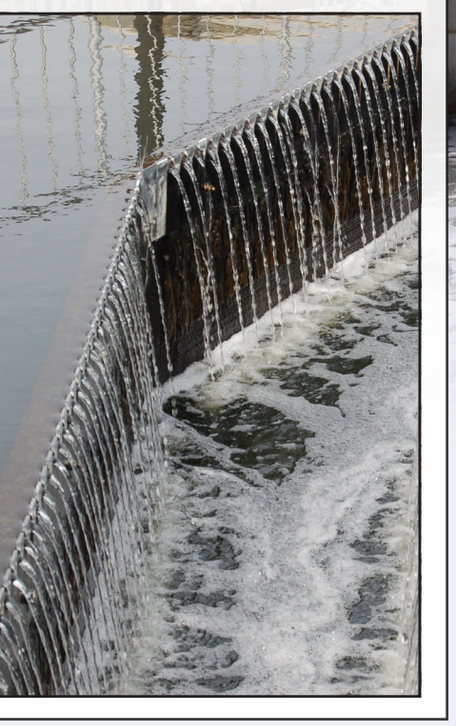








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FINAL

DUBLIN SAN RAMON SERVICES DISTRICT

Wastewater Treatment and Biosolids Facilities Master Plan

WEST YOST ASSOCIATES

in association with

HDR

Wastewater Treatment and Biosolids Facilities Master Plan

Prepared for

Dublin San Ramon Services District

Project No. 406-19-59-39



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List of Abbreviations

µg/L	Micrograms Per Liter
40 CFR 503	Title 40 of the Code of Federal Regulations, Part 503
AACE	Association for the Advancement of Cost Engineering
AB 32	California's Assembly Bill Number 32
ADWF	Average Dry Weather Flows
AHU	Air Handling Unit
Air Permit	Permit to Operate, Permit A1371, Plant #1371
AOP	Advanced Oxidation Process
ATCMs	Airborne Toxic Control Measures
BAAQMD	Bay Area Air Quality Management
BABC	Bay Area Biosolids Coalition
BAC	Biologically Active Carbon
BACT	Best Available Control Technology
BACWA	Bay Area Clean Water Agencies
Basin Plan	San Francisco Bay Basin (Region 2) Water Quality Control Plan
bhp	Brake Horsepower
BOD	Biochemical Oxygen Demand
BTU/cu-ft	British Thermal Units Per Cubic Foot
BTU/hr	British Thermal Units Per Hour
CAA	Clean Air Act
CARB	California Air Resources Board
CBOD	Carbonaceous Biochemical Oxygen Demand
CCT	Chlorine Contact Tank
CDE	Carbon Dioxide Equivalent
CECs	Contaminants of Emerging Concerns
cfm	Cubic Feet Per Minute
cfm/ ft of wall length	cfm per foot of Grit Tank Wall Length

CI	Compression-Ignited (Diesel)
CIMIS	California Irrigation Management Information System
CIPP	Cured-In-Place Pipe
CO	Carbon Monoxide
COD	Chemical Oxygen Demand
cu-ft	Cubic Foot
CWR	Clean Water Revival
DAFT	Dissolved Air Flotation Thickener
DDT	Dichlorodiphenyltrichloroethane
DDW	State Water Board Division of Drinking Water
DERWA	DSRSD-East Bay Municipal Utilities District Recycled Water Authority
DG	Digester Gas
District	Dublin San Ramon Services District
DLD	Dedicated Land Disposal
DPR	Direct Potable Reuse
dt/ac-yr	Dry Ton Per Acre Per Year
dt/yr	Dry Tons Per Year
EBDA	East Bay Dischargers Authority
EBMUD	East Bay Municipal Utilities District
EBRT	Empty Bed Resident Time
FAT	Full Advanced Treatment
FOG	Fats, Oils and Grease
fps	Feet Per Second
FSL	Facultative Sludge Lagoons
ft	Feet
ft/sec-ft	Feet Per Second Per Foot
FY	Fiscal Year
g/bhp-hr	Grams Per Break Horsepower Per Hour
GAC	Granular Activated Carbon
gal/day	Gallons Per Day
GBT	Gravity Belt Thickener
General Order	Order 96-011, General Water Reuse Requirements for Municipal Wastewater and Water Agencies
GHG	Greenhouse Gas
GMF/UV	Granular Media Filtration with UV Disinfection
GOX	Gaseous Oxygen
gpd	Gallons Per Day
gpd/sq-ft	Gallons Per Day Per Square Foot
gph	Gallons Per Hour
gpm	Gallons Per Minute
gpm/sq ft	Gallons Per Minute Per Square Foot

H_2S	Hydrogen Sulfide
HAPs	Hazardous Air Pollutants
HP	Horsepower
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilation and Air Conditioning
I&I	Infiltration and Inflow
IC	Internal Combustion
IPR	Indirect Potable Reuse
kW	Kilowatt
kWh	Kilowatt-Hour
LAVWMA	Livermore-Amador Valley Water Management Agency
lb TSS/Day	Pounds of TSS per Day
lb VSS/ 1,000 cu-ft-day	Pounds of VSS per 1,000 Cubic Feet Per Day
lb VSS/ 1,000 sq-ft-day	Pounds of VSS per 1,000 Square Feet Per Day
lb VSS/day	Pounds of VSS Per Day
lb/cu-ft-year	Pounds Per Cubic Feet Per Year
lb/hr	Pounds Per Hour
lb/sq-ft-hr	Pounds Per Square Feet Per Hour
lbs	Pounds
lbs/day	Pounds Per Day
Lower SF Bay	Lower San Francisco Bay
LOX	Liquid Oxygen
MABR	Membrane Aerated Biofilm Reactor
Master Plan	Wastewater Treatment Plant and Biosolids Facilities Master Plan
MBR	Membrane Bioreactor
MCL	Maximum Contaminant Levels
MCRT	Mean Cell Residence Time
MF	Microfiltration
MF/UV	Microfiltration with UV Disinfection
mg/kg	Milligrams Per Kilograms
mg/L	Milligrams Per Liter
Mgal	Million Gallons
mgd	Million Gallons Per Day
mg-min/L	Milligram Minutes Per Liter
Micron	Micrometer
mJ/cm2	Millijoule Per Square Centimeter
mL/g	Milliliters Per Gram
MLE	Modified Ludzack-Ettinger
MLSS	Mixed Liquor Suspended Solids
MMBTU	Million British Thermal Units

MPN/100mL	Most Probable Number of Coliform Per 100 Milliliter
MRP	Monitoring and Reporting Program
NDN	Nitrification/Denitrification
NG	Natural Gas
NO_2	Nitrogen Dioxide
NOAA	National Oceanic and Atmospheric Administration
NOB	Nirite-Oxidizing Bacteria
NO_X	Nitrogen Oxide
NPDES	National Pollutant Discharge Elimination System
NPV	Net Present Value
NSCR	Non-Selective Catalytic Reduction
NTU	Nephelometric Turbidity Units
O&M	Operation and Maintenance
OPCC	Opinion of Probable Construction Cost
ORT	Odor Reduction Tower
PCBs	Polychlorinated Biphenyls
PG&E	Pacific Gas and Electric
PHDWF	Peak Hour Dry Weather Flow
PHWWF	Peak Hour Wet Weather Flow
POTWS	Public Owned Treatment Works
pounds/ac-day	Pounds Per Acre Per Year
ppb	Parts Per Billion
ppm	Parts Per Million
ppmvd	ppm by Bolume, Dry Basis
ppt	Part Per Trillion
PQL	Practical Quantitation Limit
psi	Pounds Per Square Inch
psig	Pounds Per Square Inch Gage
PSRP	Process to Significantly Reduce Pathogens
PWWF	Peak Wet Weather Flow
RAS	Return Activated Sludge
Regional Board	San Francisco Bay Regional Water Quality Control Board
RMP	San Francisco Estuary Institute's Regional Monitoring Program
RO	Reverse Osmosis
rpm	Revolutions Per Minute
SB 918	California Senate Bill 918
SCADA	Supervisory Control and Data Acquisition
SCFI	Supercritical Fluids International
sec	Second
SIP	Policy for Implementation of Toxics Standards for Inland Surface Water, Enclosed Bays, and Estuaries of California 2000

SO_2	Sulfur Dioxide
SPA	State Point Analysis
sq-ft	Square Foot
SRT	Solids Retention Time
SRVRWP	San Ramon Valley Recycled Water Program
State Water Board	State Water Resources Control Board
SVI	Sludge Volume Index
TACs	Toxic Air Contaminants
TEQ	Toxicity Equivalency
Title 22	California Code of Regulations Title 22, Division 4, Chapter 3
Title 27	California Code of Regulations Title 27
TKN	Total Kjeldahl Nitrogen
TMDL	Total Maximum Daily Load
TOC	Total Organic Carbon
TSS	Total Suspended Solids
UF	Ultrafiltration
USEPA	United States Environmental Protection Agency
UV	Ultraviolet
UV/AOP	Combined UV and Advanced Oxidation Process
UVT	UV Transmittance
VS	Volatile Solids
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
Work Program	10-Point Climate Action Work Program, BAAQMD, 2014
WQOs	Water Quality Objectives
WSEL	Water Surface Elevation
WTP	Water Treatment Plant
WWOP	Wet Weather Facilities Operation Plan
WWTP	DSRSD Wastewater Treatment and Biosolids Facility
Zone 7	Alameda County Flood Control and Water Conservation District Zone 7

CHAPTER 1
Introduction

This Wastewater Treatment Plant and Biosolids Facilities Master Plan (Wastewater Treatment Plant Master Plan or Master Plan) provides a 20-year vision for the Dublin San Ramon Services District's (DSRSD, or District) Wastewater Treatment Plant (WWTP) that considers planned District growth, anticipated and potential regulatory changes, and new opportunities for wastewater resources recovery. This chapter presents an overview of the WWTP planning history, a description of the current WWTP facilities, and an outline of this Master Plan report.

