, Arizona for landfill disposal. The contracted hauler is Solids Solutions for the past eight years.

There is no firm limitation to the volume of solids that can be transported by truck. According to WRF staff, between 1,200 tons and 1,400 tons every 16 to 18 months are hauled away.

3.5 SUMMARY

The WRF's treatment capacities, assuming current plant operations, are summarized in **Table 3-6,** and depicted in **Figure 3-16.**

Design criteria used for the process capacity evaluation are summarized in
Table 3-7.

Table 3-6 – Summary of existing processes treatment capacities

Process Unit	Existing Capacity (mgd)
Influent Pumping Station	14
Bar Screens	11.7 / 18.7
Grit Removal	5.9
Primary Clarifiers	8.2
Activated Sludge	12.5
Aeration Blowers	10
Secondary Clarifiers	11.7
Disinfection	12.4 / 14
Digesters	15.2
Belt Filter Presses (BFPs)	8.9
Sludge Drying Bed	5.9

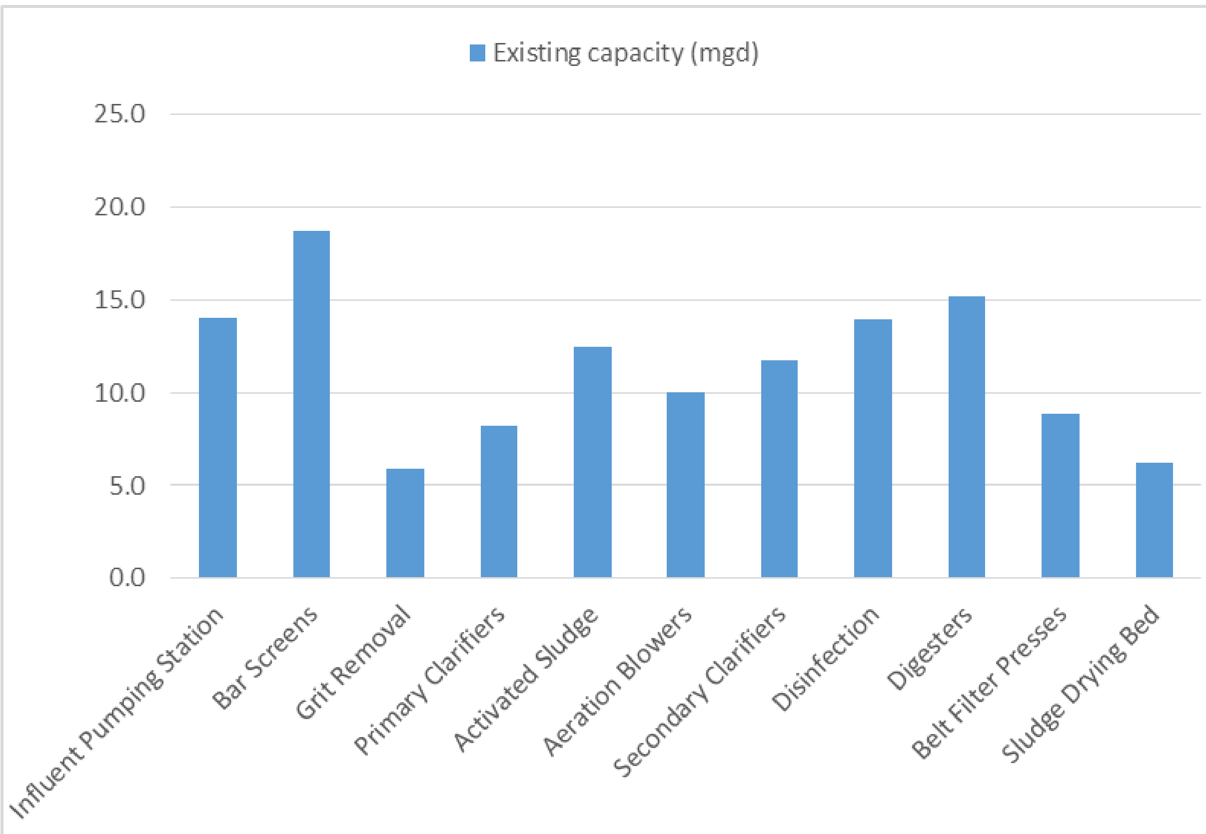


Figure 3-16 Existing Treatment Unit Process Capacities

Table 3-7 Design Criteria Summary

Process	Criteria	Value
Primary Clarifiers	Hydraulic loading at average flow (gal/day/sf)	1,200
	Hydraulic loading at peak flow (gal/day/sf)	3,000
Activated Sludge	Max MLSS - no N-removal (mg/L)	2,500
	Max MLSS - N-removal (mg/L)	3,000
Secondary Clarifiers	Hydraulic loading at average flow (gal/day/sf)	550
	Hydraulic loading at peak flow (gal/day/sf)	1,300
	Solids loading at average flow (lb/sf/hr)	1
	Solids loading at peak flow (lb/sf/hr)	1.6
Disinfection	Baffling factor	0.8
	Minimum contact time at average flow (min)	30
	Minimum contact time at peak flow (min)	15
	Title 22 - Modal contact time (min)	90
Gravity Belt Thickeners	Hydraulic loading (gal/min/m)	175
Belt Filter Presses (BFPs)	Press capacity (lb/hr)	1,500
Sludge Drying Bed	Average annual precipitation (in/yr)	3.47
	Content solid required after evaporation	0.9

Section 4 – Liquid Process Option 1: Secondary Treatment without Denitrification

Section 4
Liquid Process Option 1: Secondary without Denitrification

4.1 APPROACH

Since the process design is built around the effluent quality requirements, a separate section was devoted to each of the three qualities of effluent. However, as it is likely that effluent requirements will change, the overall approach assumes different effluent requirements for different future phases.

This section, devoted to Option 1, is based upon current effluent limitations. That would require the current process unit sizes and capacities to be upgraded to treat build-out flows and loadings, but would not require a change to the type of processes currently used. These processes include bar screens, grit removal, primary clarification, aeration basins with selectors designed for BOD removal only and no nutrient removal (nitrogen and phosphate), secondary clarifiers, and chlorine disinfection. Enhanced primary sedimentation would be improved by addition of polymer dosage in addition to the existing ferric dosing at the current aerated grit removal chamber. A process flow diagram describes the process in **Figure 4-1.**

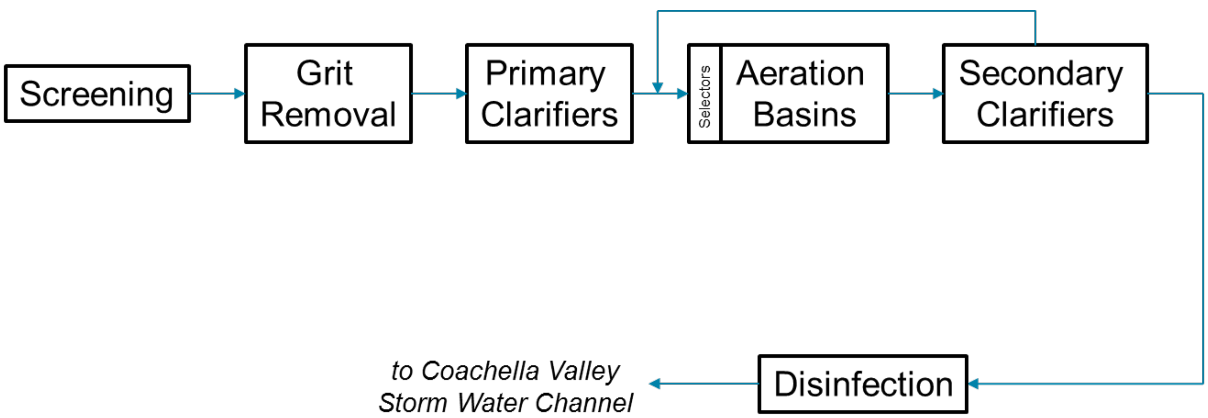


Figure 4-1 Option 1 Process Flow Diagram

4.2 REGULATORY CONSIDERATIONS

Effluent limitations for Option 1 are to remain as described in Section 3. Existing limits are for cBOD and TSS to 25, 30 mg/L respectively for monthly average, and 40, 45 mg/L respectively for weekly average.

4.3 DESIGN PARAMETERS

Flow, loadings, and influent concentration at build-out are discussed in Section 2 and summarized in Table 2-7, Table 2-8, and Table 2-9. A peaking factor of 2.2 between the build-out ADF of 20 mgd and the PDWF was used at build-out per Error! Reference source not found. analysis. More process-specific design assumptions are listed below and in **Appendix C.**

4.4 PROCESS REQUIREMENTS

Calculations for each process described below are presented in **Appendix B**.

4.4.1 Preliminary Treatment

4.4.1.1 Influent Pump Station

The existing Influent Pump Station structure has space for six submersible wastewater pumps, one in each bay. Five pumps are currently installed with a capacity of 5,600 gpm each. At build-out, one larger 6,800 gpm pump should be added to the currently vacant sixth pump bay, and two existing 5,600 gpm replaced by two larger 6,800 gpm pumps. That would allow to pump 44.5 mgd in worst-case conditions where one large 6,800 gpm pump is off-line.

The Influent Pump Station structure is sufficient for the build-out peak wet weather flow. Periodic inspections of the structure interior should be conducted to assure structural integrity for long-term service.

4.4.1.2 Bar Screens

The existing Screening Facility has two bar screens each with hydraulic capacity to treat a peak flow of 24 mgd. Higher flow can be accommodated with somewhat degraded performance. However, the screens were installed in 1999 and typically have a 30-year useful life; existing screens will have to be replaced before 2029.

The WRF staff expressed a strong preference for finer screens than what is currently installed. A discussion of screening options follows.

Screens for wastewater fall into two categories:

- Coarse screens with ¼-inch and larger openings (Chain Drive, Reciprocating Rake, Catenary and Continuous Belt Screens)
- Fine screens with less than ¼-inch openings (Band, Static Wedgewire, Drum and Stair or Step Screens)

Fine screens are generally preceded by coarse screens. For the VSD WRF, fine screens are not a good option, since there is not sufficient head for that type of device. However, a ¼-inch bar screen can work with the hydraulic constraints at build-out.

Some specifics about the setup at the Screening Facility (assuming that elevation of -500 ft is used as elevation zero datum):

• Number of Channels	3
• Channel Width	4.0 ft
• Channel Bottom Elevation	472.75 ft
• Channel Top of Grating	480.01 ft
• Downstream Water Surface at Peak Flow	478.66 ft
• Maximum WSE Upstream of Screen	479.01 ft
• Maximum Available Loss Thru Screens	0.35 ft (4 inches)
• Assumed Screen Framing (unavailable for flow)	3-inch bottom; 5-inch each side
• Number of Duty Screens	3 (all units in service)
• Desired Flow per Screen	15 mgd (34.8 cfs)

A ¼-inch bar screen with ¼-inch bar thickness will require approximately 4 inches of head loss with 30% screen blinding.[19] A multi-rake screen such as the Vulcan VMR (see **Appendix G** for catalog cut) would fit in the existing channels and provide sufficient screening capacity at ¼-inch bar spacing but exceeding the acceptable head loss.

If one screen were out of service during the design peak flow event, the loss is approximately 7.5 inches and the desired freeboard is not maintained but no overflow occurs. In addition, a 6'' curb surrounds the facility, allowing containment of potential overflow in case of high flow events. In the case of such an event, the spill would be contained to the area directly surrounding the structure and delimited by the 6" curb, and no reporting would be required.

Hence three new finer bar-screens with ¼" spacing are proposed for build-out to replace the two existing ½-inch climber screens and the manual rack. This screening facility can then accommodate the peak build-out flow of 44 mgd.

4.4.1.3 Grit Removal

Vortex grit removal systems do not require as much energy or head and do not tend to strip out VOCs the way that aerated grit chambers will. Low head loss is required to fit the constraints of the existing upstream and downstream units.

Two vortex grit removal tanks each at 22-ft diameter with a capacity of 22.5 mgd each would be installed at build-out to replace the existing aerated grit removal tank. That would provide 45 mgd of flow capacity. **Appendix G** contains vendor information on a typical vortex grit chamber.

Each vortex grit chamber would have two (1 + 1) grit pumps (recessed impeller type) and one grit cyclones / classifier each, each pump with each own independent piping to facilitate maintenance. Calculations are presented in detail in **Appendix B Grit Removal**.

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[19] Whereas 40% blinding was selected for evaluation of the existing climber screens, 30% blinding is suitable for multi-rake screens that provide more frequent cleaning.

4.4.2 Primary Clarifiers

The type of primary clarifier now in place at the WRF works well and will be replicated in future expansions. Ultimate expansion of the primary clarifiers will require demolition of the existing selectors and aeration tanks, and construction of new and deeper aeration tanks / selectors.

Further, to complete the implementation of chemically enhanced primary treatment, the existing ferric chloride feed will be supplemented with polymer addition upstream of the primary tanks.

Four additional rectangular basins, with the same dimension and design as currently installed on-site (170 ft x 20 ft x 12 ft, L x W x SWD) will be required in addition to the two existing tanks to treat average and maximum build-out flows. The size of required infrastructure was determined using:

- Average and peak loading rate of 1,000 gal/day/sf and 2,500 gal/day/sf respectively
- Retention time of 2 hours at average flow

Detailed calculations are presented in **Appendix B Primary Clarifiers**.

4.4.3 Secondary Treatment

4.4.3.1 Activated Sludge Process

For this option in which nitrogen removal is not necessary, assuming that the volume of clarifiers would double (3 additional clarifiers installed, same size as existing), and that the activated sludge process is operated with a 4 days SRT at 2,500 mg/L of MLSS, the total ASP basin volume would increases by a factor of 1.25 for the build-out ADF of 20 mgd. The operational parameters for this design are summarized as follows:

- SRT 4 days
- MLSS 2,500 mg/L
- Net yield 1 lb of VSS/lb of BOD removed

4.4.3.2 Aeration System

The aeration system associated with the activated sludge process described above would be designed to meet the total oxygen design demand associated with the biological treatment. For the option wherein nitrification is not required, the aeration capacity required is 910 cfm/mgd, with no accounting for blowers out of service.

4.4.3.3 RAS Pumps

For the build-out ADF of 20 mgd, the total required RAS pumping capacity is 30 mgd for a maximum RAS recycle ratio of 1.5. For the typical RAS recycle ratio of 0.6, the RAS pumps would be capable of handling a plant flow of 50 mgd. Assuming a total of 6 clarifiers (see below section **Secondary Clarifiers**), a total of 9 RAS pumps are recommended, with an operation configuration of 2 + 1 pumps for every two clarifiers. This requires each pump to have a capacity of 3,500 gpm and a 40 hp motor. Calculations are presented in detail in **Appendix B Return Activated Sludge Pumps**.

4.4.3.4 Secondary Clarifiers

Assuming an average flow of 20 mgd, a peak flow of 44 mgd, using typical hydraulic loadings of 550 gal/day/ft^2 at average flow and 1300 gal/day/ft^2 at peak flow, solids loading of 1 lb/sf/hr at average flow and 1.6 lb/sf/hr at peak flow, and 2,500 mg/L of MLSS, 6 secondary clarifiers would be required. In that low MLSS case, the clarifiers would be hydraulically limited. Calculations are presented in **Appendix B Secondary Clarifiers**. This is also consistent with the clarifiers volume addition required for ASP process.