1.1 WWTP PLANNING HISTORY

The District completed a comprehensive WWTP master planning effort in 1997 that laid the groundwork for the WWTP Stage 4 Improvement Project, which was subsequently completed in 2003. In 2005[1] and 2007[2], the District completed studies that reviewed the findings from the 1998 Master Plan and provided updated recommendations. However, these efforts did not revisit the overall vision for the WWTP that was presented in the 1997 work. The District also completed two separate biosolids-focused master planning efforts in 2002[3] and 2007[4].

The purpose of this Master Plan is to update the 20-year old vision for the WWTP, taking into account new information on wastewater flows, new emerging technologies for resources recovery, and potential new regulatory changes with respect to nutrient removal. The specific topics addressed in this Master Plan include the following:

- An evaluation of the current and 2035 projected flows and loads, and an assessment of current WWTP facilities and their capacity to accommodate these conditions
- An evaluation of alternatives for meeting identified liquid treatment process capacity deficiencies
- An assessment of the facilities that may be needed to address potential future nutrient removal requirements, including an assessment of the potential for nutrient removal changes to impact decisions made in the near-term
- An evaluation of alternatives for addressing biosolids treatment and disposal process capacity deficiencies
- An assessment of how a potential supercritical oxidation process constructed at the WWTP could impact long-term biosolids needs
- An evaluation of the potential treatment facilities that could be constructed at the WWTP to provide for potable reuse of the WWTP effluent
- An assessment of the current and anticipated energy demands at the WWTP, and presentation of a step-wise approach to increasing onsite energy generation through organics co-digestion

[1] *Wastewater Treatment Plant Master Plan Update*, Whitley Burchett & Associates
[2] *Wastewater Treatment Plant Expansion Planning Report*, Whitley Burchett & Associates
[3] *Phase 1 Biosolids Mater Plan*, CH_2MHill
[4] *Phase II Biosolids Master Plan*, Whitley Burchett & Associates

- Presentation of a comprehensive 20-year plan for the WWTP, including recommendations for the evaluation efforts that will be needed to further clarify the WWTP vision as new information becomes available

1.2 WWTP FACILITIES OVERVIEW

The WWTP serves approximately 143,000 people from the Cities of Dublin and Pleasanton, and a small, southern portion of the City of San Ramon. The WWTP comprises two different sites: the main WWTP site and a Dedicated Land Disposal (DLD) site. The locations of these two sites are shown on Figure 1-1.

The main WWTP site is an approximately 22-acre area located south of Stoneridge Drive and east of Interstate Highway 680. Most of the WWTP's treatment facilities are located at this site, including all the liquid treatment and recycled water treatment facilities and a portion of the solids treatment facilities. The WWTP's cogeneration facilities are also located at this site.

The DLD site is an approximately 162-acre area located across Stoneridge Drive from the main WWTP site. The DLD site includes six Facultative Sludge Lagoons (FSLs) and the biosolids DLD area. In addition, the Livermore-Amador Valley Water Management Agency (LAVWMA) export facilities are located adjacent to the DLD area of the site.

A simplified schematic of the WWTP facilities is shown on Figure 1-2. The existing WWTP facilities include liquid treatment, solids treatment, recycled water treatment, and energy generation facilities, as follows:

- **Liquid treatment facilities:** influent screenings, aerated grit removal, primary sedimentation basins, activated sludge reactors, secondary clarifiers, and chlorine disinfection, and multiple holding basins for flow equalization.
- **Recycled water treatment facilities:** two separate systems - a large sand filtration system with Ultraviolet light (UV) disinfection, and a microfiltration (MF) with UV disinfection. The MF/UV system are both components of the treatment system that was constructed under the District's Clean Water Revival program.
- **Solids treatment facilities:** a Dissolved Air Flotation Thickener (DAFT), anaerobic digesters, FSLs, and the DLD facility.
- **Energy generation facilities:** a biogas treatment system, cogeneration engines, and hot/cold water loop system to direct captured heat to various buildings on the WWTP site.

Treated secondary effluent from the WWTP is discharged to the LAVWMA export facilities, where it is combined with the City of Livermore's treated effluent and pumped through LAVWMA's pipeline to the East Bay Dischargers Authority (EBDA) Common Outfall in the Lower San Francisco Bay. In addition, the combined LAVWMA flows can be discharged to the Alamo Canal and/or San Lorenzo Creek in the event capacity in the EBDA pipeline is limited during wet weather events.

During the summer months, most the District's wastewater is treated to "Disinfected Tertiary-2.2" recycled water standards and supplied to irrigation customers of the DSRSD-East Bay Municipal Utilities District (EBMUD) Recycled Water Authority (DERWA) and the City of Pleasanton.

Following digestion, the biosolids are re-directed to the FSLs for storage and further stabilization. The digested and stabilized biosolids are applied to DLD area by subsurface injection. This is the final disposal location for this material.

1.3 MASTER PLAN ORGANIZATION

This Master Plan is organized as follows:

Chapter 1 – Introduction: Provides background information on the Master Plan and an overview of WWTP facilities.

Chapter 2 – Wastewater Flow and Loadings: Presents the existing and projected 2035 influent flow and loading conditions for the WWTP.

Chapter 3 – Existing Facilities: Describes the existing facilities at the WWTP and presents an evaluation of the available capacity of the existing facilities as compared to the 2035 flow and loading conditions.

Chapter 4 – Regulatory Requirements: Presents a review of the current and potential future regulatory requirements applicable to the WWTP. The regulations addressed include surface water discharge requirements, recycled water requirements, biosolids surface disposal requirements, and air quality requirements.

Chapter 5 – Evaluation Criteria: Describes the specific methods and assumptions used when assessing life cycle cost and applying subjective criteria in subsequent Master Plan chapters focused on WWTP alternatives.

Chapter 6 – Liquid Treatment Alternatives Evaluation: Presents an evaluation of the facilities required to meet the long-term liquid treatment process and conveyance needs for the WWTP. Also addressed are potential facilities that may be required for future nutrient treatment.

Chapter 7 – Potable Reuse Alternatives Evaluation: Presents an evaluation of the treatment facilities that would be required to provide potable reuse at the WWTP. The Chapter addresses both rehabilitation of the District's Clean Water Revival treatment system, as well as the facilities that would be needed to provide advanced potable reuse treatment of the majority of the available WWTP effluent flows.

Chapter 8 – Solids Treatment Alternatives Evaluation: Presents an evaluation of the facilities required to meet the long-term solids treatment process and conveyance needs for the WWTP. Also presented is an evaluation of the benefits that could be provided by a potential future biosolids supercritical oxidation system that is being considered by the District.

Chapter 9 – Energy Management Plan: Presents the current and anticipated energy demands for the WWTP. Also presented is a strategy for increasing onsite energy generation, with the ultimate goal of achieving a "net zero energy" condition. The initial emphasis for expansion of the onsite energy generation involves increasing the biogas being generated at the WWTP through increased hauled waste organics digestion.

Chapter 10 – Recommended Program: Details the recommended program and facilities needed to comply with existing regulatory requirements and accommodate growth in the District's service area through the year 2035. Strategies for evaluating and implementing future projects that may be needed to address potential future regulatory requirements or take advantage of emerging opportunities are also identified.

1.4 REFERENCES

CH2MHill. (2002). *Phase 1 Biosolids Mater Plan.*

Whitley Burchett & Associates and Brown & Caldwell. (1997). *Wastewater Treatment Plant 1997 Master Plan Update Project CIP 96CH142 Phase 1 Report*.

Whitley Burchett & Associates and Brown & Caldwell. (1998). *Wastewater Treatment Plant 1997 Master Plan Update Project CIP 96CH142 Phase I1 Report.*

Whitley Burchett & Associates. (2005). *2005 Wastewater Treatment Plant Master Plan Update.*

Whitley Burchett & Associates. (2007a). *2007 Wastewater Treatment Plant Expansion Planning Report.*

Whitley Burchett & Associates. (2007b). *Phase II Biosolids Master Plan.*

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Symbology

- Freeways
- Main WWTP Site
- Facultative Sludge Lagoons (FSLs) and Dedicated Land Disposal (DLD) Area
- LAVWMA Export Facilities

N

0 1,250 2,500

Scale in Feet

WEST YOST ASSOCIATES

Figure 1-1

Site Location

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

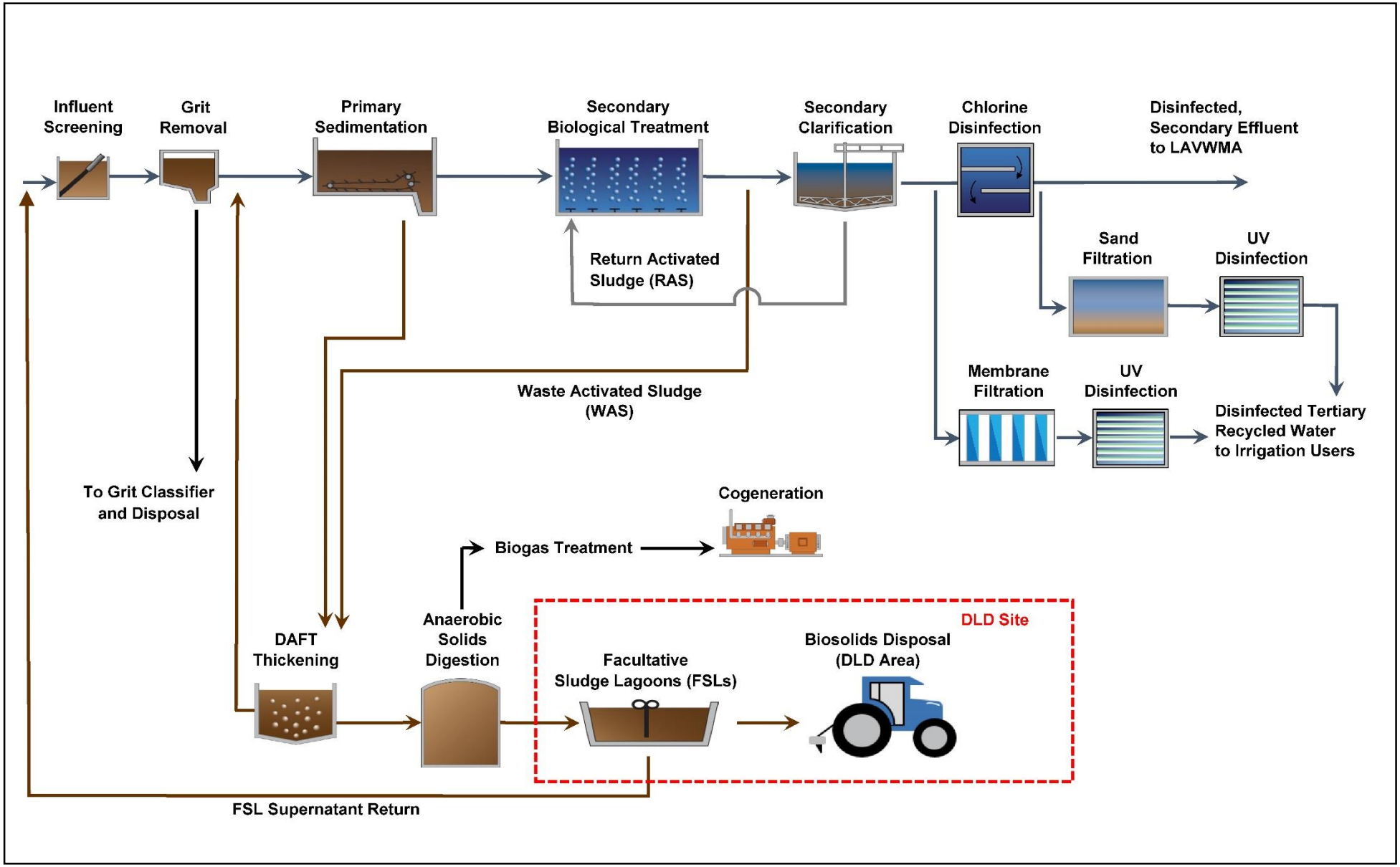


Figure 1-2

Simplified WWTP Schematic

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

CHAPTER 2
Wastewater Flow and Loadings

The purpose of this chapter is to present the existing and projected 2035 influent flow and loading conditions for the District's WWTP. The major topics covered in this chapter include:

- Evaluation of Available Data
- Plant Return Stream Characterization
- Existing Influent Flow and Load Conditions
- Projected Influent Flows and Loads
- Summary of Existing and Projected Influent Conditions

The District's wastewater service area encompasses the entire boundaries of the Cities of Dublin and Pleasanton, and a small, southern portion of the City of San Ramon.

2.1 EVALUATION OF AVAILABLE DATA

This section provides an evaluation of the adequacy of the available influent flow data. Relevant topics include:

- Data Issues
- Future Monitoring Recommendations

2.1.1 Data Issues

Key data issues of concern in this analysis include the following:

1. District staff report that four new influent mag-meters were installed in February 2011. Other key flow metering findings and activities, as communicated by DSRSD staff, include the following:
    a. Around 2009, District staff became aware that the WWTP influent Parshall flume flow meters that are installed in the major trunk lines entering the Plant were not providing reliable readings. The Pleasanton Parshall flume was questionable due to frequent operation under submerged conditions and one of the two parallel Dublin trunk Parshall flume meters produced erratic flow measurements. Due to these issues, influent flow data reported prior to this time are likely an over-projection of actual flows.
    b. Starting in 2009, reported influent flows were estimated from the effluent export flows meters of the LAVWMA, minus the effluent flow from the City of Livermore to LAVWMA and plus DSRSD recycled water production. However, in early 2012, DSRSD staff learned there was also a problem with LAVWMA export flow meters, thus making this method of estimating influent flows unreliable.

c. During the period when influent flows were estimated from the LAVWMA flow meters, the relative contributions of influent flow from the DSRSD member agencies were being estimated using some of the Parshall flume meter data. Specifically, the City of Pleasanton was calculated as the sum of Pleasanton Parshall flume and the East Amador Lift Station mag-meter (although, as noted above, the accuracy of the Pleasanton Parshall flume was questionable due to frequent operation under submerged conditions). The Dublin/San Ramon portion of the influent flow was calculated as the WWTP influent flow (estimated from effluent flow information, as discussed above) less the Pleasanton flow.

d. On January 29, 2010, DSRSD staff completed the addition of a second level sensor in the downstream throat of the Pleasanton Parshall flume. The installation of the downstream sensor included programming so that the meter would automatically switch back and forth between normal flow measurements using a single throat sensor, to surcharged flow measurements using the downstream sensor in conjunction with the throat sensor.

e. In September 2010, a lift station was constructed that enabled flows from the WWTP inner plant sewer to be redirected to discharge into the primary clarifiers (downstream of the influent flow meters). Previously, the inner plant sewer emptied into the headworks, which complicated attempts to measure the WWTP influent flow. However, because the inner plant sewer flows include on-site runoff, any measured peak wet weather flow underestimates the overall flows entering the primary clarifiers and downstream processes.

f. In October 2010, staff disconnected the downstream sensor that was previously installed in the Pleasanton Parshall flume because wave action in the influent wet well appeared to be causing the meter to read incorrectly. Staff suspected this condition may have existed ever since the second sensor was first installed in late January 2010.

g. In January 2011, DSRSD staff lowered the influent wet well level setpoint, which reduced the wave action at the Pleasanton Parshall flume. Shortly afterward, staff replaced the downstream level sensor on the Pleasanton Parshall flume and improved the formula used to calculate flow during submerged conditions.

h. In February 2011, staff began using four new influent mag-meters downstream of the Influent Pump Station to measure the WWTP influent sewage flow. At that time, staff began calculating the influent flow as the sum of the influent mag-meters minus the recycled water filtration backwash and cap water return flows from the District's FSLs. The new mag-meters have proven to be reliable and consistent for measuring the influent flow. The Dublin San Ramon flow has continued to be calculated as the WWTP influent flow less the Pleasanton flow due to continued erratic readings produced by the West Dublin trunk sewer meter.

2. Based on a review of the available flow data, the influent flow data recorded prior to March 2011 are highly erratic and do not appear reliable. Therefore, the flow data used in this analysis is limited to the period of March 2011 through March 2015.
3. District staff have indicated that the influent composite sampler was relocated in May or June 2012. A summary of the influent Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), and Total Suspended Solids (TSS) concentrations data for the period of January 2008 through March 2015 are shown in Figure 2-1. As shown, the recorded monthly average concentrations are highly erratic between January 2008 and May 2012, but largely stabilizes after this period. For that reason, only influent quality data for the period of June 2012 through March 2015 are considered in this analysis.
4. As noted above, the influent flow data reported by the District (and used for the analyses presented in this Chapter) are equal to the flows measured downstream of the Influent Pump Station, minus the flows measured for the backwash and cap water return streams. However, because the influent water quality samples are also collected downstream from where the filtration backwash and FSL cap water are returned, the effect of these return flows on the influent concentrations needs to be considered. Additional analysis of the return streams and their impact on the influent concentrations and flows is presented in the next section of this Chapter.
5. Existing influent ammonia and Total Kjeldahl Nitrogen (TKN) data are quite limited. Specifically, a total of six data values were collected over the period of July 2012 through February 2015. As a result, it is not possible to distinguish monthly average ammonia and TKN results from daily results (as is done with the other parameters of interest), and any conclusions drawn from the use of those data will have limited reliability.

2.1.2 Future Monitoring Recommendations

The following changes to existing monitoring activities are recommended to address some of the data deficiencies identified above:

1. Monitoring of the influent ammonia and TKN should be conducted on a weekly basis.
2. Flows in the lift station associated with the WWTP inner plant sewer should be continuously monitored. Doing so will allow a more reliable calculation of average and peak flows entering the primary clarifiers.
3. The water quality characteristics of the flows in the WWTP inner plant sewer should be monitored for BOD, COD, TSS, ammonia, and TKN on a weekly basis. Doing so will allow a more reliable calculation of primary clarifier performance.
4. The water quality characteristics of each of the major return streams (membrane filter backwash; sand filter backwash; and FSL cap water return) should be monitored for BOD, COD, TSS, ammonia, and TKN on a weekly basis.

2.2 PLANT RETURN STREAMS CHARACTERIZATION

The following three major recycle streams are returned to the WWTP influent: FSL cap water return, sand filter backwash return, and membrane filter reject return. As noted previously, these three return streams have been subtracted from the measured influent flow and are therefore not included in the influent flow data used in the analysis presented in this Chapter. However, it is important to understand the characteristics of these three flows as in the WWTP master planning process. The topics of interest are as follows:

- Return stream flows
- Return stream concentrations
- Effective of return streams on influent concentrations

2.2.1 Return Stream Flows

Monthly average return flow rates for these three streams are shown on Figure 2-2. Summary flow statistics are shown in Table 2-1. As indicated, the aggregate of the three return flow streams averages about 0.5 million gallons per day (mgd).

Table 2-1. Return Stream Flow Summary Statistics, 2011–2015

Statistic	Units	Facultative Sludge Lagoon Supernatant	Sand Filter Reject	Membrane Filter Reject	Aggregate Return Flow
Average	mgd	0.23	0.24	0.05	0.52
Min	mgd	0.00	0.00	0.00	0.00
Max	mgd	1.66	2.90	0.88	3.07
Count	—	1,247	1,247	1,242	—
Count >0	—	553 [(a)]	870	431	—
Proportion >0	Percent	44 [(a)]	70	35	—

[(a)] A threshold of 0.1 mgd is used for the FSL supernatant.