4.4.4 Disinfection

The existing chlorine contact tank has a 14.2 mgd average flow capacity, which will not be sufficient for anticipated build-out flows. It is recommended to retrofit the existing basin and install UV systems inside the existing structure.

UV disinfection presents multiple advantages:

- Very compact footprint and reuse existing infrastructure (low capital cost)
- Well adapted to moderate coliform discharge limits seen currently[20]
- Does not require chlorine addition for disinfection (chemical savings)
- Does not require sodium bisulfite for chlorine quenching (chemical savings), and eliminate risk to violate chlorine discharge permit (eliminates fines)

It should be noted that UV disinfection has low capital but high power consumption. However, the Plant will have solar panels installed by then that should reduce the Plant's power costs.

Disinfection for potential Title 22 reuse would be a separated system and is presented in **Section 6**.

4.5 ANCILLARY SYSTEMS AND FACILITIES

4.5.1 Biofilters

Compost-bed biofilters will be used for odor control, for off-gas from the three new vortex grit chambers and the TWAS building. Twelve air changes per hour will be required to ventilate these buildings, and a conservative hydrogen sulfide concentration of 60 mg/m^3 in this off-gas was assumed. Biofilters were sized for a 90 m/hr gas loading rate and a minimum of 45 sec empty bed gas residence time. The biofilters were assumed to have a maximum sulfide elimination capacity of 130 g/m^3-hr based on typical data, and they were assumed to require 3 kg of lime buffer in the bed per kg of sulfide removed. Calculations are presented in detail in **Appendix B.11**. However, lime is often not required, and lime addition would be evaluated based on biofilter performances once in operation.

---

[20] For Fecal Coliforms the geometric mean shall not exceed a Most Probable Number (MPB) of 126 per 100 millimeters, and none of the sample taken shall exceed a density of 400 MPN/100mL

For the grit chambers, the ventilation system will include 22" diameter ductwork and an 8.5-hp blower delivering an off-gas flow of 8,700 cfm. This will be treated by a 43 ft x 43 ft square biofilter with a bed depth of 4 ft. This biofilter would provide adequate gas residence time and sulfide elimination capacity. Assuming a useful life of 2 years for the compost bed gives a total lime buffer requirement of 46,700 kg.

The TWAS building will be ventilated at a rate of 14,700 cfm by a 15 hp blower and 28" diameter ductwork. To provide sufficient gas contact time and sulfide elimination, this off-gas will be treated by a 55 ft x 55 ft biofilter with a 4 ft deep compost bed. The lime buffer requirement would be 78,800 kg for a two-year bed life.

It should be noted that the approach presented above is conservative and assume that biofilters already installed and used at the plant would be abandoned (their footprint would be available for other construction).

4.5.2 Electrical

It is anticipated that at build-out, electrical demand will increase significantly given additional aeration capacity that will have to be installed.

4.5.3 Chemical

Chemical feeding systems will have to be upgraded as well. Since the chlorine contact will be retrofitted to UV, only ferric dosing system and storage will have to be upgraded to accommodate the higher flows.

Currently, ferric chloride is added at the aerated grit chambers. This provides mixing for the chemical addition. Since the aerated grit chamber will no longer be employed, future addition of polymer to provide chemically enhanced primary treatment will also require mixing. Relocation of the existing ferric feeding system ahead of the new vortex chambers will be required. Polymer addition typically requires mixing as well. Injection in the influent channel right on top of the influent pipe penetration would be a good location for required mixing. Diffused air or other means could be used to enhance polymer mixing in the existing 7' x 6' riser box.

4.5.4 Operation and Maintenance

Both an Operation and a Maintenance building extension will be built. The maintenance building extension would be approximately 70 feet by 70 feet based on available room in the proximity of the existing Maintenance building, which is the VSD preferred location for the extension. The operation building would be approximately 150 feet by 70 feet based on sizes seen in the industry and room available in proposed future location. Staffing requirements presented in Section 8 for Phase 2b (Option 1 assumed to be operative) are based on discharge of all effluent to the CVSC (no reclamation). Reclamation will increase staffing required by two.

Section 5 - Liquid Process Option 2: Secondary Treatment with Nitrification

Section 5 Option 2: Secondary with Nitrification

Option 2 assumes that the current effluent limitations would be modified to limit nutrient discharge, especially nitrogen. That would impact the activated sludge process, and would require the installation of mixed anoxic zones and internal mixed liquor recycling. Other processes in the plant would not be affected and would be identical to Option 1. A process flow diagram for Option 2 is provided in **Figure 5-1**.

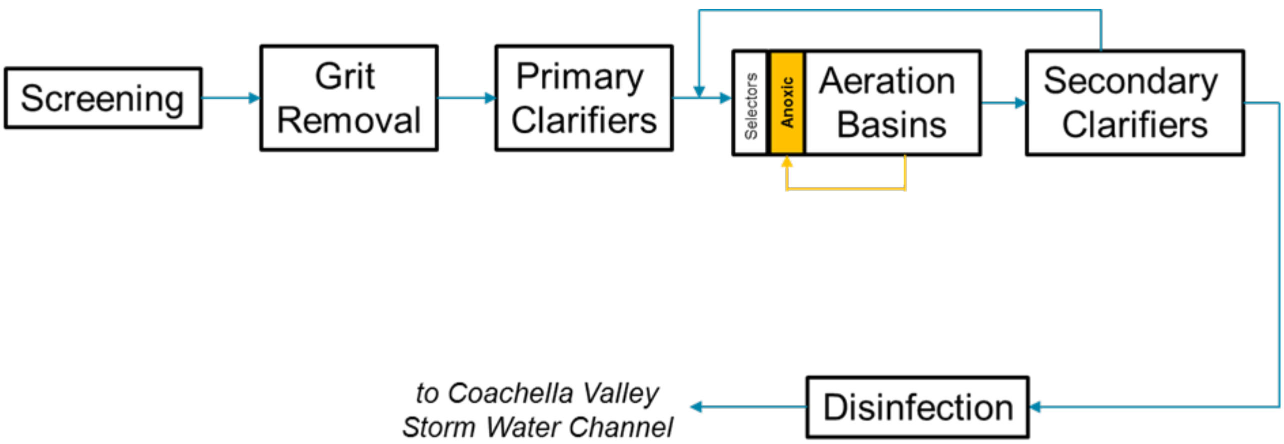


Figure 5-1 Option 2 Process Flow Diagram

5.1 REGULATORY CONSIDERATIONS

This Option was considered in order to analyze the changes required to the plant's processes if the nitrogen discharge limit of 10 mg/L of total nitrogen were established. This would be a major change, but would also be likely to happen in the upcoming decades, especially if the current drought context persists and reuse or groundwater recharge could degrade a potential water supply. For this master plan, as developed further in **Section 8,** this change was anticipated to happen after 2050. Those assumptions are made for phasing presented in Section 8.

5.2 DESIGN PARAMETERS

Flow, loadings, and influent concentration at build-out are discussed in Section 2 and summarized in Table 2-7, Table 2-8, and Table 2-9. More process-specific design assumptions are listed below and developed in **Appendix C**.

5.3 PROCESS REQUIREMENTS AT BUILD-OUT

5.3.1 Nitrogen Removal

5.3.1.1 Activated Sludge Process (ASP)

An ASP designed for nitrogen removal involves a larger aeration basin size, an anoxic zone with mixing, and internal mixed liquor recycle pumps. The influent ammonia concentration is estimated as 80% of the 90th percentile TKN concentration in the current plant influent. The operational parameters for this design are as follows:

- SRT 10 days
- MLSS 3,000 mg/L
- Net yield 1 lb of VSS/lb of BOD removed

Given these parameters, and a build-out ADF of 20 mgd, 6 basins with a total dimension of 281 ft x 30 ft x 20 ft (L x W x SWD) each would provide the required level of treatment. This would also require mixers in the anoxic zones, and 9 total internal recycle pumps (6 + 3 configuration) for all basins, with 11,000 gpm and 40 hp for each pump. Calculations are presented in detail in **Appendix B Activated Sludge Process**.

5.3.1.2 Aeration System

The aeration system serving the ASP for nitrogen removal would also have to be larger compared to Option 1 because of nitrification, which is an aerobic process and hence consumes oxygen. A total of six blowers (one per basin) will still be required, but the blower size will increase to 6,800 cfm with a 350 hp motor to deliver sufficient airflow and provide adequate mixing energy. Calculations are presented in detail in **Appendix B Aeration System**.

5.4 ANCILLARY EQUIPMENT

The ancillary equipment required would be similar to what presented in Section 4. Even though the blowers would need to be larger to provide additional oxygen to nitrifying bacteria, this is not anticipated to have a significant impact on ancillary equipment.

Section 6 - Liquid Process Option 3: Title 22

Section 6
Liquid Process Options 3: Title 22

6.1 OPTION 3: TITLE 22

Option 3 assumes that the demand for recycled water in the Coachella Valley will justify full water recycling at the WRF at build-out.[21] According to the previously conducted Recycled Water Master Plan (Carollo, 2011), the maximum day demand is 28.5 mgd, and is above the build-out flow at the WRF (MWH, 2013) of 20 mgd, which justifies the demand assumption above. This section first presents different reuse strategies along with regulatory considerations, and then details process recommendations, requirements, and sizing for Title 22. A process flow diagram of Option 3 is shown on **Figure 6-1.**

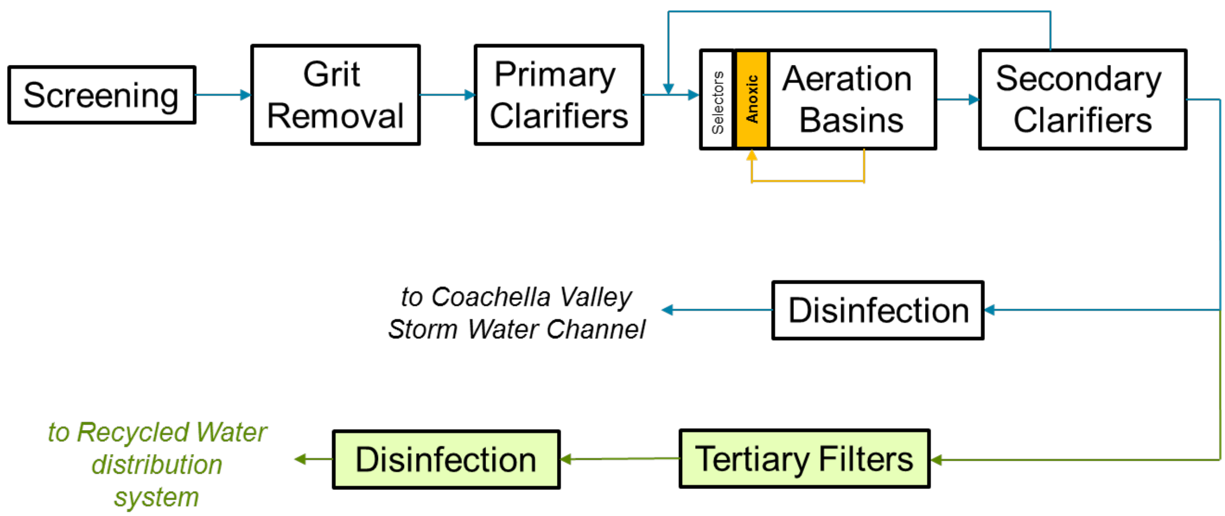


Figure 6-1 Option 3 Process Flow Diagram

6.2 REGULATORY CONSIDERATIONS

As stated by Title 22 regulation (CDPH, 2014), in order to reuse recycled water for food crops, parks and playground, school yards, residential landscaping, and unrestricted golf courses, the water shall be a disinfected tertiary recycled water:

- **Disinfected tertiary recycled water** means a filtered and then disinfected wastewater that meets the following criteria:
    1. The filtered wastewater has been disinfected by either a chlorine disinfection process that provides a CT (product of total chlorine residual and modal contact time measured at the

[21] Note that some portion of the flow must be maintained to the CVSC and is not available for reclamation due to habitat protection. Estimates range on the order of 1 mgd.

same point) values of no less than 450 mg.min/L at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow; or
  2. A disinfection process that has been demonstrated to inactivate and/or remove 99.999 percent of the plaque forming units of F-specific bacteriophage MS2, or polio virus in wastewater.
- The median concentration of total coliform bacteria measured in the disinfected effluent does not exceed an MPN of 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed and the number of total coliform bacteria does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30 days period. No sample shall exceed an MPN of 240 total coliform bacteria per 100 milliliters.

Filtered wastewater means an oxidized wastewater that either:
- Has been coagulated and passed through natural undisturbed soils or a bed of filter media pursuant to the following:
  1. At a rate that does not exceed 5 gallons per minute per square foot of surface area in mono, dual or mixed media gravity, upflow or pressure filtration systems, or does not exceed 2 gallons per minute per square foot of surface are in travelling bridge automatic backwash filet
  2. So that the turbidity of the filtered wastewater does not exceed any of the following:
     - An average of 2 NTU within a 24-hour period
     - 5 NTU more than 5 percent of the time within a 24-hour period; and
     - 10 NTU at any time
- Has been passed through a microfiltration, ultrafiltration, nanofiltration, or reverse osmosis membrane so that the turbidity of the filtered wastewater does not exceed any of the following:
  1. 0.2 NTU more than 5 percent of the time within a 24-hour period; and
  2. 0.5 NTU at any time

The regulation allows a relatively wide variety of options to produce Title 22 water. Since the water will be sent to a “purple pipe” type distribution system instead of being percolated to the aquifer (Carollo, 2011), the secondary effluent will have to go through tertiary filtration and disinfection.

In order to maintain the distribution system, a residual chlorine concentration will have to be maintained in the purple pipeline. This makes the chlorine disinfection with a 450 mg.min/L CT a disinfection process of choice to serve a dual purpose: disinfection and pipeline maintenance. Other disinfection processes such as ozone, or UV, would provide a 99.999 percent coliform reduction, but the water would need to be chlorinated for a disinfectant residual in the distribution system. In that regard, it is recommended to use chlorine disinfection for recycled water.

For the filtration process, various filtration technologies have already been approved by CDPH for Title 22 production. Among them, cloth media filters offer Title 22 filtration along with small footprint, low head loss, low maintenance costs, low operation costs, and low capital costs. They consist of cloth-media disks made of 5-microns nominal pore size cloth media, and operate on a filter/backwash cycle with the advantage of filtering not being interrupted during backwash.

6.3 DESIGN PARAMETERS

According to previous studies (Carollo, 2011), during high demand months (summer), the demand will be higher than the secondary effluent wastewater flows. The wastewater flow is then controlling the distribution flow, and secondary flow is assumed to be treated to Title 22 in its entirety during high demand months. Filters and chlorine contact tanks are then sized on secondary effluent flows. This sizing is conservative, since it is anticipated that 1 mgd of effluent discharge to the CVSC will be required for habitat that depend on this water source.

Variation in demand and increased distribution flows due to short irrigation windows (typically 8 hours) will be addressed by a large storage tank at the WRF. Part of Pond 3 will be used for the recycled water storage reservoir. A distribution pump station was also sized in this option to pump from recycled water storage.