2.2.2 Return Stream Concentrations

BOD and TSS concentration data are available for all three return streams, and ammonia concentration data are also available for the FSL cap water return flow. The sand filter and membrane filter backwash are expected to have minimal ammonia content. Summary statistics for the available concentration data, expressed as milligrams per liter (mg/L), are indicated in Table 2-2. As shown, with the exception of the ammonia levels in the FSL return stream, the constituent concentrations in the return streams are not dissimilar from the influent water quality.

Table 2-2. Return Stream Concentrations Summary Statistics

Statistic	BOD	TSS	Ammonia
FSL Cap Water Return			
Average, mg/L	217	341	239
Max, mg/L	279	420	720
Min, mg/L	160	180	128
Count	6	6	35
Data Period	Dec 2008 – Feb 2009	Dec 2008 – Feb 2009	Feb 2008 – Sep 2009
Sand Filtration Backwash Return			
Average, mg/L	128	318	—
Max, mg/L	249	776	—
Min, mg/L	18	66	—
Count	16	16	0
Data Period	Apr 2014 – Jun 2015	Apr 2014 – Jun 2015	—
Membrane Filtration Reject Return			
Average, mg/L	137	125	—
Max, mg/L	200	220	—
Min, mg/L	53	43	—
Count	10	10	0
Data Period	Mar 2014 – Mar 2015	Mar 2014 – Mar 2015	—

2.2.3 Effect of Plant Return Streams on Influent Concentrations

As noted above, influent flow and water quality data represent the influent flow combined with the return streams from the recycled water filtration systems (i.e. backwash) and the FSLs (cap water return). In addition, although the flow data used in this analysis have the effects of the return streams subtracted out, the influent water quality data is potentially impacted by the effects of WWTP return flows. The effects of those return flows on the reported influent concentrations are considered here. The purpose of this assessment is to determine whether it is appropriate (and/or possible) to account for the impacts of the return stream on the influent concentrations as part of the analysis of the influent water quality data.

In general, the return stream concentration data are sparse, so it is not possible to pair individual data points with influent flow and concentration data. As a result, the average return stream concentrations that were shown in Table 2-2 are used in this analysis. The average was taken over a period of 2012 to 2015, when the District's service area did not experience dramatic changes. Due to the limited available data, the following assumptions have been made:

- The filter backwash ammonia concentrations are assumed to equal the secondary effluent ammonia concentrations.
- COD and TKN have not been measured in any of the return streams, so the average influent BOD/COD and ammonia/TKN ratios from the period of June 2012 through March 2015 are assumed to be applicable to the return streams also.

Using the data available, the raw influent concentrations can be estimated by subtracting out the effects of the three return streams as follows:

$$C_{ADJ} = [C_{INF} * (Q_{INF} + Q_{FSL} + Q_{SF} + Q_{MF}) - (C_{FSL} * Q_{FSL} + C_{SF} * Q_{SF} + C_{MF} * Q_{MF})]/Q_{INF}$$

Where:

C_{ADJ} = The adjusted influent concentration to remove the effects of return streams
C_{INF} = The measured influent concentration that includes the effects of return streams
Q_{INF} = The influent flow that excludes the return streams
C_{FSL} = The average facultative sludge lagoon cap water concentration
Q_{FSL} = The facultative sludge lagoon cap water return flow rate
C_{SF} = The average sand filter backwash concentration
Q_{SF} = The sand filter backwash return flow rate
C_{MF} = The average membrane filter backwash concentration
Q_{MF} = The membrane filter backwash return flow rate

The back-calculated raw influent concentrations are compared to the measured concentrations in Table 2-3.

Table 2-3. Effects of Return Streams on Measured Influent Concentrations

Constituent	Unadjusted Concentration, mg/L	Adjusted Concentration, mg/L	Difference
BOD	240	245	-1.9%
COD	687	699	-1.7%
TSS	261	259	0.8%
Ammonia	37.9	34.5	9.1%
TKN	53.5	48.6	9.1%

As indicated in Table 2-3, subtracting out the effects of the return streams, changes the BOD, COD, and TSS concentrations by less than 2 percent, which is considered inconsequential for this analysis. As a result, no adjustments are applied to the influent BOD, COD, and TSS concentrations are needed to account for return streams for the analyses presented in this Chapter.

By contrast, Table 2-3 shows that the ammonia and TKN values decrease by almost 10 percent as a result of the adjustment, where this decrease is largely attributable to the effects of the FSL cap water return. However, because there are only six data points each for influent ammonia and TKN, any evaluation of these data involves a very low level of precision. Specifically, the 95th percent confidence interval for the average influent ammonia concentration ranges from 33.8 mg/L to 42.0 mg/L, and the adjusted influent ammonia concentration of 34.5 mg/L falls within this range. Therefore, the adjusted concentration cannot be considered statistically different from the unadjusted concentrations. As a result, adjustments cannot reasonably be applied to the influent ammonia and TKN concentrations.

In summary, the impacts of the three return streams on the measured influent water quality concentrations are either negligible or can't be evaluated given limited data. Therefore, no adjustment was made to the influent water quality data to account for these return flows in the analysis previously presented in this Chapter. As noted previously, additional data collection for the return streams is recommended so that future analyses can better account for the influence of these flows.

2.3 EXISTING INFLUENT FLOW CONDITIONS

The analysis of existing influent flow conditions consists of the following elements:

- Average and Seasonal Flow Variations
- Average Flow Conditions
- 30-Day Peak and Maximum Day Flows
- Diurnal Flow Conditions
- Peaking Factors

2.3.1 Average and Seasonal Flow Variations

In this section, average and seasonal variations of the influent flows are considered. Monthly average flows for the period of record are shown against monthly rainfall totals on Figure 2-3. Average dry weather flows (ADWF), defined as the average flows for the lowest three consecutive dry season months (typically August through October) for the year 2011 through 2014 are indicated on Figure 2-4.

As indicated on the figures, flows have been trending downward throughout the period of record, despite increasing population within the wastewater service area. Two major factors may contribute to the reduction, both related to the ongoing drought conditions. First, water use restrictions and other incentives to reduce water use have contributed to a reduction of flows, especially from residential sources. Second, the lack of rainfall has contributed to declining groundwater levels, which may be resulting in reduced infiltration into the wastewater collection system.

2.3.2 Average Flow Conditions

The ADWF is used as the basis for future projections in this analysis. As indicated on Figure 2-4, and as just discussed, ADWF have been decreasing over the period of record. In this analysis, it is assumed that the average value of the ADWF values from 2012 through 2014 represent current conditions. The logic of this assumption is first that this period coincides with the period for which reliable flow data exist (as discussed above), and second that during the current drought, water usage and wastewater generation may be lower than could be expected for more normal rainfall years. Therefore, use of the most recent, single year of flow record would not be representative of "normal" conditions. The calculated ADWF and annual average flow that represents the existing influent conditions are shown in Table 2-4.

Table 2-4. Existing Influent Conditions, 2012 through 2014

Statistic	Flow, mgd
Average Dry Weather	9.7
Average	10.3

2.3.3 30-Day Peak and Maximum Day Flows

30-day peak and maximum day flows are derived from the 30-day average and maximum daily flow values for the available period of record. The results and the associated peaking factors are indicated in Table 2-5. Flow peaking factors are defined as the given flow parameter (e.g., maximum day flow) divided by the existing ADWF value of 9.7 mgd.

Table 2-5. Existing 30-Day Peak and Maximum Day Flows, June 2012 through March 2015

Statistic	Flow, mgd	Peaking Factor
Average Dry Weather Flow	9.7	1.0
30-Day Peak Flow	12.3	1.3
Maximum Day Flow	20.1	2.1

2.3.4 Diurnal Flow Conditions

Key topics of interest with regard to diurnal flow conditions include:

- Dry Weather Diurnal Flow Patterns
- Peak Hour Dry Weather Flow
- Peak Hour Wet Weather Flow

2.3.4.1 Dry Weather Diurnal Flow Patterns

Weekday and weekend dry weather diurnal flow curves are indicated on Figure 2-5. These dry weather flow patterns are based on hourly data from September 2014. The curves are normalized by dividing each value by the average flow for the month. As indicated, the peak diurnal flow condition occurs on the weekend, where a diurnal peaking factor of just under 1.6 is indicated.

2.3.4.2 Peak Hour Dry Weather Flow

For this analysis, hourly dry season flow data was not available for an entire dry season. Therefore, peak hour dry weather flow (PHDWF) was estimated as follows:

1. For the three dry seasons for which reliable flow data are available (2012–2014), calculate daily flow factors for each day of the season by taking the daily flow value and dividing by the ADWF for that season.
2. Assume that the diurnal peaking factors shown on Figure 2-5 applies throughout the season, according to the appropriate day type (weekday or weekend).
3. Calculate the resulting PHDWF factor by multiplying the corresponding daily flow factor with the corresponding diurnal peaking factor (either the weekday or weekend factor).

This approach is generally conservative because on dry days with higher overall flows, it is likely that the diurnal peaking factors do not remain proportional. Nevertheless, using this approach, the synthesized peak hour flow factors for 2013 (the year in which the highest such factors occurred) are indicated on Figure 2-6. As indicated, the highest such calculated factor was just over 2.0. However, it is possible that this one value is anomalous. The use of the 99th percentile value for the three-year period, rather than the maximum overall value, is more insulated from the possibility of outlier values, and is therefore preferred. As a result, the 99th percentile value of 1.9 is assumed to representative of the PHDWF. Applying this factor to the existing ADWF of 9.7 mgd yields an existing PHDWF value of 18.4 mgd.