6.4 PROCESS REQUIREMENTS AT BUILD-OUT

6.4.1 Transfer Pumps and Equalization Basin

The tertiary filters will be installed downstream of the secondary clarifiers. Since water needs to be pumped to the downstream cloth-media filters, an equalization basin will be built to serve as a wet well for the transfer pumps, and to allow splitting the flow between the channel discharge and the Title 22 treatment processes. A splitter box controlled by the equalization basin water level will allow splitting the flow by gravity.

At build-out, eight (8) 2,400 gpm pumps will be installed to pump from the equalization basin to the filters, with six (6) duty pumps, and two (2) stand-by.

6.4.2 Cloth-media Filters

Three (3) cloth-media filters units will be installed at build-out, with 20 disks each for a total of 60 disks as described in **Appendix G**.

6.4.3 Disinfection

As described earlier, disinfection process shall provide a CT (the product of total chlorine residual and modal contact time measured at the same point) value of no less than 450 mg.min/L at all time with a modal contact time of at least 90 minutes, based on a peak dry weather design flow.

Using a safety factor of 1.15, two basins 220 feet long, 100 feet wide, and 12 feet SWD will have to be installed. Detailed calculations are presented in **Appendix B Disinfection**.

6.5 STORAGE AND DISTRIBUTION

One twelve (12) million gallon (MG) reservoir would be installed at the WRF for storage of 14 hours of recycled water production in order to match the high Title 22 water during a standard irrigation window (8:00 pm to 6:00 am). Given that the average day demand is 20 mgd, a 10-hour irrigation window results in a 48 mgd flow.

That would require installing a large pump station including fifteen (15) 2,800 gpm pumps with a 350 feet of total dynamic head (TDH) each. Twelve (12) pumps would be on-duty, whereas three (3) pumps would be on stand-by. Detailed calculations are presented in **Appendix B Recycled Water Pump Station.**

Section 7 – Biosolids Management

Section 7
Biosolids Management

7.1 GENERAL DESCRIPTION

Biosolids production is anticipated to be identical for the three liquid treatment options considered in **Sections 4, 5 and 6**. Solids handling process units and infrastructure presented below will remain the same at build-out.

The following subsections will present a discussion of biosolids handling starting with raw solids as they are removed from the liquid treatment processes, through thickening, stabilization, dewatering, drying, and ultimate disposal. **Appendix E** “Process Flow Diagrams” includes biosolids or sludge processes and can be referenced along with this Section.

7.2 SOLID HANDLING SYSTEMS

7.2.1 Solids Generated

The WRF will generate solids from screening, grit removal, primary clarifiers and from secondary clarifiers. The quantities of solids generated were calculated as shown below and summarized in **Table 7-1**.

Table 7-1 Solids Generated, Quantities

Parameter	Quantity	Unit	Comment
Screenings	9	cf/MG	Screenings volume will increase considerably with ¼-inch screens from 3 to 9 cf/MG
Grit	1.0	cf/MG	Common for separate sewer system (as opposed to combined sewers)
Primary Solids	1,250	lb/MG	Based on 90%-ile TSS in influent 250 mg/L, 60% TSS removal in primary clarifiers
Waste Activated Sludge (WAS)	1,960	lb/MG	Based on 90%-ile BOD. Assumes 25% BOD removal in primary tanks. Observed yield of 1.0 lb TSS/lb BOD removed.
WAS Flow	47,000	gpd/MG	Assumes 0.5% solids at 90%-ile condition.
Digested Sludge Flow	8,375	gpd/MG	Assumes PS + WAS, 60% VS reduction.

7.2.2 Secondary Sludge (WAS) Thickening

Although primary sludge can reach 4 to 6% solids under most circumstances, and therefore does not require thickening prior to digestion, WAS before thickening will only reach 0.5 to 1.0% solids, far too low for digestion. At this time, the WRF does not employ WAS thickening aside from gravity thickening in Pond 2. The WRF also has a large storage volume for WAS and can thicken or dewater WAS on a small number of hours per week when desired.

When Pond 2 is no longer available for WAS storage, the scheduling flexibility for WAS thickening will be greatly reduced. WAS must be removed from the secondary clarifiers continuously or nearly continuously. Hence, the thickening process must operate most of the day each day.

Several methods are commercially available to achieve acceptable thickening of WAS for subsequent digestion:

- Dissolved Air Flotation Thickener (DAFT)
- Gravity Belt Thickener
- Rotary Drum Thickener
- Centrifuge

A brief discussion of these four methods follows.

Dissolved Air Flotation Thickener (DAFT)

The circular DAFT is commonly employed at large and medium size wastewater treatment facilities. Rectangular DAFTs are less common in Southern California.

The process operates on the principle that under pressure air will dissolve much more gas than it will at atmospheric pressure. A saturation tank is employed with air compressor to produce a very highly saturated recycle water stream for introduction to the DAFT tank. The high pressure liquid recycle stream is allowed to depressurize inside the tank as it mixes with incoming WAS and polymer (if used). The bubbles released by the depressurized recycle stream float the solids in the WAS and create a floating mat on top of the tank that is then skimmed off. The separated liquid (underflow or subnatant) either passes under a baffle and weir or is removed via submerged perforated pipe (submerged launder) in the case of DAFTs with variable level. **Figure 7-1** shows a typical DAFT process schematic.

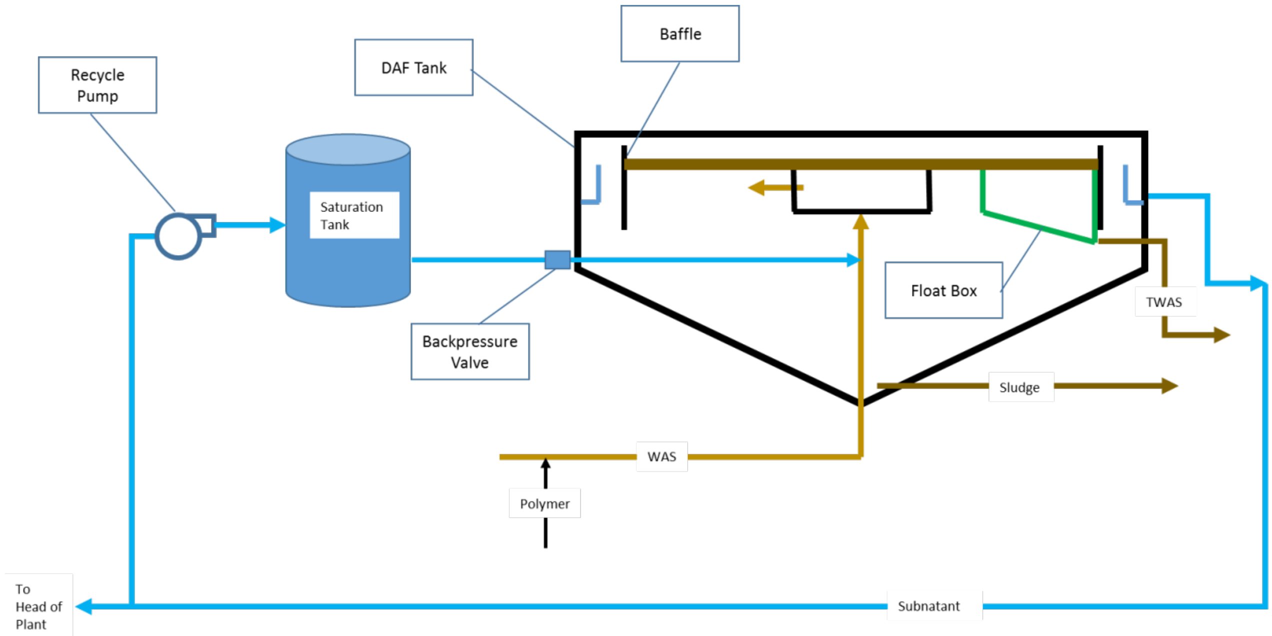


Figure 7-1 DAFT Process Schematic

A typical DAFT arrangement provides one or two circular DAFT tanks with an adjoining building to house the air compressors, recycle pumps, TWAS pumps, electrical power distribution equipment and control panels. The DAFT tanks are normally open to atmosphere and do not generate significant odor.

Common characteristics of DAFT units are given in **Table 7-2** below.

Because gasses in air are not absorbed equally, the saturation tank will be depleted of oxygen and become less efficient if the air in the headspace is not vented periodically or continuously.

Advantages of DAFTs are there is no washwater requirement and solids capture is very high. Disadvantage is high power cost for the recycle pumps and air compressors. Maintenance items include the rotating mechanism, the float scraper arm adjustment, the recycle and TWAS pumps and the air compressor. Polymer feed system is optional but will normally be employed at some point.

The DAFT can require an hour or more to start it up once it is shut down. If the facility does not have enough WAS on a continuous basis to feed a single DAFT unit, the start and stop cycles may be a daily occurrence. One alternative for the low solids production scenario is to leave out polymer for the low-flow years and operate at lower solids loading.

The offsetting advantage is that the DAFT can run continuously with very little operator attention if there is sufficient WAS or WAS storage to warrant continuous operation.

Table 7-2 Dissolved Air Flotation Thickener Parameters

Parameter	Quantity	Unit	Comment
Solids Loading	0.4 - 2.0	lb/sf/hr	Typical 0.8 w/o polymer; 2.0 with polymer.
Air-to-Solids Ratio	0.025 – 0.04	--	Determines size of air compressors required
Polymer Dose	4 to 8	lb active per ton dry solids	DAFT can also operate without polymer, but at much lower loading rate (0.8 lb/sf/hr) and producing lesser TWAS%.
Thickened Waste Activated Sludge	3 to 6%	--	Varies depending on loading, effectiveness of polymer. 3-5% w/o polymer, 4-6% with polymer. Above 6% can be difficult to pump.
Solids Capture	95 - 99%	--	
Recycle Pumps	--	--	ANSI horizontal end-suction type. Must be capable of producing 80 psig
Air Compressor	--	--	Screw compressor is typical, others are acceptable. Capable of producing 125 psig. Requires air receiver.
TWAS Transfer Pumps	--	--	Normally employ progressing cavity or rotary lobe pumps.
VSD Average Flow Capacity of One Unit	21.1	mgd	Assumes 40-ft diameter DAFT run 22 hours/day 7 days per week at 1.5 lb/sf/hr

Gravity Belt Thickener (GBT)[22]

The GBT is similar to a belt press, except that it stops at the gravity drainage stage, incorporating only five rollers as opposed to 14 rollers as on a typical dewatering belt press. **Appendix G** contains a manufacturer brochure for a typical GBT.

[22] Portions from National Biosolids Partnership / Water Environment Federation, *National Manual of Good Practice for Biosolids*, Chapter 5.

WAS is pumped onto the porous belt media which retains the solids and allows clarified filtrate to pass through. Solids are scraped off at one end of the gravity drainage section. Approximately 100 gpm of washwater per GBT unit is required to clean the belt following solids (TWAS) removal.

Each GBT would be provided with a TWAS pump. The GBTs and TWAS Pumps would be housed in a building or under a canopy.

Variables for GBT operation include polymer dose, WAS feed rate, mixing of WAS and polymer and belt speed. The TWAS ramp angle must be sufficient to prevent solids buildup.

Startup of a GBT is a 10 minute process. Shutdown can take 10 to 20 minutes.

GBT typical design parameters are listed in **Table 7-3** below.

The GBT is a relatively simple process and is simple to start and stop. Energy consumption is low. Disadvantage is the continuous use of washwater while in service. The washwater has to be pumped to the GBT and drained back to the Plant Drain Pump Station, ultimately to the Influent Pump Station.

The scaling of the GBT is also a good fit for VSD with one unit capable of handling 9.8 mgd (more with slightly higher loading **–** see **Table 7-3** for scheduling assumptions).

Table 7-3 Gravity Belt Thickener Parameters

Parameter	Quantity	Unit	Comment
Liquid Loading	150 – 250	gpm/meter	Normal feed is 0.4 to 0.8% solids WAS. 1.0, 1.5, 2.0 and 3.0- meter GBTs are common.
Polymer Dose	3 to 10	lb active per ton dry solids	Polymer is not optional.
Thickened Waste Activated Sludge	4 to 8%	--	Varies depending on loading, effectiveness of polymer. Above 6% can be difficult to pump.
Solids Capture	90 - 98%	--	Typical is 95%
TWAS Transfer Pumps	--	--	Normally employ progressing cavity or rotary lobe pumps
VSD Average Flow Capacity of One Unit	9.8	mgd	Assumes 2-meter GBT at 175 gpm/meter. Run 22 hr/day 7 days/week

Rotary Drum Thickener (RDT)

Like the GBT, a RDT uses a porous medium, in this case mounted on a rotating drum. However, its success is best when the solids to be thickened have a fibrous content, such as would be true of primary sludge, but less so with WAS. Success with municipal WAS is variable and dependent on solids characteristics. Polymer requirements are a concern because of floc sensitivity.

Due to its variable success with municipal WAS, the RDT was not considered for VSD.

Centrifuge Thickener

Centrifugal thickeners use centrifugal force to separate solids from liquid, much as they would do for dewatering applications. The solid bowl conveyor type of centrifuge has been successfully used for solids thickening.

Centrifuge thickeners require the smallest footprint of all the four thickening methods and requires the most power and polymer. Maintenance is more complex than any of the other methods. Capital cost for the thickening centrifuge is approximately double the cost of a GBT with the same liquid handling capacity.

WAS Thickening Conclusion

For the purposes of the Master Plan, the Gravity Belt Thickener was selected due to its relatively simple operation, low capital cost and low power cost. Although the GBT requires washwater, the 2-meter unit scales better for the flow range that the WRP will treat in the planning period.

One GBT can accommodate flows up to 9.8 mgd. Two units should be provided initially and a third added when flows exceed 10 mgd. Three units will operate as 2 + 1 for the build-out condition. Calculations are presented in **Appendix B Gravity Belt Thickeners.**

7.2.3 Solids Stabilization

7.2.3.1 Digesters

Future digester capacity will be based on the same type of digester as the existing one: anaerobic mesophilic digestion, at approximately 95 degrees F. At build-out, it is assumed that Thickened Waste Activated Sludge (TWAS) is sent to the digester along with the Primary Sludge (PS). Further assumptions used for digester sizing:

Solid Residence Time (SRT) = Hydraulic Residence Time	20 days
Primary Clarifier TSS Removal Efficiency	60%
Primary Clarifier BOD Removal Efficiency	25%
WAS Observed Yield in ASP	1.0 lb-WAS/lb-BOD removed
VSS in Primary Sludge	85%
VSS in WAS	80%
VSS Destruction in Digesters	60%

Calculations are presented in **Appendix B Digester**

Although the existing digester achieves a remarkably high 73% VS reduction, the VS reduction for the future digesters will be based on 60% VS reduction to account for the slower stabilization of WAS as compared with PS and a lower SRT than is the case for the currently underloaded digester.

Table 7-4 summarizes the loads and capacity of anaerobic digesters. A total of three digesters have the capacity for the build-out flow of 20 mgd. Four digesters will be provided (3 duty plus one standby).