2.3.4.3 Peak Hour Wet Weather Flow

The analysis of peak hour wet weather flow (PHWWF) is limited to the period of record for reliable flow data, which encompasses the period of March 2011 through March 2015, as noted previously. Based on daily rainfall data collected at the Dublin-San Ramon Fire House rainfall gauges, the largest rainfall events for the period in question include:

- March 22–25, 2011
- January 20–21, 2012
- November 30–December 2, 2012
- December 21–24, 2012
- December 2–3, 2014
- December 11–12, 2014
- February 6–8, 2015

Peak flow, peak infiltration and inflow (I&I), and peaking factor statistics were obtained for each of these rain events as follows:

1. For each event, hourly average flows were calculated from 5-minute influent flow data.
2. For each storm, an underlying dry weather diurnal flow pattern was established by multiplying the hourly diurnal flow factors indicated in Figure 2-5 by the ADWF value for the applicable year.
3. The I&I was obtained by superimposing the appropriate diurnal flow curve (weekday or weekend), and subtracting the results from the hourly average flows for that storm event.
4. The theoretical PHWWF was obtained by taking the calculated peak I&I value and shifting the timing of that peak to coincide with the weekend dry weather diurnal flow peak.
5. Peaking factors were obtained by dividing the actual PHWWF and the theoretical PHWWF by the ADWF value for the period of 2012–2014.

The use of the theoretical PHWWF and associated peaking factor may appear somewhat conservative due to the relatively low probability that a major storm event peak would coincide with the diurnal peak flow on a weekend. However, any such conservatism is likely to be offset by the relatively limited period of record for which reliable flow data exist.

Key statistics for each of the storms under consideration are indicated in Table 2-6. As shown, the highest theoretical peaking factors are associated with the November 30–December 2, 2012 and December 11–12, 2014 wet weather events. Storm event flows, I&I, rainfall, and underlying diurnal base flows are shown for the seven events listed in Table 2-6 are provided in **Appendix A**.

Table 2-6. Peak Wet Weather Events, 2012–2015

Storm Event	ADWF Year	Actual PHWWF, mgd	Peaking Factor	Peak I&I, mgd	Theoretical PHWWF, mgd	Theoretical Peaking Factor
March 22–25, 2011	2011	31.6	3.3	19.5	34.4	3.5
January 20–21, 2012	2011	19.1	2.0	7.1	22.1	2.3
November 30 – December 2, 2012	2012	36.2	3.7	21.6	36.6	***3.8***
December 21–24, 2012	2012	23.9	2.5	11.0	26.0	2.7
December 2–3, 2014	2014	23.8	2.5	13.1	28.1	2.9
December 11–12, 2014	2014	31.0	3.2	20.7	35.7	***3.7***
February 6–8, 2015	2014	27.2	2.8	12.8	27.8	2.9

Key rainfall data for the storm events in question are summarized in Table 2-7. As shown, the highest rainfall event totals (6-hour, 12-hour, 24-hour, and 48-hour rainfall totals) generally occurred during the December 11–12, 2014 event, with the November 30–December 2, 2012 event generally being second. Antecedent rainfall conditions (previous week, previous 30 days, season to date rainfall) do not appear to be a major factor.

Table 2-7. Peak Wet Weather Event Rainfall Statistics, 2012–2015

Storm Event	Rainfall, inches						
	6-Hour Max	12-Hour Max	24-Hour Max	48-Hour Max	Previous Week	Previous 30-Days	Season To Date
March 22–25, 2011	1.13	1.43	1.53	1.80	1.66	3.67	15.02
January 20–21, 2012	1.12	1.30	1.40	1.45	0.03	0.03	2.21
November 30 – December 2, 2012	1.41	2.04	2.21	2.52	0.88	2.47	3.19
December 21–24, 2012	0.65	0.72	0.92	1.50	3.40	4.65	5.71
December 2–3, 2014	0.93	1.15	1.76	2.49	0.73	1.65	1.92
December 11–12, 2014	1.38	2.09	2.84	2.87	1.84	2.76	3.03
February 6–8, 2015	0.72	0.85	1.17	1.83	0.30	0.30	9.82

Source: California Data Exchange, Dublin-San Ramon Fire House gauge.

To assess the magnitude of the storms in question, rainfall statistics for various rainfall periods ranging from 6 hours to 48 hours were compared to storm return period information for the immediate area, as maintained by the National Oceanic and Atmospheric Administration (NOAA) at the Dublin Blvd. weather station. The expected rainfall totals for various durations and return periods are summarized in Table 2-8. Given these results, the two largest storm events in question do not appear to be greater than 3-year events for the 12-hour and 24-hour averaging periods.

Table 2-8. NOAA Rainfall Event Return Periods, Dublin Blvd. Station

Expected Rainfall Totals, inches				
	Event Duration			
Return Period, years	6-Hour	12-Hour	24-Hour	2-Day
1	1.2	1.6	2.1	2.6
2	1.5	2.0	2.8	3.5
5	1.8	2.6	3.6	4.5
10	2.1	3.0	4.3	5.3
25	2.6	3.6	5.1	6.4

It should be noted that rainfall variability with a service area as large as the District's wastewater service area may be high. Therefore, it remains uncertain whether the statistics in Table 2-7 are truly representative of the wastewater service area as a whole. Rainfall event statistics associated with the ten largest two-day rainfall totals for the period of 2004–2015, as reported at a gauge located in Pleasanton, are shown in Table 2-9. (This gauge has a longer period of record than the Dublin gauge, which starts in late 2008; however, the Pleasanton gauge only provides daily rainfall data, not hourly data.) As indicated in Table 2-9, since 2004, only two storms have had a greater two-day rainfall than the December 11–12, 2014 storm, and one of those (the October 2009 storm) had no antecedent rainfall at all, while the other (the December 2005) storm had comparable previous 30-day and season to date rainfall. These data give no indication that any storms since 2004 would have produced significantly more runoff than the two largest flow-producing storms indicated in Table 2-6, above.

Table 2-9. Peak Wet Weather Event Rainfall Statistics, 2004–2015

	Rainfall, inches			
Storm Event Start	2-Day Total	Previous Week	Previous 30-Days	Season to Date
December 30, 2004	2.19	1.49	3.01	8.03
December 30, 2005	3.58	1.43	4.44	4.98
January 4, 2008	3.41	0.53	2.59	4.25
October 13, 2009	3.88	0.00	0.00	0.00
January 19, 2010	2.43	1.05	1.60	8.07
March 23, 2011	2.01	2.08	4.60	16.70
November 30, 2012	2.98	0.78	2.24	2.78
December 2, 2014	3.02	0.61	1.93	2.23
December 11, 2014	3.48	0.08	5.00	5.33
February 7, 2015	2.22	1.21	1.22	12.06

Source: California Irrigation Management Information System (CIMIS) gauge #191, Pleasanton

Given the results in Table 2-6 and the review of available rainfall data for the period of reliable influent flow data presented above, a PHWWF peaking factor of 3.8 is assumed to be reasonably representative of existing and future peak flow conditions. Applying this factor to the existing ADWF of 9.7 mgd yields an existing PHWWF value of 36.6 mgd.

2.3.5 Summary of Influent Flows and Peaking Factors

A summary of the calculated influent flow peaking factors is provided in Table 2-10.

Table 2-10. Existing Influent Flows and Flow Peaking Factors

Statistic	Flow, mgd	Peaking Factor
Average Dry Weather Flow	9.7	1.0
Year-Round Average Flow	10.3	1.1
30-Day Peak Flow	12.3	1.3
Max Day Flow	20.1	2.1
PHDWF	18.4	1.9
PHWWF	36.6	3.8

2.4 EXISTING INFLUENT LOAD CONDITIONS

The analysis of existing influent load conditions consists of the following elements:

- Average and Seasonal Flow Variations
- Average Flow Conditions
- Diurnal and Peak Flow Conditions
- Peaking Factors

2.4.1 Average and Seasonal Variation

Monthly average BOD, COD, and TSS loads, expressed as pounds per day (lbs/day), are indicated on Figure 2-7 for the period of record. Twelve-month running averages for the BOD, COD, and TSS loads are shown on Figure 2-8 These figures indicate that loadings gradually increased from May 2013 to Late 2014, but that a decrease in loads had occurred from late 2014 to March 2015.

Twelve-month running averages for the BOD, COD, and TSS concentrations are shown on Figure 2-9. Comparing Figure 2-9 to Figure 2-8, it is demonstrated that the BOD loads and BOD concentrations exhibit a similar trend, but the decreases in BOD concentrations that occurred in late 2014 are not nearly as significant as the BOD load decreases that occurred during this same period. For COD or TSS, the concentrations do not decrease in late 2014, whereas the COD and TSS loads do.

The information presented in Figure 2-9 to Figure 2-8 supports a conclusion that the recent BOD, COD and TSS loading decreases are not associated with reduced concentrations, but are instead associated with the reduced flows that occurred over this period. It should be noted that if the population had remained constant from late 2014 to March 2015, the decreased flow associated with drought conditions should have resulted in increased influent concentrations. However, the fact that concentrations did not increase over the approximate six-month period when flows decreased is difficult to evaluate given the limited six-month time frame.

2.4.2 Average Load Conditions

Due to the relatively minor changes in loadings that have occurred over the period of record, the dry weather averages for the period of June 2012 to March 2015 are assumed to represent current loading conditions. This period of record also represents the period when reliable influent concentration data is available.

Annual average versus average dry weather concentrations and loads for BOD, COD, TSS, ammonia and TKN are compared in Table 2-11. As shown, the average dry weather and average TKN and ammonia concentrations and loads are shown as the same value. This is because there are only six data points available for the TKN and ammonia data, all of which were collected in the dry period. Therefore, for this analysis, it is assumed that the annual average is equal to the calculated dry weather averages value.