Table 7-4 Digester Design Parameters

Parameter	Quantity	Unit	Comment
Solids Retention Time (SRT) = Hydraulic Retention Time	20	days	Minimum required is 15 days to meet vector attraction reduction per 503 regs. 20 days is slightly conservative.
VSS Loading	150	lb per 1,000 cf per day	Range is 100 to 300 for anaerobic digesters operated at 95 deg F.
Limiting Sizing Parameter	--	--	SRT
Digester Tank Diameter	85	ft	Match existing
Digester Tank Side Water Depth	27	ft	Match existing
Digester Tank Working Volume	153.2	1,000 cf	Ignores conical bottom volume, due to likelihood of loss of cone due to settled solids.
Design Feed	56,000	gpd	Limiting factor
Actual SRT	20.5	day	
WRF Flow Capacity Each Digester	6.84	mgd	

7.2.3.2 Ponds

Due to space constraints, the area now occupied by Pond 2 will be required for additional Sludge Drying Beds. Since Pond 2 is the point of discharge for WAS at the current time, it will be necessary to move away from the ponds as a means of storing and stabilizing WAS.

In the near term, the following are characteristics of Pond 2 in its role as WAS storage and stabilizing facility:

- Stabilization of WAS is effective, yielding a SOUR of 0.6 mg O2/hr/gm, which is well below the 503 regulation of 1.5.
- WAS that is dredged from the ponds is between 2.0 and 2.5% solids, indicating that Pond 2 with its dredging arrangement is also quite effective in WAS thickening.[23]

Pond 2 must remain in service for WAS storage at least until a mechanical thickening facility has been commissioned.

7.2.4 Solids Dewatering and Drying

Digested sludge is now dewatered using belt presses. Stabilized WAS dredged from Pond 2 is also dewatered via belt press. What will change in the future solids handing scheme is that WAS will be thickened and then TWAS will be fed to the anaerobic digesters in parallel with primary sludge. Refer to "Solids Process Flow Diagram – Phase 2b" in Appendix E.

Other means of dewatering digested municipal sludge are available. A brief discussion of each follows:

<u>Centrifuge</u>

Centrifuges are often used at large wastewater plants, especially when drier cake is a premium concern, when limited space is available for the dewatering device, and when staff are available for the relatively complex maintenance requirements for a large rotating machine (typical speeds are 3,000 rpm). Centrifuge dewatering is best done on a 24-hour operating shift due to the 15 to 30 minute initial period required to begin producing good quality cake. Daily starting and stopping of dewatering activities is not a good fit for centrifuge dewatering.

Centrifuges can produce 4 to 6% points of drier cake than belt presses will do. For example if the belt press produces 17% cake, the centrifuge will produce 21% to 23% cake, possibly better. Centrifuges also do not require washwater (which a belt press uses continuously).

There are several reasons why centrifuge dewatering is not recommended for the WRP.

- The WRP has solar drying beds and will benefit less than some agencies would for slightly drier cake.

[23] Note that when feed solids to a belt press exceed 1.3%, the belt press is solids limited. Hence feeding thicker than 1.3% WAS is not necessary. Thickening is advised, however, if WAS is less than 1.3% solids.

- The WRP already has two belt presses. Adding another technology for dewatering would complicate operations and maintenance.
- Space is available for belt press dewatering; the benefit of a smaller footprint does not carry as much weight at the WRP.
- Current staffing does not support 24-hour dewatering operation.

Screw Press

Screw presses can achieve a similar cake percent as the belt press. The units available are relatively long and would not fit in the existing building without considerable modification. The advantages are these units can run with relatively little operator attention and they operate at a very low rpm. The screw press, like the centrifuge, does not require washwater.

The benefits of the screw press do not outweigh the space difficulties of fitting this into the existing plant, or of introducing a new dewatering technology.

Dewatering Technology Selection

For the WRP, continued use of and expansion of the existing belt press type of dewatering is recommended.

7.2.4.1 Belt Presses

The WRF has two 2-meter 14-roll belt presses at this time. Each belt press has two belts and the sludge is squeezed with progressively greater pressure to force the liquid in the sludge through the belt fabric, retaining the solids between the belts until discharged. Belts are normally woven synthetic fibers with stainless clipper or zipper seams. The seam is a wear point. The rollers are often made of stainless steel, perforated and with rubber coatings. Other key elements of the belt press are:

- Bearings
- Belt Tracking System
- Belt Wash System
- Drainage Capture
- Discharge Blades
- Controls

The presses are operated so that one unit dewaters digested primary sludge and the second dewaters WAS dredged from Pond 2. Both presses operate less than 40 hours per week at current flows and loads.

The building housing the belt presses should be configured to permit the use of either a bridge crane or space to allow use of a rolling crane to remove rollers from the belt press when needed. A rolling crane is the more economical choice, since unless there is a struvite[24] problem with the digested sludge, roller removal should be occurring once every 10 years. However, each belt press has 14 rollers. Expected maintenance on the rollers is replacement bearings and occasional replacement of rollers due to damage or wear.

Design criteria and loading projections are shown in **Table 7-5**.

Table 7-5 Belt Press Parameters[25]

Parameter	Quantity	Unit	Comment
Liquid Loading	50 - 75	gpm/meter	
Solids Loading	400 - 750	lb/hr per meter	Expect the belt presses to be solids limited in all cases.
Polymer Dose	6 - 16	lb active per ton dry solids	
Solids Capture	85 - 95%	--	Typical is 90%+
Cake Solids	16 – 25%		
WRF Capacity each Belt Press (Condition A)	5.4	mgd	Condition A: 9-80 schedule operation (9 days and 80 run hours every two weeks)
WRF Capacity each Belt Press (Condition B)	7.2	mgd	Condition B: 12 hour per day 5 days per week operation

For the build-out flow, a total of three duty plus one standby belt press will be required, operating at the Condition B hours in Table 7-4. Alternatively, all four belt presses can operate at the Condition A schedule and process all solids with no backup.

7.2.4.2 Sludge Drying Beds

Following dewatering, biosolids are hauled and spread to the WRF's paved drying beds. Solar drying drives off moisture and increases the fully dried cake to 90% solids. This degree of drying can qualify the dried biosolids for Class A pathogen reduction criteria as per the 503 regulations, if further testing is conducted. However, the current means of disposal (see following paragraph) does not require a Class

[24] Struvite (magnesium ammonium phosphate) is a mineral that can precipitate out of digested sludge and deposit on piping or equipment that handles digested sludge.
[25] Figures are typical for anaerobically digested blend of primary and WAS solids.

A product. Hence, the drying to 90% is for reduced hauling cost, not for achieving Class A pathogen reduction at this time.

Sizing of drying beds is based on the parameters shown in Table 7-6 below. Calculation is in **Appendix B Sludge Drying Beds**.

Table 7-6 Solar Drying Beds Design

Parameter	Quantity	Unit	Comment
Cake Solids Applied	1,787	lb/MG	For future condition wherein TWAS is sent to digester. Note this is less than the 2,021 lb/MG used for capacity without TWAS digestion.
Cake Solids Content	16%	--	Low end of belt press cake solids for anaerobically digested sludge
Dried Solids Content	90%	--	Desired solids content to limit hauling and disposal cost
Annual Rainfall	3.47	inches/year	http://www.usclimatedata.com/climate/indio/california/united-states/usca0512
Evaporation Rate	105	inches/year	Mean pan evaporation rate, Indio Date Garden
Resulting Area per mgd	0.26	acres	Note that the cake solids sent to the drying beds reduces the existing drying bed capacity requirement from 0.29 acres/mgd to 0.26.

7.3 BIOSOLIDS ULTIMATE DISPOSAL

The District currently contracts with Solids Solutions for dried biosolids removal and disposal. Hauling trucks come to the site approximately once every 16 to 18 months to empty the drying beds and transport solids to Yuma, Arizona for disposal. VSD is charged $45/ton of solids hauled.

7.4 CO-GENERATION FEASABILITY ANALYSIS

An analysis was done to determine the feasibility of installing co-generation facilities at VSD's wastewater treatment plant. A co-generation system utilizes the biogas generated in anaerobic digesters to fuel an engine that provides electricity for the plant to use and heat energy to maintain appropriate temperatures in the digesters. A typical co-generation schematic is shown below, in **Figure 7-2**.

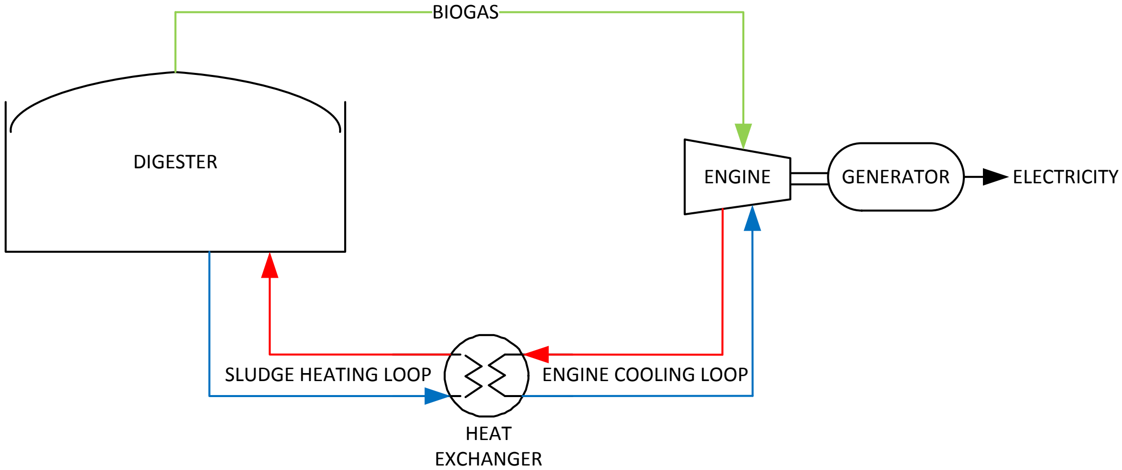


Figure 7-2 Typical Co-Generation Schematic

7.4.1 Biogas and Energy Production

The amount of biogas that is produced at the facility is a primary factor in determining the feasibility of co-generation. The solids production values presented in **Sections 3 and 7** were used in conjunction with biogas production parameters to determine the amount of biogas production that would occur at build-out. **Table 7-7** summarizes the parameters from plant data provided by VSD as well as industry standard values that are typically used.

Table 7-7 Biogas Production Parameters

Parameter	**Units**	**Historical Data Value**	**Industry Standard Value**
Volatile Solids Reduction	%	72.1 %	55 %[26]
Biogas Yield	ft^3/lb VS Destroyed	17.8	15
Methane Content	%	-	60 %
Heating Value	BTUs/ft^3	-	600
Solids Retention Time	Days	57.4	20

The data provided by VSD shows above average values for volatile solids reduction (VSR) and biogas yield which can be attributed to the extremely high solids retention time (SRT) of 57.4 days. At build-

[26] Note that the balance of the Master Plan used 60% VS reduction. 55% represents a more conservative figure for estimating digester gas available for co-generation.

out, the facilities have been designed to provide an SRT of 20 days with four digesters. Industry standard values were used in this evaluation to estimate the biogas production at build-out. **Table 7-8** summarizes the calculated biogas production at build-out, detailed calculations are available in **Appendix B Cogeneration**.

Table 7-8 Biogas Production at Build-Out

Parameter	Units	Value
Total Solids Loading	lbs/day	57,000
Volatile Solids Loading	lbs/day	47,913
Volatile Solids Destroyed	lbs/day	26,352
Biogas Production	ft^3/day	395,283
	ft^3/hr	16,470
	BTU/hr	9,882,069

As shown above, biogas production at build-out is estimated at roughly 9.88 MBTU/hr, this translates to a total annual production value of 86,567 MBTUs per year.

In order for co-generation to be technically feasible, energy production provided by the biogas must be greater than the energy required to maintain acceptable digestion temperatures in the anaerobic digesters. The heat energy required by the digesters was calculated for each month of the year based on climate data provided in **Section 2**, these calculations are available in **Appendix B Cogeneration**. As **Figure 7-3** shows below, there is a substantial amount of energy available above what is required for heating the digesters at build-out.

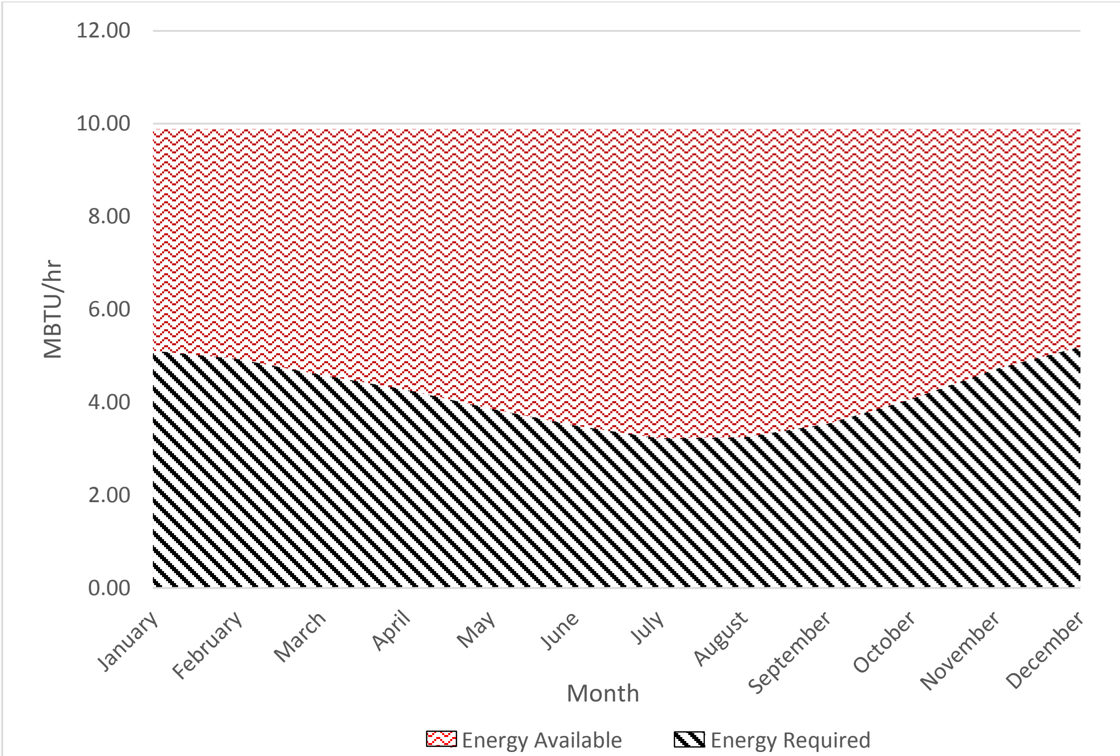


Figure 7-3 Energy Required versus Energy Available

Given the large amount of excess energy available at build-out, co-generation is technically feasible at VSDs wastewater treatment plant as far as the energy balance is concerned.

7.4.2 Co-Generation Technology Review

There are several different technologies available to implement co-generation with the use of biogas produced through anaerobic digestion. The most common technologies used for co-generation systems using biogas as fuel are internal combustion engines (ICEs), micro-turbines, and fuel cells. These technologies are compared in **Table 7-9**.

Table 7-9 Co-Generation Technology Comparison

Technology	Pros	Cons
Internal Combustion Engines	• High efficiency • Fast start-up • Relatively low cost • Operate on low pressure gas • Durable in varying conditions	• High emissions • Must be cooled even if recovered heat is not used • High levels of low frequency noise
Microturbines	• Compact size • Not many moving parts • No cooling required	• High costs • Low mechanical efficiency • Limited heat recovery available • Documented problems in similar climates to VSD
Fuel Cells	• Low emissions • Low noise • High efficiency • Modular design	• High costs • Require a very high level of biogas treatment • Limited installations using biogas

The technologies compared above were also evaluated based on their electrical, heat recovery, and overall efficiencies. The efficiencies for each technology are outlined in **Table 7-10**.

Table 7-10 Co-Generation Technologies Efficiency Comparison

Technology	Electrical Efficiency	Heat Recovery Efficiency	Overall Efficiency
Internal Combustion Engines	36 %	45 %	81 %
Microturbines	28%	38 %	66 %
Fuel Cells	42%	30 %	72 %

Each of the co-generation technologies evaluated are available in containerized skids which eliminate the need for a dedicated generator building and simplify the construction process by providing simple connection points for biogas and for tying in the existing sludge heating water loop. **Figure 7-4** shows an example of a containerized ICE.

Figure 7-4 Containerized ICE

Internal combustion engines (ICEs) are the most efficient technology among the co-generation technologies evaluated and typically the most economic technology available. They have been proven in many installations to be long lasting, especially when the biogas is treated appropriately to reduce maintenance requirements. ICEs are the best technology available at this time to pursue for the implantation of co-generation at VSD.

Another emerging technology that should be considered for future implementation at VSD is Organic Rankine Cycle (ORC) systems. ORC systems utilize waste heat to provide additional electrical power and can use the surplus heat generated in co-generation systems. Given that ICEs will allow for more heat to be recovered than will be required by the digesters, the excess heat may be able to run an ORC system. ORC systems have gained popularity in Europe and the progress of this technology should be monitored for possible implementation at VSD in the future.

7.4.3 Biogas Treatment

Biogas generated in anaerobic digesters is a mixed gas composed primarily of methane and carbon dioxide with trace amounts of other gases, such as hydrogen sulfide, nitrogen, water vapor, and siloxanes.

Water vapor, hydrogen sulfide, and siloxanes should be removed when digester gas is used as a fuel in any co-generation application. The levels of these compounds in the biogas depend on the sludge and treatment process and the level of pre-treatment required depends on the final use of the biogas.

Hydrogen sulfides are typically removed using either liquid or solid scavengers. The most common system used for hydrogen sulfide removal is the iron sponge. An iron sponge uses a media covered in iron oxide or hydroxide which the hydrogen sulfide attaches to as biogas is run through the media. Eventually the media becomes saturated and must be replaced. There are several proprietary technologies that use more efficient coatings and media to remove hydrogen sulfide.

Water vapor is typically removed from biogas with the use of chillers which allow the moisture to condense, precipitate, and be collected in moisture traps. Absorption and adsorption can also be used to remove water vapor with the addition of chemicals such as glycol or silica gels.

In internal combustion engines, siloxane is reduced to silica and oxygen. The free silica readily deposits on hot surfaces in the form of white silica powder. These deposits can accumulate inside engines, greatly increasing maintenance costs and reducing efficiency. An example of silica deposits on an engine cylinder head is shown in **Figure 7-5**.

Figure 7-5 Siloxane Deposits on Cylinder Head

There are also environmental regulations that drive the need to treat biogas. Specifically, South Coast Air Quality Management District (AQMD) Rule 1110.2 sets emissions limitations for biogas fueled engines. These emission limits are summarized in **Table 7-11**.

Table 7-11 Emissions Limits as per Rule 1110.2

Parameter	**Units**	**Value**
NO_x	ppm	11
VOC	ppm	30
CO	ppm	250

The NO_x level (11 ppm) required by Rule 1110.2 is impossible to achieve with an internal combustion engine alone, and the engines must be used in conjunction with an emissions control device. The only proven method for reducing NO_x emissions to an acceptable level is the use of selective catalytic reduction (SCR). SCR systems selectively reduce NO_x to N_2 in the presence of a reducing agent. The use of an SCR requires the removal of nearly all siloxanes as siloxane deposits have been known to deactivate the reducing agents required in SCRs within hours of startup.

There are two primary methods used for the removal of siloxanes from biogas, either consumable media systems or regenerative media systems. Consumable media systems usually consist of multiple vessels in series with activated carbon that absorb the siloxane deposits. Eventually the activated carbon becomes saturated and must be replaced. Regenerative media systems require at least two treatment vessels in parallel so that one may be used while the other is in regeneration mode. The regeneration process usually purges the media by back-flowing hot air through the vessel and flaring the purge air to eliminate the emission of greenhouse gases.

Typically, complete gas treatment systems can be provided by a single vendor to remove water vapor, hydrogen sulfides, and siloxanes as required by the specific co-generation system utilized. It will be necessary to test the biogas generated by the digesters at VSD at the time of implementation in order to determine the extent of biogas treatment required and the best system to use to accomplish the required treatment.

7.4.4 Recommended Co-Generation System

The recommended co-generation system for implementation at VSD consists of containerized internal combustion engines and further biogas treatment with the addition of a biogas treatment system that is provided by a single vendor, mounted on a skid.

Two containerized ICE packages consisting of the generator set, co-generation system, and switchgears should be utilized. Connection points for biogas and integration into the existing sludge heating water loop will be available on the containers. The approximate electrical output of each of these units will be 630 kW, providing a total output of 1260 kW.

At the time of implementation, the biogas being produced at VSD should be tested to determine the specific composition of the biogas and determine the treatment needed. A complete treatment system should be utilized that operates as a standalone system or in conjunction with the existing gas scrubbers.

The piping from the existing biogas system to the treatment skid and to the ICE engines should be 304 SS and have a diameter of 10” to accommodate the expected biogas flows. Piping used to tie in the ICE engines to the digester heating water loop should be matched to the existing pipe in size and material. Piping that routes heated water from the ICE engines should be insulated. Instrumentation and control systems should be provided by the ICE and biogas treatment vendors and integrated into the existing SCADA system as desired by VSD.

The system described above was used to calculate a return on investment for implementing co-generation at VSD assuming that VSD would purchase, install and operate those units.

7.4.5 Return on Investment Calculation

In order to determine the financial feasibility of implementing co-generation at VSD, a simple return on investment (ROI) was calculated for a 10 year period after implementation of co-generation. This simple calculation did not account for escalation or inflation.

The return on investment was calculated based on estimated electrical production revenues, natural gas savings, O&M costs, and capital costs. The equation used is shown below.

$$ROI = \frac{(Annual\ Elec.Revenue + Annual\ Nat.Gas\ Savings - O\&M\ costs)x\ 10years - Capital\ Costs}{Capital\ Costs}$$

Based on a 20 year design life, the 10-year ROI must be at least 50% to make implementation of co-generation feasible at VSD. The utility costs outlined in **Table 7-12** were used in the ROI calculation and were obtained from the United States Energy Information Administration (EIA).

Table 7-12 ROI Calculation Parameters

Parameter	Units	Value
Electricity Cost	$/kWh	.108
Natural Gas Cost	$/MBTU	7.75

Capital costs were determined by first calculating the main equipment costs and then estimating construction costs. The cost of containerized ICE engines was determined by contacting vendors and obtaining quotes. The cost of the biogas treatment system was determined by using the Biogas Toolkit Cost Estimator provided by AQMD. The estimator from AQMD was used due to the uncertainty of the biogas quality that will be available at the time of implementation. The estimated capital costs for the major equipment components are outlined in **Table 7-13**. A breakdown of the total estimated capital costs is provided in **Appendix B Cogeneration**.

Table 7-13 Co-Generation Equipment Cost Estimates

Parameter	Units	Value
ICE Engine Cost	$	1,980,000
Biogas Treatment System Cost	$	1,495,460

The operational and maintenance costs for the ICE engines were estimated to be approximately $0.02 per kWh based on information provided in the "Catalog of CHP Technologies" published by the USEPA. This value was confirmed with ICE engine vendors. The O&M costs for the biogas system were estimated using Biogas Toolkit Cost Estimator provided by AQMD. The estimated O&M costs are shown in **Table 7-14**.

Table 7-14 Co-Generation Estimated O&M Costs

Parameter	Units	Value
ICE Engine O&M Cost	$/year	199,623
Biogas Treatment System O&M Cost	$/year	174,598

The values used to calculate the ROI for implementing co-generation at build-out are summarized in **Table 7-15**. Detailed calculations are provided in **Appendix B Cogeneration** for implementation at various plant flows.

Table 7-15 ROI Calculation, Co-Generation Implemented at Build-Out

Parameter	Units	Value
Electrical Revenue	$/year	1,005,553
Natural Gas Savings	$/year	284,146
O&M Costs	$/year	374,221
Project Costs	$	7,366,580
ROI	%	39 %

The 10-year ROI for implementing co-generation is highly dependent on the current cost of electricity, and the ROI was calculated for implementation in Phase 3, Phase 4 and for build-out at a range of electrical costs. **Figure 7-6** depicts the results of this evaluation.

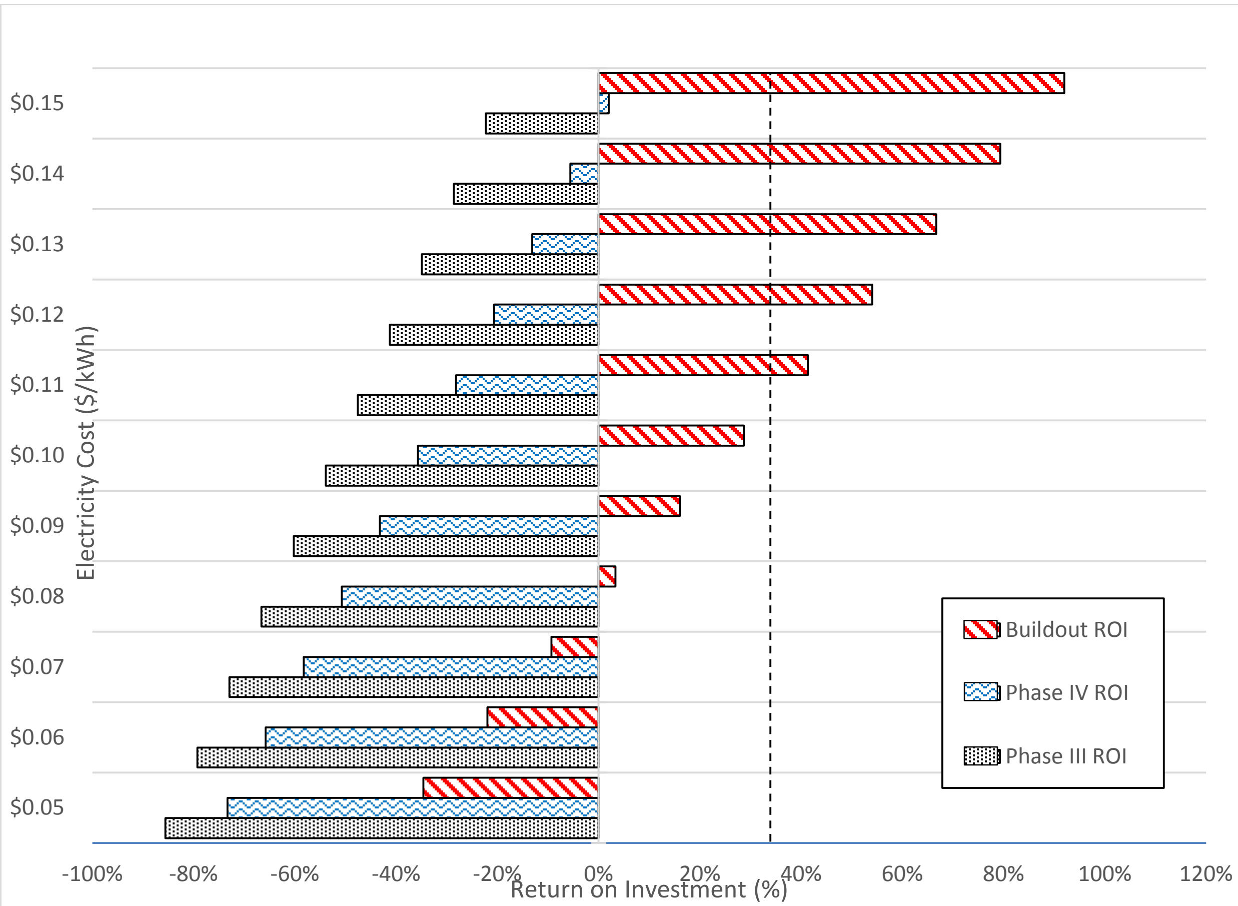


Figure 7-6 ROI versus Electrical Cost

At the current electricity cost of $0.107/kWh implementation of co-generation does not provide the required ROI of 50% to be financially feasible. As the figure above shows, electricity costs need to rise above $0.11/kWh to provide an adequate ROI.

7.4.6 Power Purchase Agreement

Another option for VSD would be contracting a specialized company through a Power Purchase Agreement (PPA) to purchase, install, maintain, and operate cogeneration units at the WRF. Such a company would guarantee an electrical output given a guaranteed gas production by VSD. VSD, in exchange, would buy the power produced by the cogen units.

Typically, those types of agreement become beneficial when the production of the biogas production of the WRF reaches 100,000 cf/day. This type of gas production would be attained at VSD at around 6 mgd once TWAS digestion starts occurring.

Depending on the size of the system and the cost of power, electricity rate could be as low as 80% of the utility rate, which would represent cost savings for VSD as well as biogas reuse. A sample contract is provided in **Appendix B Cogeneration**.

7.4.7 Conclusion

Operating cogeneration units requires very specialized knowledge and skills, and constant attention. In addition, the ROI was shown to be very limited given the electricity rates paid by VSD and the anticipated flow and biogas production if VSD had to purchase, operate, and maintain the units.

Alternatively, a PPA would allow reuse of biogas without initial capital investment, would save power, and potentially reduce costs for VSD. In addition, the risks associated with operation of a complex gas conditioning and energy recovery system would be transferred to the PPA.

Section 8 – Recommended WRF Site Master Plan and Phasing

Section 8
Recommended WRF Site Master Plan and Phasing

In this Section, upgrades are recommended along with a design, construction, and commissioning timeline. Based on those recommendations, costs for the recommended upgrades are then detailed for each anticipated phase.

8.1 RECOMMENDED EFFLUENT REUSE BLENDS

The formation of a Joint Power Authority in the area in order to produce and distribute recycled water (Title 22), as well as the recent Recycled Water Feasibility Study (Carollo, 2013) suggestions and recommendations pertain to Tertiary Treatment at the WRF.

According to the feasibility study, the demand in the area is high enough to match the entire wastewater treatment flow of the WRF if a Title 22 (tertiary filtered) treatment stream were to be installed. Treatment processes and preliminary sizing are presented at build-out in **Section 6**.