Table 2-11. Existing Influent Conditions, June 2012 through March 2015

Statistic	BOD	COD	TSS	Ammonia	TKN
Concentrations, mg/L					
Average Dry Weather	240	680	263	38	54
Average	239	686	260	38	54
Loads, lbs/day					
Average Dry Weather	19,400	55,000	21,300	3,300	4,600
Average	20,500	58,900	22,300	3,300	4,600

Notable from Table 2-11 is that the ratio of the BOD and COD concentrations is 0.35 for the average dry weather condition and 0.34 for the average condition. The monthly average BOD:COD ratios for the period of June 2012 through March 2015 are shown on Figure 2-10. These ratios are based on a total of three to five data points per month. As indicated on the figure, the BOD:COD ratios range from 0.29 to 0.39. The textbook "Wastewater Engineering, Fourth Edition" [Metcalf and Eddy] states that: "Typical values for the ratio of BOD/COD for untreated municipal wastewater are in the range of 0.3 to 0.8." It thus appears that the BOD:COD ratios of the WWTP influent are at the low end of the typical range.

2.4.3 30-Day Peak and Maximum Day Loads

A summary of the calculated 30-day peak and maximum day loads calculated from the available data are shown in Table 2-12. The maximum day loads shown are calculated as the 99th percentile rather than the maximum (100th percentile). The use of the 99th percentile insulates the maximum day value from more extreme, potential outlier values. In the case of the 30-day peak values, the averaging effect itself tends to insulate the result from extreme values, and therefore, the maximum value is used. Table 2-12 also shows that the 30-day peak loads cannot be calculated for ammonia and TKN due to lack of data. In addition, the maximum day ammonia and TKN load calculations warrant the caveat that the amount of data available probably does not represent the full range of actual loadings.

Table 2-12. Existing 30-Day Peak and Maximum Day Loads

Constituent	30-Day Peak, lbs	Maximum Day, lbs
BOD	25,200	29,100
COD	71,500	82,500
TSS	25,600	32,000
Ammonia	insufficient data	4,200
TKN	insufficient data	6,000

2.4.4 Summary of Influent Loads and Peaking Factors

A summary of the calculated existing influent loads and associated peaking factors is provided in Table 2-13.

Table 2-13. Existing Influent Loads and Load Factors

Constituent	Average Dry Weather	Annual Average	30-Day Peak	Maximum Day
Loads, lbs/day				
BOD	19,400	20,500	25,200	29,100
COD	55,000	58,900	71,500	82,500
TSS	21,300	22,300	25,600	32,000
Ammonia	3,300	3,300	insufficient data	4,200
TKN	4,600	4,600	insufficient data	6,000
Peaking Factors				
BOD	—	1.1	1.3	1.5
COD	—	1.1	1.3	1.5
TSS	—	1.1	1.2	1.5
Ammonia	—	1.0	insufficient data	1.3
TKN	—	1.0	insufficient data	1.3

2.5 PROJECTED INFLUENT FLOWS AND LOADS

This section provides an assessment the influent wastewater flow and loadings anticipated in 2035, which is the ultimate planning horizon for this Master Plan. The major topics covered in this section include:

- Growth Basis
- 2035 Flows and Loads
- Comparison to Stage 4 Improvements Design

2.5.1 Growth Basis

The major geographic entities within the District's wastewater service area are the Cities of Dublin and Pleasanton, and a portion of the City of San Ramon. It is assumed for this analysis that the rates of population growth for these three service areas also apply to wastewater flow and loading growth rates. However, population growth rates for the portion of San Ramon that discharges to the WWTP are undefined. Therefore, it is assumed for this analysis that the combined projected growth rates of Dublin and Pleasanton also apply to the portion of San Ramon that discharges to the WWTP. This approach is reasonable because the area of San Ramon that discharges to the WWTP is a very small portion of the overall flow to the facility. This approach is also slightly conservative because the area of San Ramon that discharges to the WWTP is almost fully grown out and future growth is expected to be limited.

Table 2-14 provides an assessment of the combined population growth rate for Dublin and Pleasanton. As indicated, the population growth projections are currently only available for the Cities of Dublin and Pleasanton through 2030. Therefore, it has been assumed that the rates of growth for the period of 2025–2030 also apply to the period 2030–2035.

Table 2-14. Wastewater Service Area Population Projections through 2035

Year	Dublin Population [a]	Pleasanton Population[b]	Combined Population	Combined Annual Growth Rate
2015	54,468	72,200	126,668	—
2020	59,499	75,600	135,099	1.30%
2025	64,530	78,800	143,330	1.19%
2030	69,561	82,300	151,861	1.16%
2035[c]	74,592	86,300	160,892	1.16%

[a] Population data was provided by the District via email on March 10, 2015. The 2035 population is based on number of persons per household per recent census data, multiplied by the number of households per the City pf Dublin General Plan. Intermediate year population values are based on linear interpolation between 2015 and 2035

[b] 2015 to 2030 population data are presented in the City of Pleasanton's 2010 Urban Water Management Plan, June 2011. 2035 population value is extrapolated values based on the growth rates for the period of 2025–2030.

2.5.2 Projected 2035 Flows and Loads

The ADWF projections through 2035, calculated based on the growth rates show in Table 2-14, are shown on Figure 2-11. The PHWWF projections through 2035 are shown on Figure 2-12. The influent load projections through 2035 for BOD, COD, TSS, ammonia, and TKN are provided in **Appendix B**. The projected 2035 average dry weather, average, 30-day peak, and maximum day flows and loads are shown in Table 2-15.

Table 2-15. Projected 2035 Flows and Loads

Statistic	Flow, mgd	BOD, lbs/day	COD, lbs/day	TSS, lbs/day	Ammonia, lbs/day	TKN, lbs/day
Average Dry Weather	12.3	24,600	69,900	27,100	4,100	5,800
Average	13.5	27,100	76,900	29,800	4,100	5,800
30-Day Peak	16.0	32,000	90,900	32,500	(a)	(a)
Maximum Day	25.8	36,900	104,900	40,700	5,300	7,500
Peak Hour Dry Weather	23.4	(a)	(a)	(a)	(a)	(a)
Peak Hour Wet Weather	46.7	(a)	(a)	(a)	(a)	(a)

(a) Not calculated.

2.5.3 Comparison to Stage 4 Improvements Design

The design flows and loads for the existing WWTP are documented in the DSRSD Wastewater Treatment Plant Stage 4 Improvements Contract Documents, January 2000 (Stage 4 Improvements Design). The projected 2035 flows and loads from the current analysis are compared to those in the Stage 4 Improvements Design in Table 2-16.

Table 2-16. Projected 2035 Influent Conditions versus Stage 4 Design Parameters

Statistic	Units	Projected 2035 Influent Conditions	Stage 4 Improvements Design[(a)]
Influent Flow			
ADWF	mgd	12.3	17.0
PHDWF	mgd	23.4	Not Defined
PHWWF	mgd	46.7	60.7
PHWWF Factor	—	3.8	3.6
Influent BOD			
Avg Dry Weather Concentration	mg/L	240	277
Avg Dry Weather Load	lbs/day	24,600	39,300
30-Day Peak Load	lbs/day	32,000	60,100
30-Day Peak Load Factor	—	1.3	1.5
Max Day Load	lbs/day	36,900	100,100
Max Day Load Factor	—	1.5	2.5
Influent TSS			
Avg Dry Weather Concentration	mg/L	260	279
Avg Dry Weather Load	lbs/day	27,100	39,600
30-Day Peak Load	lbs/day	32,500	62,400
30-Day Peak Load Factor	—	1.2	1.6
Max Day Load	lbs/day	40,700	97,000
Max Day Load Factor	—	1.5	2.4

[(a)] Source: Dublin San Ramon Services District Wastewater Treatment Plant Stage 4 Improvements Contract Drawings, Brown and Caldwell.

As indicated in Table 2-16, the 2035 projected flows and loadings from this analysis are considerably lower than those that served as the basis for the Stage 4 Improvements Design. The ADWF value, for example, is much lower in the current analysis, which is at least partly a reflection of the much slower rate of growth in wastewater flow rates seen within the District's service area (and many other areas throughout the region) compared to earlier assumptions. Another factor in the disparity between the current and previously-projected ADWF may be that the flow metering data that served as the basis for the Stage 4 Improvements Design may have been overestimating actual flows. It should be noted that the PHWWF peaking factor from the current analysis is slightly higher than that used in the Stage 4 Improvements Design, but because the ADWF is so much lower, the projected PHWWF is also considerably lower.

In the case of BOD and TSS loads, the average, 30-day peak, and maximum day loads are all much lower in the current analysis, ranging from 37 to 68 percent of the values in the Stage 4 Improvements Design, and the peaking factors are also lower, especially for the max day condition. It thus appears that the Stage 4 Improvements Design made use of a data set in which a much higher degree of variability existed than the data set used in the current analysis.

2.6 SUMMARY OF EXISTING AND PROJECTED INFLUENT CONDITIONS

The existing and projected 2035 flows and loads, as derived in this chapter, are summarized in Table 2-17.

Table 2-17. Summary of Existing and 2035 Conditions

Parameter	Units	Average Dry Weather	Average	30-Day Peak	Max Day	PHDWF	PHWWF
Existing Conditions							
Flow	mgd	9.7	10.3	12.3	20.1	18.4	36.6
BOD	lbs/day	19,400	21,300	25,200	29,100	[a]	[a]
COD	lbs/day	55,000	60,500	71,500	82,500	[a]	[a]
TSS	lbs/day	21,300	23,400	25,600	32,000	[a]	[a]
Ammonia	lbs/day	3,300	3,300	insufficient data	4,200	[a]	[a]
TKN	lbs/day	4,600	4,600		6,000	[a]	[a]
Projected 2035 Conditions							
Flow	mgd	12.3	13.5	16.0	25.8	23.4	46.7
BOD	lbs/day	24,600	27,100	32,000	36,900	[a]	[a]
COD	lbs/day	69,900	76,900	90,900	104,900	[a]	[a]
TSS	lbs/day	27,100	29,800	32,500	40,700	[a]	[a]
Ammonia	lbs/day	4,100	4,100	insufficient data	5,300	[a]	[a]
TKN	lbs/day	5,800	5,800		7,500	[a]	[a]
Average Concentrations (All Conditions)							
BOD	mg/L	240	250	240	[a]	[a]	[a]
COD	mg/L	680	710	690	[a]	[a]	[a]
TSS	mg/L	260	270	250	[a]	[a]	[a]
Ammonia	mg/L	40	38	insufficient data	[a]	[a]	[a]
TKN	mg/L	57	54		[a]	[a]	[a]

[a] Not calculated.