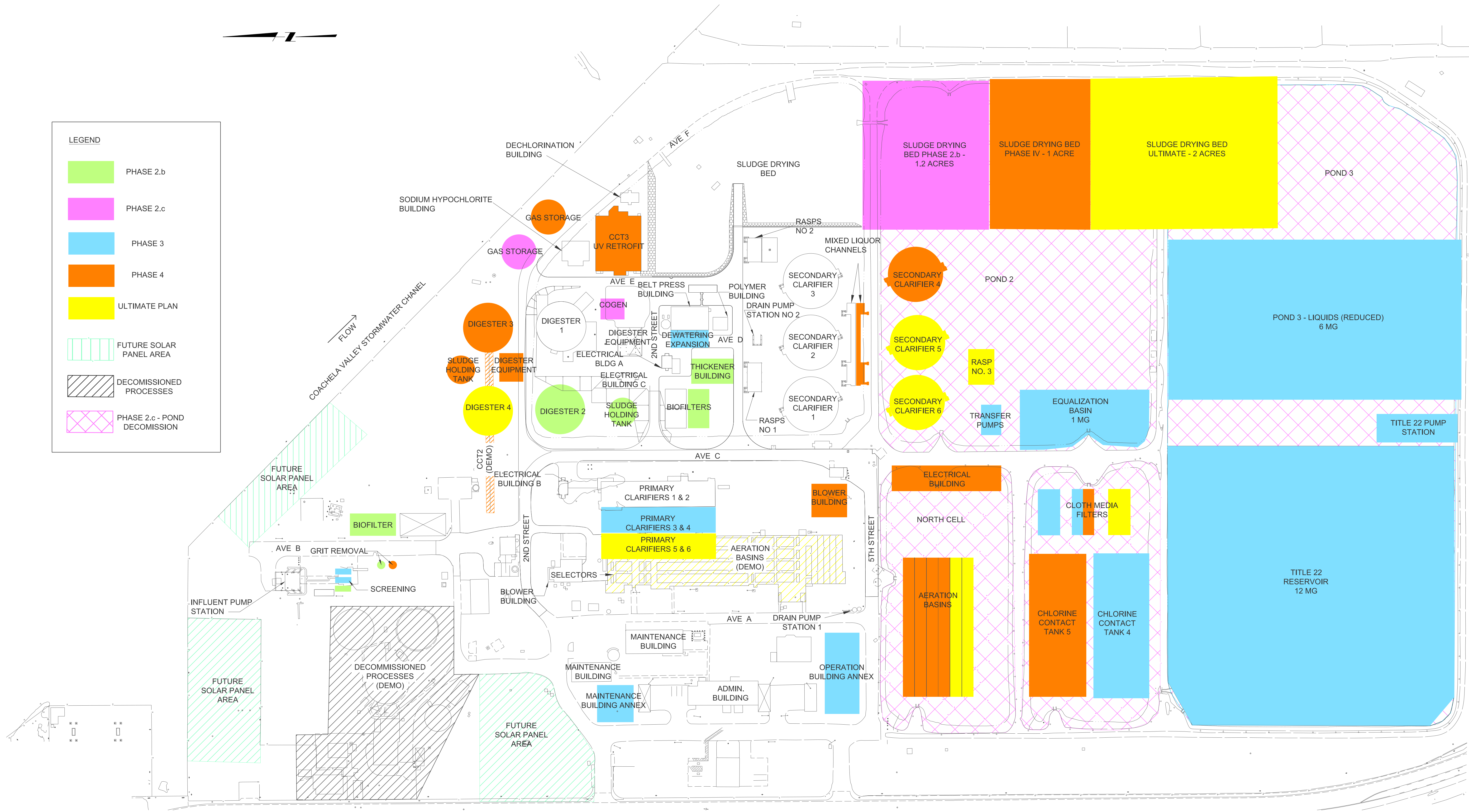
Although most of the time the effluent[27] will be treated to Title 22 standards and sent to a recycled water distribution system, during winter the demand will be significantly lower than the WWTP flow, and storage in the distribution system will not be able to contain potentially overproduced recycled water. It is then recommended to keep the disinfection and channel discharge treatment stream of the current WRF in service and able to treat the full Plant's flow for discharge.

The phasing presented below uses the recommendations above.

8.2 SITE MASTER PLAN PHASING

Four phases, Phase 2b, Phase 2c, Phase 3, and Phase 4, as well as Build-out upgrades were considered in this WRF Master Plan. Infrastructure updates required for each of those phases are detailed below. Design criteria for each phase are summarized in **Appendix C**. **Figure 8-1** shows the site plan for the existing plant and the recommended phasing. These are also provided in **Appendix D**.

[27] Note that approximately 1 mgd of effluent must be maintained as discharge to the CVSC for habitat maintenance. The bulk of the effluent would be reclaimed in summer months, excluding the 1 mgd.



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VALLEY SANITATION DISTRICT

Figure 8-1
Site Plan and Phasing

8.2.1 Phasing assumptions

Flow projections used to determine when design, construction, and commissioning should occur were based on population projections presented in **Table 2-5** through year 2050. The timing of the last phase, build-out, is not within the planning horizon and is not shown.[28] Design, construction, and commissioning duration assumptions are summarized in **Table 8-1**. Finally, capacity calculations per processes and per phases are presented in detail in **Appendix B**.

Table 8-1 Assumed Duration of Phase Stages

Stage	Design	Construction	Commissioning
Duration (years)	1.5	2.7	0.3

8.2.2 Phases 2b and 2c

Phase 2b and 2c include upgrades that are of highest priority for the WRF. Both phases will occur sequentially, since area required for drying beds will require decommissioning of Pond 2 as a WAS storage and stabilization facility. The design for those upgrades should start as soon as possible in order to provide continuous liquid treatment and solids disposal.

Capacity Upgrades

As shown in **Table 3-6**, some processes require immediate upgrade in order to treat the current existing flow of 6.0 mgd. Those processes are grit removal, and sludge drying.

- Grit chamber: one new 22-ft diameter vortex grit chamber should be installed to replace the existing undersized aerated grit chamber, and would provide 22 mgd of peak flow treatment capacity (10 mgd average flow) for grit removal. If the existing biofilter cannot be reused for the new chamber, one new biofilter would be installed to treat foul air from the grit removal unit.
- Sludge drying beds: existing sludge drying beds are currently rated to 5.9 mgd, although operation staff expressed concerns about the beds being already out of capacity, especially during the winter when solar drying is less efficient. WAS is currently stored in Pond 2. At present, stabilized WAS is dewatered using the dredge system at Pond 2, feeding one belt press about two days per week from October to April and five days per week from May to September. One belt press is fed digested primary sludge three days per week.[29] It is anticipated that 1.2 acres of additional drying bed will be required to comfortably extend the WRF capacity to 10 mgd. It is also recommended to install the drying beds extension adjacent to the existing ones in order to optimize sludge hauling time. This means that Pond 2 will have to be decommissioned in order to make room available for the additional drying bed once GBTs and additional digestion capacity is installed.

Redundancy

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[28] The 20 mgd buildout flow to the WRF is from the *Collection System Master Plan*, November 2013, p. 4-15.
[29] Conference call with VSD staff, May 21, 2015.

The WRF should consider installing additional units to add redundancy for the following processes:

- Bar screen: Add one bar screen to replace existing manual rack in the third screen channel. The bar screen added will be a multi-rake bar screen with ¼” opening as opposed to the existing ½” opening climber screens. The benefit is greater capture of solid materials that can pass through a ½-inch opening, but not a ¼-inch opening, thereby saving maintenance in downstream equipment and reducing rag buildup in the digesters. The finer screen will collect somewhat more fecal matter than the existing ½” climber screens, but since all flow goes through the Influent Pump Station, much of this matter will be dispersed. The added ¼” screen will be provided with a screenings washer and compactor to greatly reduce the objectionable matter in the washed and compacted screenings Typically, in fine bar screen configurations with pump upstream, no screenings disposal issues are to be anticipated as long as the screenings are run through a washer/compactor and ultimately sent to a landfill. In addition, as described in **Section 4.4.1.2**, head loss resulting from additional rag build-up would not be a concern due to the proposed multiple-rake screen type, providing more rake passes than the existing climber type. Sufficient free-board is available, along with emergency containment area around the structure in case of surcharge. Adjustment to the high level alarm set point may however be necessary.
- Digester and sludge holding tank: the Plant currently owns and operates one digester only. Even though it has sufficient capacity to handle primary sludge digestion up to 15 mgd of ADF, it is recommended to add an additional digester for redundancy. Redundancy would be particularly valuable to store the sludge when the time comes to take off line the existing digester for cleaning and maintenance. In addition, a sludge holding tank would be installed to allow the digesters to overflow in the sludge holding tank and to operate at constant level. The sludge holding tank provides equalization to enable dewatering to be conducted 5 days per week.
- Flare: only one flare rated at 7,200 standard cubic feet per hour (scfh) was installed at the WRF. It is anticipated that this gas flow will be produced by the digesters at around 9 mgd of ADF if the WAS is thickened and digested instead of being sent to Pond 2 as it is currently being done. However, if the existing flare should fail, there is currently no facility to store the biogas. Hence, it is recommended to install an additional flare for redundancy purposes. If cogeneration units were to be installed, a redundant flare would be less critical since it would only be used when the cogeneration unit is shut down.

Process Changes and Additions

In order to keep the drying bed expansion close to the existing ones, Pond 2 will have to be taken out of service to free space for drying bed installation. Since Pond 2 is currently used for WAS stabilization, prior to drying bed expansion, WAS must be thickened with Gravity Belt Thickeners (GBTs), and the TWAS sent to the digesters. Those changes would require the following modifications:

- Thickening: 2 GBTs will have to be installed to thicken the WAS, and three (3) pumps (2+1) installed to pump the TWAS to the digesters. Associated piping will also have to be installed.
- Drain lines re-routing: Two (2) drain pump stations currently discharge into Pond 2, and will have to be rerouted before Pond 2 is taken out of service. It is proposed to connect the discharge lines to the 48’’ sewer main in front of the office building.

Cogeneration PPA

Since TWAS digestion will occur GBT experience with cogeneration units operation and maintenance. Once the digestion of TWAS begins, gas production should be significant enough (>100,000 cubic foot per day) to make a PPA attractive for both VSD and a potential cogeneration business partner. If the gas volume and quality as well as the electricity buyback price is mutually beneficial, installation of the units would not require any capital investment from VSD.

The changes and modifications described above would bring the Plant's capacity up to 8.2 mgd of ADF treatment capacity.

8.2.3 Phase 2b and 2c sequencing

As mentioned earlier, Pond 2 will have to be decommissioned in order to install the additional sludge drying beds. Hence, two phases are required as presented below.

Phase 2b:

- Install 22-ft diameter grit chamber with biofilter
- Construct thickener building (2 GBTs) with biofilter
- Construct digester
- Install sludge holding tank
- Redirect drain water going to Pond 2 to sewer main (Plant influent)
- Conduct start-up and commission installed processes above.

The above items would permit cessation of Pond 2 for WAS storage and stabilization and enable the start of Phase 2c.

Phase 2c:

- Decommission Pond 2
    - Drain free water from Pond 2 and Pond 3 through CCT2 to the CVSC
    - Dredge sludge out of Pond 2 into Pond 3 for solar drying
- Build sludge drying bed extension
- Install digester gas holding facility
- Implement cogeneration with digester gas
- Start-up and commission installed processes

8.2.4 Phase 3

Phase 3 design should start 6 years before the projected flow reaches the post Phase 2b upgrade capacity of 8.2 mgd. It is anticipated to occur in 2030, which means that Phase 3 design should start 5 years before in 2025 according to **Table 8-1.**

Capacity Upgrades

Using standard hydraulic loadings, existing clarifiers have a capacity of 8.2 mgd. Additional basins will need to be installed adjacent to the existing ones as shown in **Figure 8-1**. Adding two basins, with a design identical to basins currently installed, would provide extra capacity and allow to operate with one basin off-line for maintenance.

The belt presses would reach their capacity assuming 9-hours shifts, and a third belt press would need to be installed to bring the capacity to 16.2 mgd. Since there is no room available for the new equipment, the belt-press building will need to be expanded as shown on the proposed site plan in **Figure 8-1**.

It is also anticipated that at flows at 8.2 mgd, additional maintenance and operation buildings will be required to account for both additional staff and equipment to maintain. Proposed locations and approximate sizes are presented in **Figure 8-1**.

End-of-life/Preventive Replacement

The bar screens at the plant have been installed in 1998 and by 2031 will have reached their useful life (30 years). Two new ¼-inch spacing bar screens will replace the two remaining older ½-inch spacing bar screens

Title 22 System

A tertiary treatment system would be installed to produce up to 10 mgd of Title 22 water for landscaping and irrigation in the surrounding area. As described in greater detail in **Section 6,** filtration, disinfection, equalization, storage, and pumping capacity will need to be installed to match the recycled water demand.

A 1 million-gallon (MG) equalization basin will be installed downstream of the secondary clarifiers to serve as a wet well for the filter influent transfer pumps to the cloth media filters. Three (3) 16-disks units will be installed, as well as a chlorine contact tank with chemical dosing system (CCT4). A 12 MG reservoir as well as a Title 22 pump station will be installed downstream of CCT4 as well.

The Title 22 infrastructure is shown in **Figure 8-1.**

Even though a Title 22 system is not crucial for wastewater treatment performances and compliances, given the drought in California, it is also advised to have the installation as soon as possible to reduce potable water consumption in the area.

Ponds

Pond 3 will be retrofitted into the 12 MG Title 22 reservoir and 6 MG of liquid peak shaving capacity. The South Cell will be decommissioned and the footprint used for filters and chlorine contact tank as shown in **Figure 8-1**. North cell would be decommissioned to plan for future Phase 4 expansion.

The changes and modifications described above would bring the Plant's capacity up to 10 mgd of ADF treatment capacity. Even though this phase does not represent a significant increase in treatment capacity, it still is a major capital investment because of the Tertiary treatment system. Changes are shown on a site plan in **Figure 8-1**, and on the process flow diagrams in **Appendix E** (E.III A through E.III D).

8.3 PHASE 4

Phase 4 design should start 6 years before the projected flow reaches the post Phase 3 upgrade capacity of 10 mgd. It is anticipated to occur in 2048, which means that Phase 4 design should start 6 years before, in 2043.

Capacity Upgrades

A list of Phase 4 capacity upgrades follows.

- One additional grit removal chamber, identical to the one built in Phase 2b, will be installed.
- One additional belt –press
- One additional gravity belt thickener (GBT)
- Four brand new secondary aeration basins, completely replacing the existing selectors and aeration basins.[30]
- New mixed-liquor channel, since the existing channel would not be hydraulically suitable at build-out.
- New blower building.
- One new secondary sedimentation basin
- RAS and WAS pumping capabilities
- One (1) acre of additional sludge drying bed will be installed
- One additional filter unit
- One additional chlorine contact tank, CCT5

Redundancy

For redundancy purposes, the following items are included in Phase 4 construction.

- Additional digester to operate in a 2+1 (two digesters in service, one digester standby) configuration.
- Additional digester equipment including flare, boiler, and pumps
- A second sludge holding tank will be installed in order to operate the new digester (and future one) at constant level.
- Retrofit of CCT3 to UV
- Electrical upgrades including new Electrical Building to house switchgear and emergency power generators

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[30] The new basin will be required since it is anticipated that at this time, treatment capacity of the existing basins will be close to reaching their limit, and additional aeration volume might be required for nitrogen removal.

The existing chlorine contact tank, CCT3, has a 14.2 mgd capacity with both its channels online. However, if one channel was shut-down for maintenance purposes, the remaining basin would not be able to treat the existing flow. It is then recommended to retrofit both basins to UV disinfection. This transition to UV is particularly recommended in this situation given that the coliform discharge level is easily reachable using a UV system. It also would eliminate the need to dechlorinate along with the risk to discharge chlorine into the Chanel. Please note that this option is based on maintaining the existing coliform limitations for discharge to the Channel.