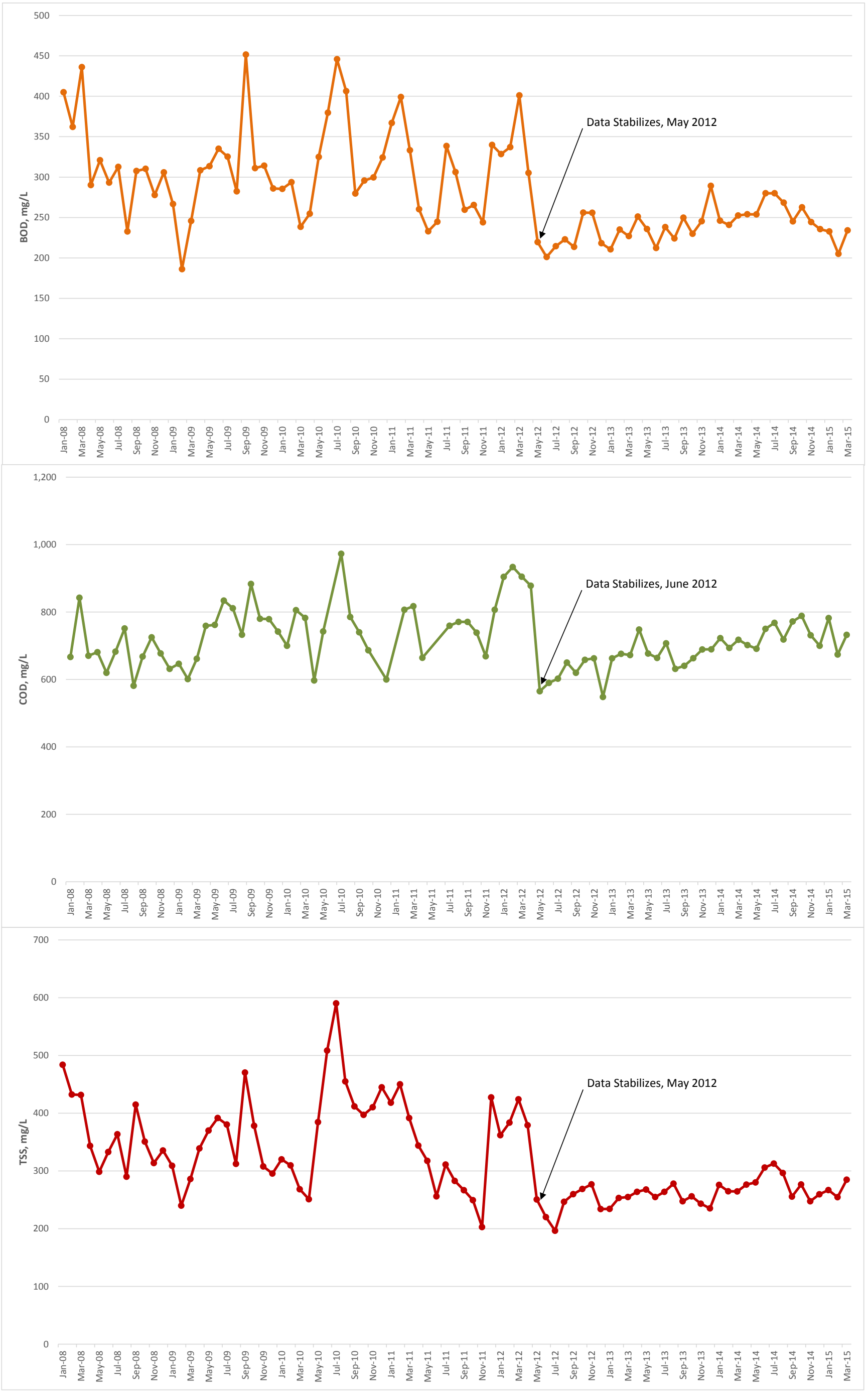


Figure 2-1
Monthly Average Influent Concentrations, 2008–2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

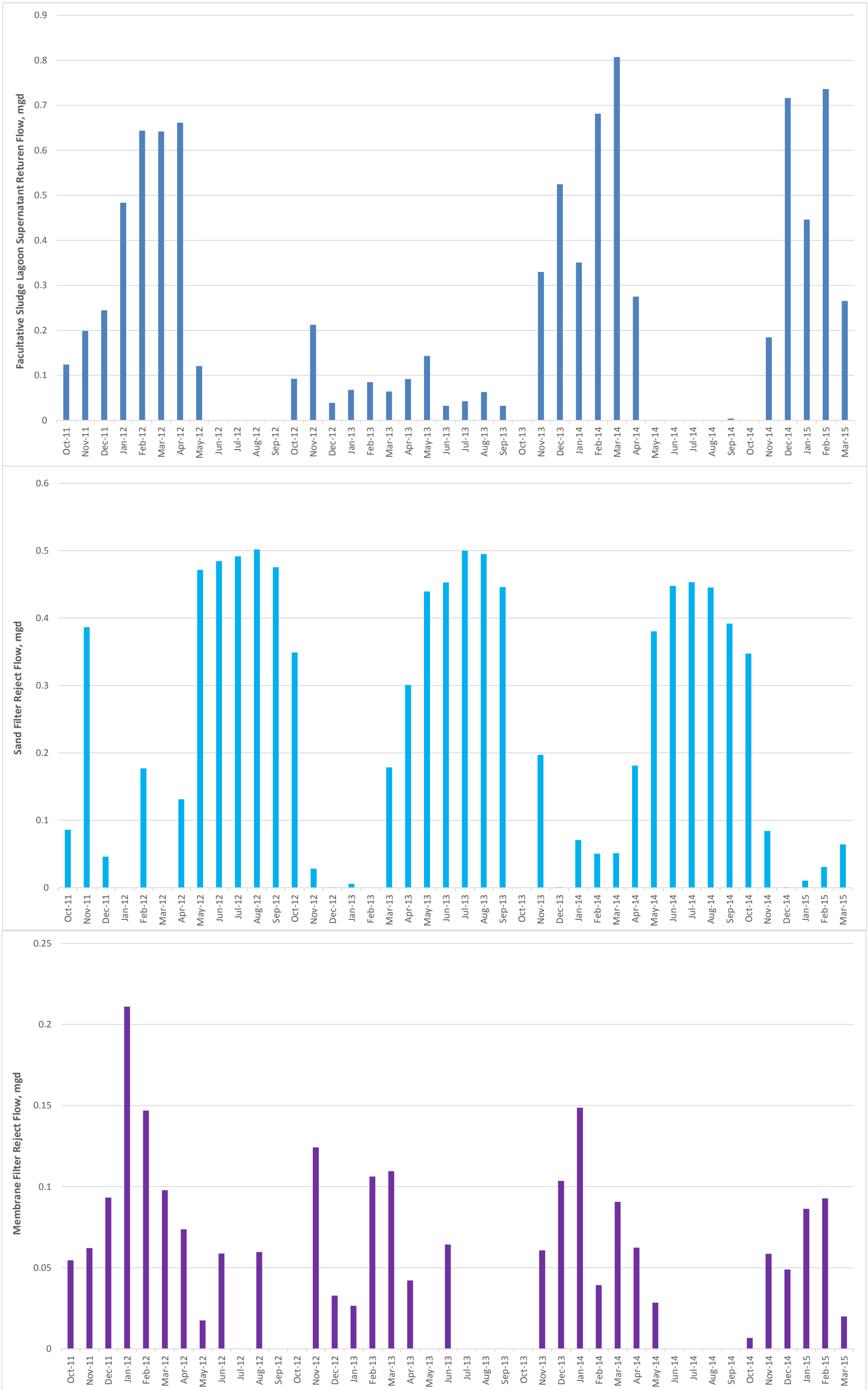


Figure 2-2
Monthly Average Return Flow Rates
2011–2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