Phase 4 Summary

The changes and modifications described above would bring the Plant's capacity up to 13.3 mgd of ADF treatment capacity. Changes are show on a site plan in **Figure 8-1**, and in the process flow diagrams in **Appendix E** (E.IV A through E.IV D).

8.3.1 Phasing summary

Existing process capacity, along with incremental capacities from each phase are summarized in **Figure 8-2.** Similarly to the existing capacities, each phase capacity is controlled by the lower capacity of each of the treatment processes.

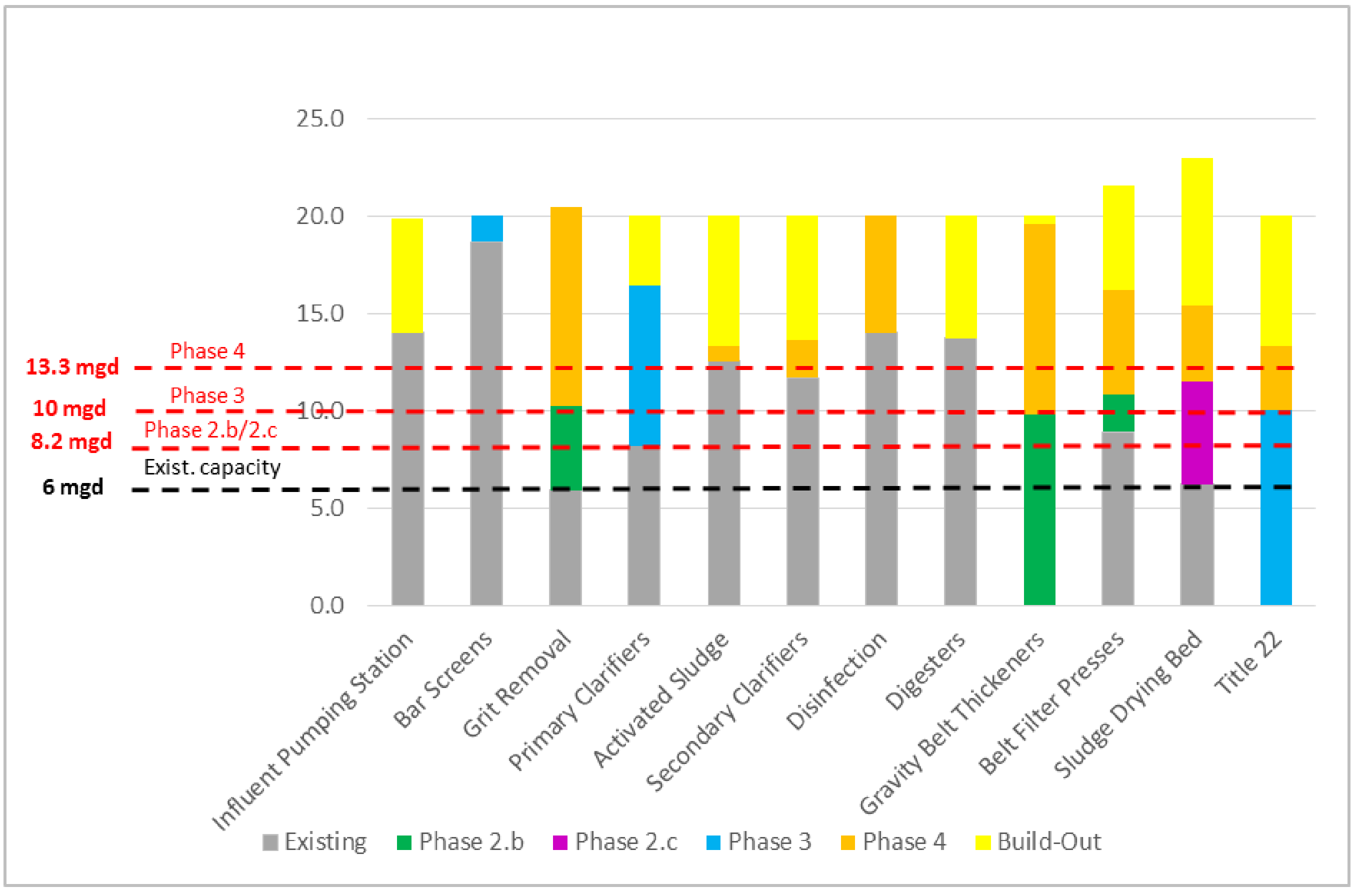

Figure 8-2 Process Treatment Capacities by Phases

8.3.2 Build-out

No projection was performed in term of anticipated year for Build-out flows. However, in order to lay-out previous phases' infrastructure, the required infrastructure at build-out was determined to plan for future upgrades, using a 20-mgd ADF build-out flow obtained from MWH *Collection System Master Plan* (November 2013, page 4-14). Detailed equipment list at build-out is provided in **Appendix C**, and site plan in **Figure 8-1.** Process upgrades are summarized below:

- Primary clarifiers: two (2) new primary clarifiers channel, primary clarifiers 5 & 6, will be installed.
- Aeration basin: two (2) new channels will be added in the aeration basin.
- Secondary clarifiers: two (2) additional primary clarifiers will be added.
- Digester: one (1) digester along with related equipment will be added to continue operating with one redundant unit.
- Sludge drying beds: two (2) additional acres would need to be added.
- Cloth-media filters: two (2) additional 16-disks units will be installed.

Flow projection and plant capacity per phases versus time are presented in **Figure 8-3**.

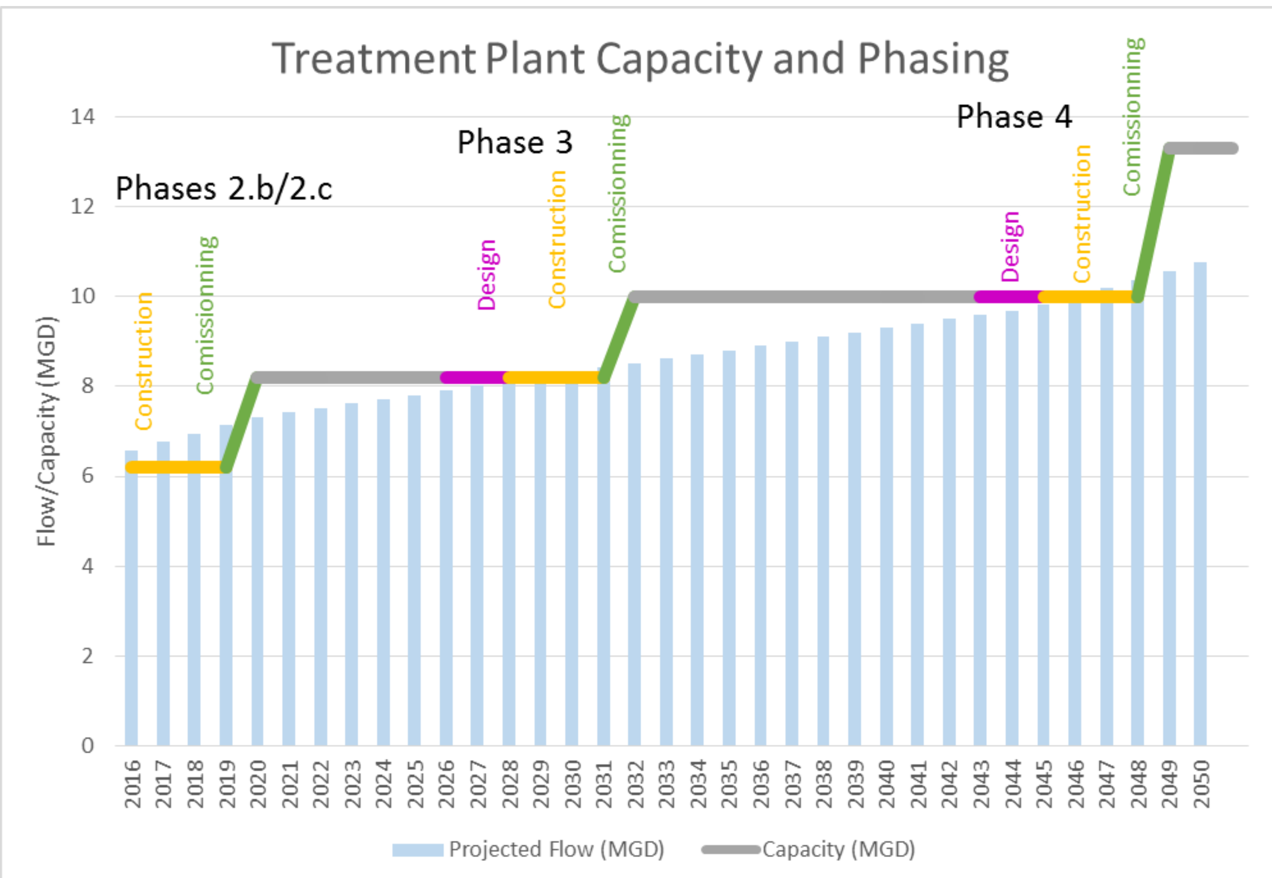

Figure 8-3 Flow Projection, Pant Capacity, and Proposed Phasing

8.4 BIOLOGICAL TREATMENT POND SYSTEM DECOMMISSIONING PLAN

The existing Biological Treatment Pond System (BTPS) serves no wastewater treatment purpose as discussed in Section 3.

All three cells may be fully decommissioned, or a partial decommissioning may be done leaving one or two cells in place. Partial decommissioning will reduce but not eliminate the annual cost of maintaining the system. However, it is recommended to completely decommission all three of the BTPS cells. Two alternatives are presented below for the BTPS decommissioning.

Alternative 1 (Preferred Method):

- *Step 1:* Isolate BTPS Cells A, B, and C from the upstream treatment pond system by closing the valve upstream of Cell A as shown on **Figure 8-4**.

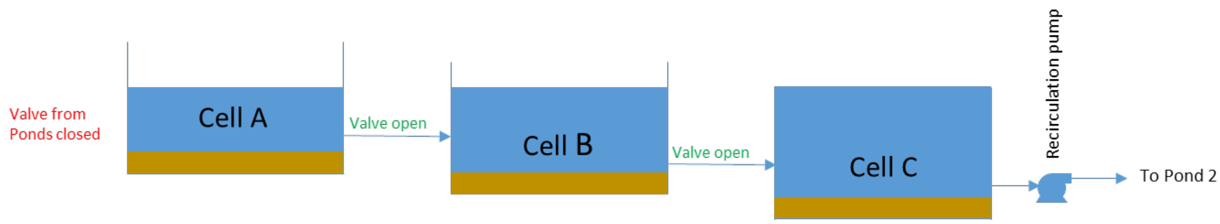

Figure 8-4 BTPS Decommissioning: Step 1

- *Step 2:* Continue via the recirculating pump currently installed in Cell C to transfer water from the BTPS to Pond 2 as shown on **Figure 8-5**. Since the three cells A, B, and C will still be hydraulically connected, the entire system water level will drop until cell A then cell B water level reaches its respective outlet pipe effluent invert elevation.

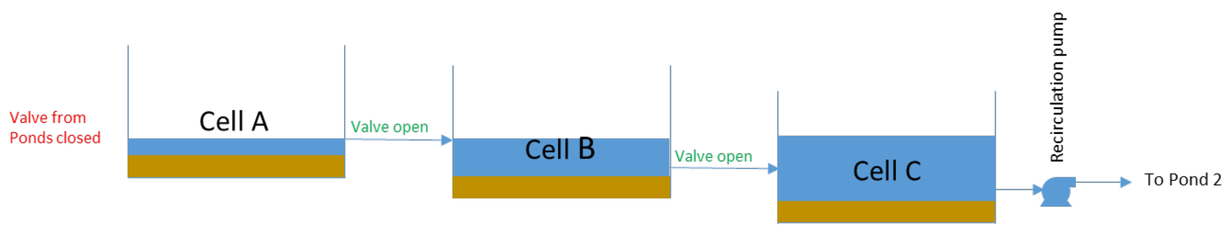

Figure 8-5 BTPS Decommissioning: Step 2

- *Step 3:* Close the valve between cell A and cell B. Install a VSD-owned, 1-mgd transfer pump in cell A to transfer its remaining water to Cell C while Cell C water is transferred to Pond 2 as shown on **Figure 8-6**. When all the water has been transferred, the sludge at the bottom of Cell A is left to dry.

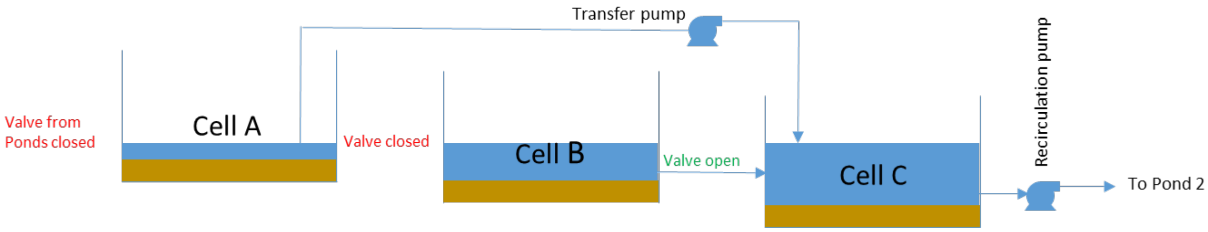

Figure 8-6 BTPS Decommissioning: Step 3

- *Step 4:* Shut down the recirculation pump, move the transfer pump to Cell B to transfer its water to Cell C, and restart the pumps to empty Cell B as shown on **Figure 8-7**. When all the water has been transferred, the sludge at the bottom of Cell B is left in the cell to dry.

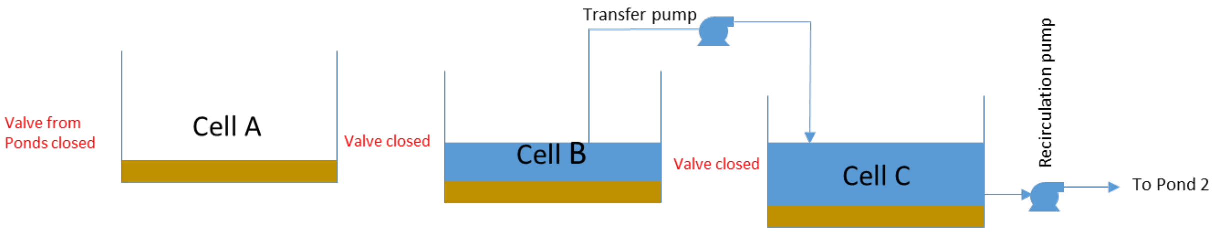

Figure 8-7 BTPS Decommissioning: Step 4

- *Step 5*: Close the valve between Cell B and Cell C. Empty Cell C using the recirculation pump as shown on **Figure 8-8**. The sludge at the bottom of Cell C is left to dry as shown on **Figure 8-9**.

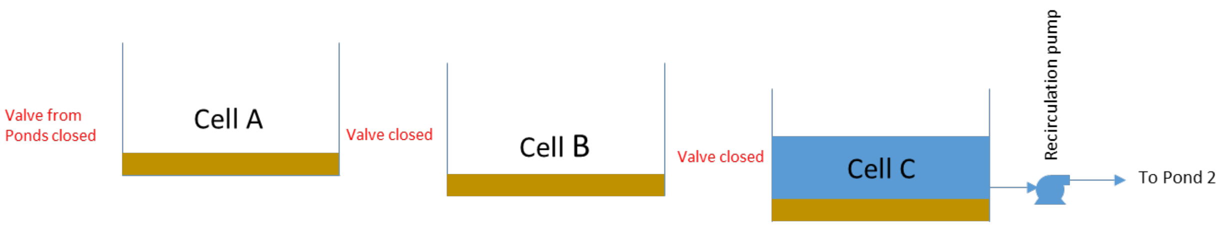

Figure 8-8 BTPS Decommissioning: Step 5

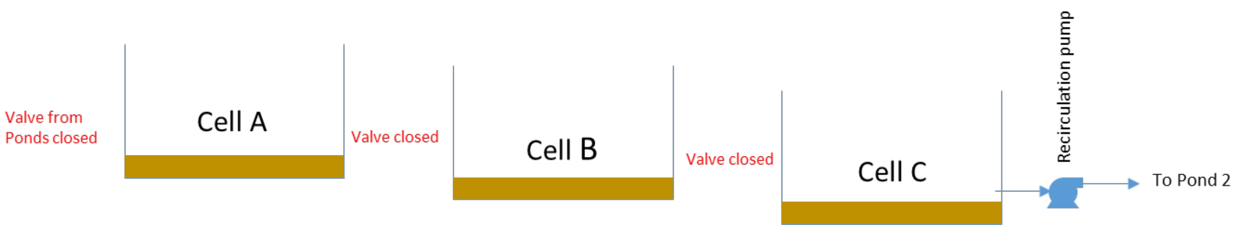

Figure 8-9 BTPS Decommissioning: Sludge Drying

Once the sludge is dry, it will be tested to verify that it is not hazardous and large openings will be cut into the liner at all low points[31] in order to allow drainage of rainfall once the cells are filled with dirt.

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[31] If the bottoms of the cells are found to be flat, a backhoe may be employed to tear openings in the liner on a 40-ft grid. Estimating five days per cell for liner ripping in that case, costs $300 per hour for operator + equipment.

The cells will be back-filled with native material initially used to build the BTPS embankments. It is assumed that no import will be necessary to backfill the cells.

An estimation of the time required to drain the cells is presented in **Table 8-2**. Note that Table 2 emptying times for cells is based on operating the transfer pump nine hours per day.

In order to take into account the time necessary to move the transfer pump from one Cell to the other, staff availability, and other unplanned events, it is conservatively anticipated that the Cells will be drained 4 days per week. The entire system drainage would take a total of 33 weeks.

Drying the sludge before burying it will take some time as well. During summer time, pan evaporation would range between 14 and 18" per month, and would allow to dry the sludge much faster than during the rest of the year. The sludge depth may reach two feet within the Cells. In all probability it will be necessary to turn over the sludge, similarly to how the drying beds are currently being operated, in order to accelerate the sludge drying process. If the sludge is regularly turned over and dried during the summer, six weeks should allow sufficient drying time for the sludge contained in a given Cell. Since Cell C will be the last pond to be emptied, and assuming that sludge is being turned over in empty cells while the other cells are being drained, a total of 39 to 45 weeks (33 weeks to empty the cells, 6 weeks to dry the sludge in summer, 12 weeks in winter) will be necessary to get the Cells ready to be back-filled with native material after having the liner perforated at critical places to allow for drainage.

This last step (backfill) should take about 10 weeks of actual work by a Contractor with the appropriate equipment.

A proposed schedule is shown in **Table 8-2**.

Table 8-2 Proposed Schedule for Alternative 1

Task	Schedule
Empty the cells	March to October 2016
Dry the cell	November 2016 to February 2017
Perforate liner in last cell	March 2017
Back-fill Ponds with native material	March to May 2017

Expected duration for decommissioning of the BTPS for Alternative 1 is 12 to 15 months.

Alternative 2:

A second alternative would employ consecutive draining and drying. This alternative would allow more time for animal species self-relocation. Steps are summarized below:

- Drain and pump Cell A to B.
- Allow Cell A to dry. Perforate liner when bottom sludge is dry enough to support equipment.
- Drain and pump Cell B to Cell C.
- Allow Cell B to dry. Perforate liner when bottom sludge is dry enough to support equipment.
- Pump Cell C to Pond 2.

- Allow Cell C to dry. Perforate liner when bottom sludge is dry enough to support equipment.

Once dry, similar to Alternative 1, the cells would be backfilled with native soil. The total duration of the decommissioning would be 20 months. A proposed schedule for this Alternative is shown in **Table 8-3**.

Table 8-3 Proposed schedule for Alternative 2

Task	Schedule
Empty Cell A	March to May 2016
Dry Cell A	June to July 2016
Empty Cell B	August to October 2016
Dry Cell B	November 2016 to January 2017
Empty Cell C	February to April 2017
Dry Cell C	April to July 2017
Back-fill Ponds with native material	August to October 2017

Recommendation and Estimated Cost:

It is recommended to decommission the BTPS as part of Phase 2b and by the preferred method of Alternative 1 presented earlier. Estimated costs for either alternative are presented in **Table 8-4** based on costs developed in OPCC provided in **Appendix F.**

Table 8-4 BTPS Decommissioning Cost Estimate

Task	Cost of work	Mark-up (10%)	Contingency (20%)	Construction Management (10%)	Total
Rip Liner	$36,000	$3,600	$7,900	$4,800	$52,300
Backfill	$ 740,000	$ 74,000	$ 162,800	$ 97,700	$ 1,074,500
Environmental mitigation	$ 68,000	$ 6,800	$ 15,000	$ 9,000	$ 98,800
Total	**$ 844,000**	**$ 84,400**	**$ 185,700**	**$ 111,500**	**$ 1,225,600**

8.5 CONSTRUCTION COST ESTIMATES

A detailed construction cost estimate is provided in **Appendix F** in 2015 dollars.

8.6 RECOMMENDED CAPITAL IMPROVEMENT PLAN

Based on the phasing described previously, the recommended Capital Improvement Program is presented below in Table 8-5**.** The Class 5 OPCC that was used as a basis for each major line item in the CIP is attached in **Appendix F**.[32]

Anticipated O&M costs including an estimate of Treatment Plant O&M staffing are presented in **Table 8-6**. Note that O&M costs do not include depreciation and are exclusive of Laboratory, Collection System and Administration costs. **Appendix F** includes a detailed listing by category of annual O&M costs, for the current fiscal year and for each phase. All costs are in 2015 dollars.

O&M staffing was estimated as follows: For existing plant processes and existing flow (6.2 mgd), the number of O&M staff for the WRF is 13.[33] Starting from this baseline, staff were added using a formula that takes into account efficiencies due to multiple process trains to handle greater capacity. The formula is $(Qf / 6.2)^{0.6} \times 13 = Sf$ where:

Qf = future plant capacity
Sf = future staffing

For tertiary processes not currently in place, two additional staff were added for Phase 3 and 4. Three additional staff were added for Build-out.

[32] The basis for costs in Appendix F were in some cases modified. Table 8-6 presents line items that may differ due to quantities of work to be done from the items presented in Appendix F.
[33] Collection system, administration and laboratory staff are not included in the WRF O&M staff count.

Table 8-5 CIP

Phase	Construction (starting year)	Upgrade Treatment Capacity (MGD)	Process/Facility	Total Construction Cost ($)	20% Design and Scope Contingency	15% Eng. And Admin	10% Constr. Mgmt. ($)	Rounded Total Cost ($)
2.b	2016	6	Bar Screens	$595,039	$119,008	$107,107	$71,404.68	$893,000
			Grit Removal & Demolition	$1,346,868	$269,374	$242,436	$161,624.16	$2,020,000
			Biofilters for Grit Chambers	$971,783	$194,357	$174,921	$116,614	$1,458,000
			Digester & Related Systems	$8,213,864	$1,642,773	$1,478,496	$985,664	$12,321,000
			Sludge Holding Tank	$1,685,113	$337,023	$303,320	$202,214	$2,528,000
			Thickeners Building	$3,505,095	$701,019	$630,917	$420,611	$5,258,000
			Biofilter for TWAS	$1,082,967	$216,593	$194,934	$129,956	$1,624,000
			BTPS / Wetland Decommissioning	$928,400	$185,680	$0	$111,408	$1,225,000
			Sub-total	**$18,329,129**	**$3,665,826**	**$3,132,131**	**$2,199,495**	**$27,327,000**
2.c	2017	8.2	Sludge Drying Bed Extension	$1,400,336	$280,067	$252,060	$168,040	$2,101,000
			Gas Storage	$1,000,000	$200,000	$180,000	$120,000	$1,500,000
			Pond System Decommissioning	$8,071,672	$1,614,334	$1,452,901	$968,601	$12,108,000
			Sub-total	**$10,472,008**	**$2,094,402**	**$1,884,961**	**$1,256,641**	**$15,708,000**
3	2027	10	Bar Screens	$1,190,078	$238,016	$214,214	$142,809	$1,785,000
			Primary Clarifiers	$3,005,937	$601,187	$541,069	$360,712	$4,509,000
			Chlorine Contact Tank	$6,332,515	$1,266,503	$1,139,853	$759,902	$9,499,000
			Cloth Media Filters	$4,628,750	$925,750	$833,175	$555,450	$6,943,000
			Filter EQ Basin	$2,900,940	$580,188	$522,169	$348,113	$4,351,000
			Title 22 Final Storage	$3,981,296	$796,259	$716,633	$477,756	$5,972,000
			Recycled Water PS	$5,223,260	$1,044,652	$940,187	$626,791	$7,835,000
			Belt Press	$2,593,183	$518,637	$466,773	$311,182	$3,890,000
			Maintenance Building	$2,514,832	$502,966	$452,670	$301,780	$3,772,000
			Operations Building	$3,920,519	$784,104	$705,693	$470,462	$5,881,000
			Sub-total	**$35,101,232**	**$7,020,246**	**$6,318,222**	**$4,212,148**	**$52,652,000**
4	2045	13.3	Grit Removal & Demolition	$596,380	$119,276	$107,348	$71,566	$895,000
			Aeration and Anoxic Basins - new	$15,447,113	$3,089,423	$2,780,480	$1,853,654	$23,171,000
			Aeration Blowers - new	$5,263,508	$1,052,702	$947,431	$631,621	$7,895,000
			Secondary Clarifiers	$2,328,681	$465,736	$419,163	$279,442	$3,493,000
			Chlorine Contact Tank	$6,796,307	$1,359,261	$1,223,335	$815,557	$10,194,000
			Cloth Media Filters	$1,854,581	$370,916	$333,825	$222,550	$2,782,000
			Recycled Water PS	$896,545	$179,309	$161,378	$107,585	$1,345,000
			Digester & Related Systems	$7,585,792	$1,517,158	$1,365,443	$910,295	$11,379,000
			Sludge Holding Tank	$1,492,437	$298,487	$268,639	$179,092	$2,239,000
			Thickeners building	$1,327,098.7	$265,420	$238,878	$159,252	$1,991,000
			Belt Presses	$1,057,607	$211,521	$190,369	$126,913	$1,586,000
			Sludge Drying Bed	$1,172,800	$234,560	$211,104	$140,736	$1,759,000
			Electrical Building	$2,133,824	$426,765	$384,088	$256,059	$3,201,000
			Sub-total	**$47,952,674**	**$9,590,535**	**$8,631,481**	**$5,754,321**	**$71,929,000**

Phase	Construction (starting year)	Upgrade Treatment Capacity (MGD)	Process/Facility	Total Construction Cost ($)	20% Design and Scope Contingency	15% Eng. And Admin	10% Constr. Mgmt. ($)	Rounded Total Cost ($)
Ultimate	Build-Out	20	Influent PS	$777,750	$155,550	$139,995	$93,330	$1,167,000
			Primary Clarifiers	$2,983,059	$596,612	$536,951	$357,967	$4,475,000
			Aeration and Anoxic Basins	$8,214,554	$1,642,911	$1,478,620	$985,746	$12,322,000
			Aeration and Anoxic Blowers	$1,585,881	$317,176	$285,459	$190,306	$2,379,000
			Secondary Clarifiers	$3,837,252	$767,450	$690,705	$460,470	$5,756,000
			Cloth Media Filters	$3,000,000	$600,000	$540,000	$360,000	$4,500,000
			Recycled Water PS	$2,190,324	$438,065	$394,258	$262,839	$3,285,000
			Digester & Related Systems	$5,827,903	$1,165,581	$1,049,023	$699,348	$8,742,000
			Belt Presses	$1,057,607	$211,521	$190,369	$126,913	$1,586,000
			Sludge Drying Bed	$2,275,670	$455,134	$409,621	$273,080	$3,414,000
			Sub-total	**$31,750,000**	**$6,350,000**	**$5,715,000**	**$3,810,000**	**$47,625,000**
TOTAL				**$143,605,043**	**$28,721,009**	**$25,681,796**	**$17,232,605**	**$215,241,000**

Table 8-6 Summary of Capital Improvement Program Capital Cost

Phase	Constr. Year	Flow Capacity, mgd	Project Cost – WRF only (2015 $M)	Project Cost – Tertiary only (2015 $M)	Total Capital Cost (2015 $M)
Current	--	6.2	--	--	--
2b	2016	6.2	$27.0	--	$27.0
2c	2017	8.2	$15.7	--	$15.7
3	2027	10.0	$18.0	$34.6	$52.6
4	2045	13.3	$57.6	$14.3	$71.9
Build-out	--	20.0	$39.8	$7.8	$47.6

Table 8-7 Summary of Capital Improvement Program O&M Costs w/o Tertiary

Phase	Constr. Year	Flow Capacity, mgd	O&M Staffing – no tertiary	Annual O&M Cost (2015 $M)	Annual O&M per MG (2015 $)
Current	--	6.2	13	$3.22	$1,420
2b	2016	6.2	13	$3.22	$1,420
2c	2017	8.2	15	$3.99	$1,330
3	2027	10.0	17	$4.73	$1,295
4	2045	13.3	21	$6.01	$1,220
Build-out	--	20.0	26	$8.33	$1,141

Table 8-8 Summary of Capital Improvement Program O&M costs with Tertiary

Phase	Constr. Year	Flow Capacity, mgd	O&M Staffing – with tertiary	Annual O&M Cost (2015 $M)	Annual O&M per MG (2015 $)
Current	--	6.2	13	$3.22	$1,420
2b	2016	6.2	13	$3.22	$1,420
2c	2017	8.2	15	$4.09	$1,370
3	2027	10.0	19	$5.57	$1,530
4	2045	13.3	23	$7.26	$1,500
Build-out	--	20.0	29	$10.0	$1,370

Section 9 – References

Section 9 References

A Plain English Guide to the EPA Part 503 Biosolids Rule, U.S. Environmental Protection Agency, September 1994, pg. 123.

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Indio Water Authority, *Recycled Water Feasibility Study*, 90% Draft Report, Carollo, January 2015 (Carollo, 2015)

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Jurisdictional Delineation with Least Environmentally Damaging Practical Alternative for the Valley Sanitary District Facility, prepared by L&L Environmental, Inc., May 2015

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Wastewater Engineering Treatment and Reuse, George Tchobanoglous, et. al. (Metcalf & Eddy), 2012

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