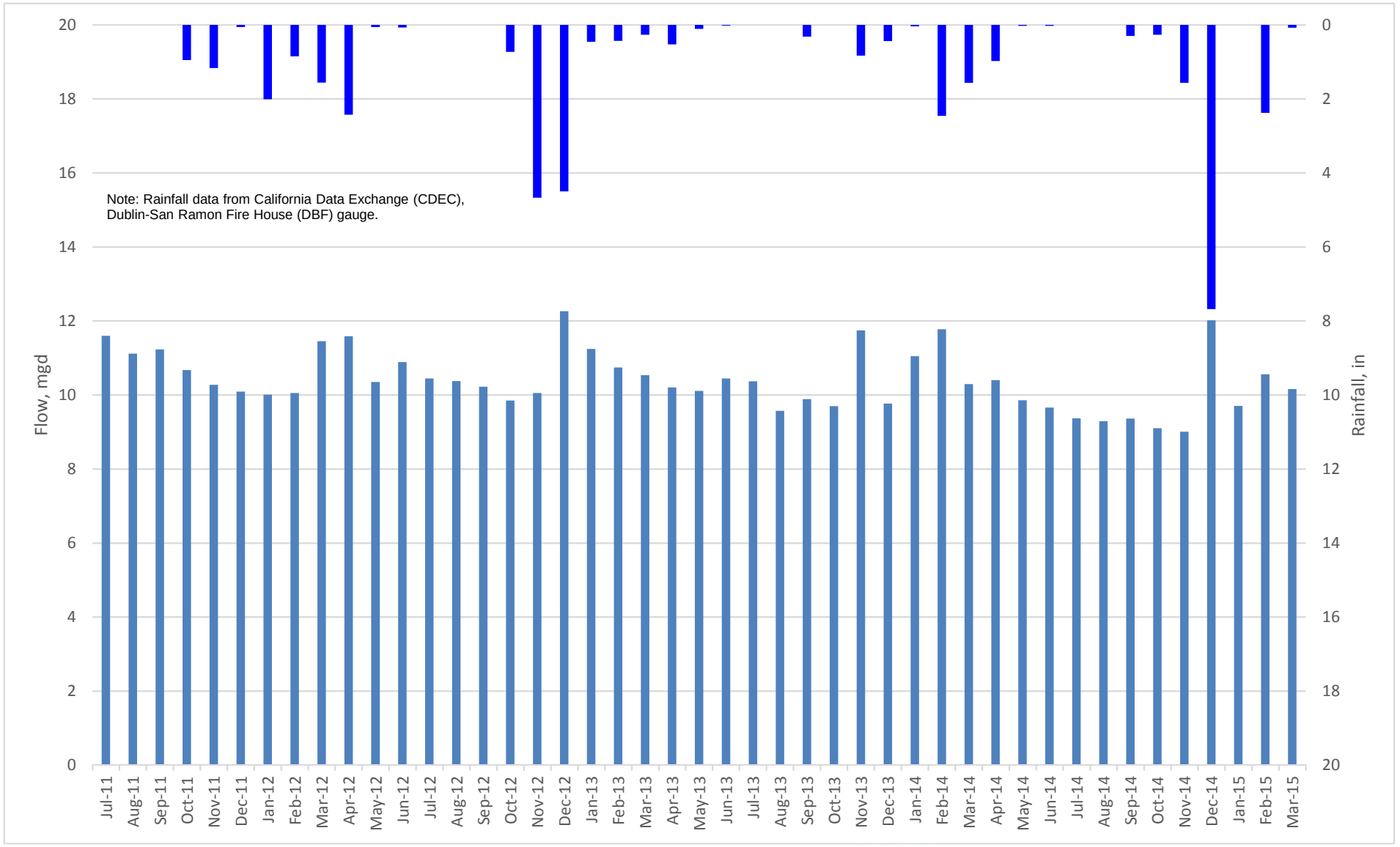


Figure 2-3
Monthly Average Influent Flow and Rainfall, July 2011 – March 2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

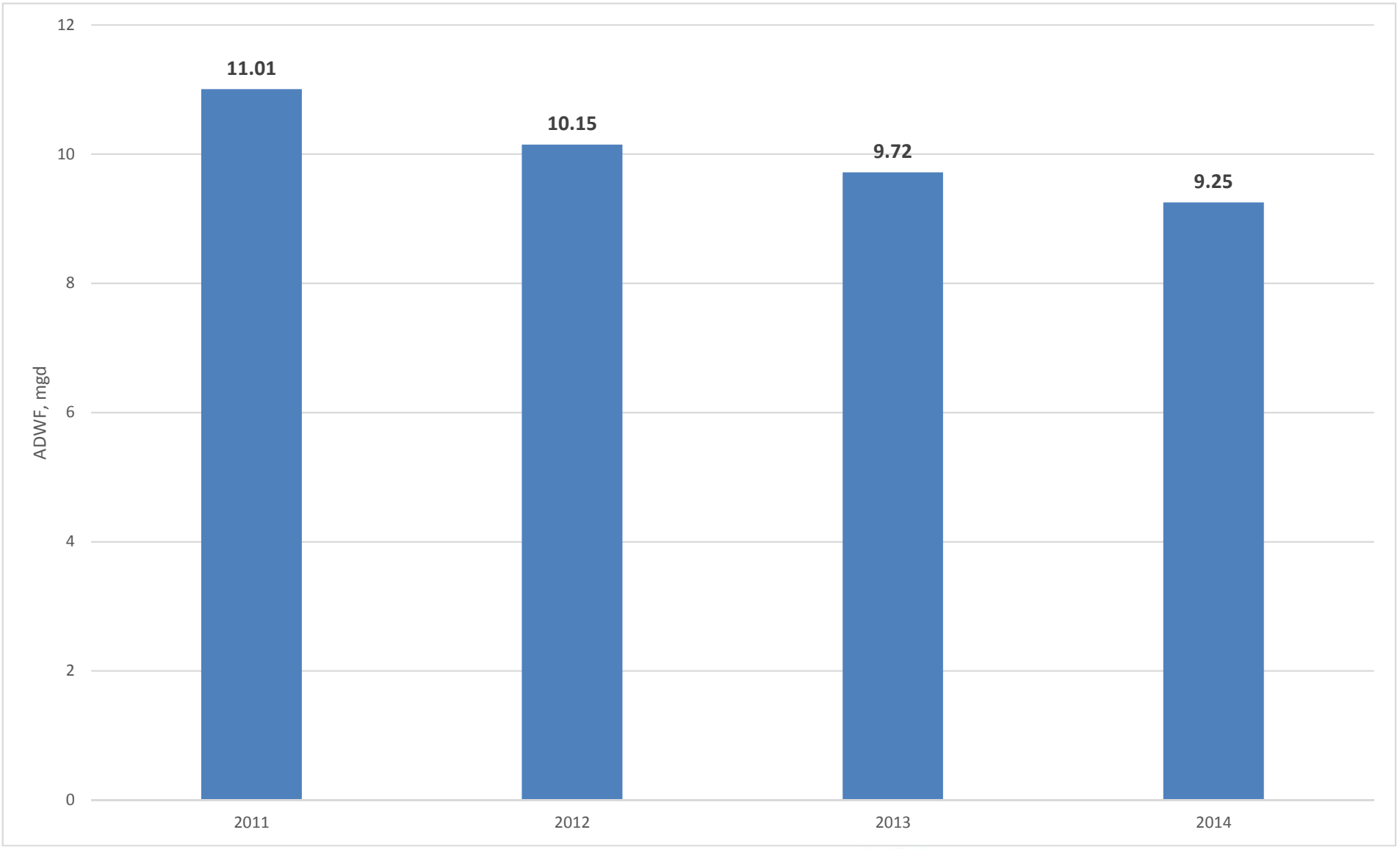


Figure 2-4
Average Dry Weather Flow, 2011–2014

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

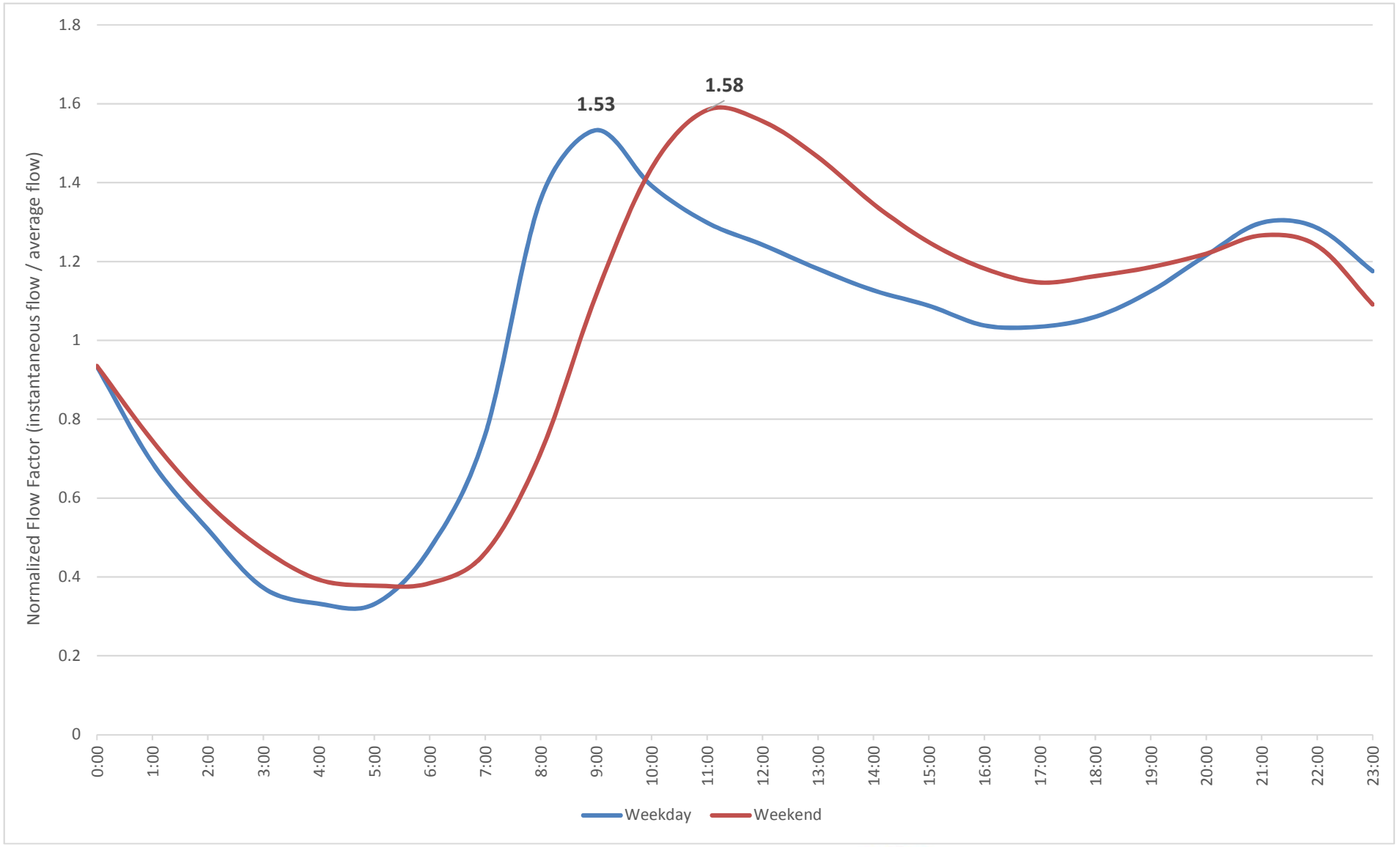


Figure 2-5
Normalized Diurnal Flow, Weekday vs. Weekend September 2014

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

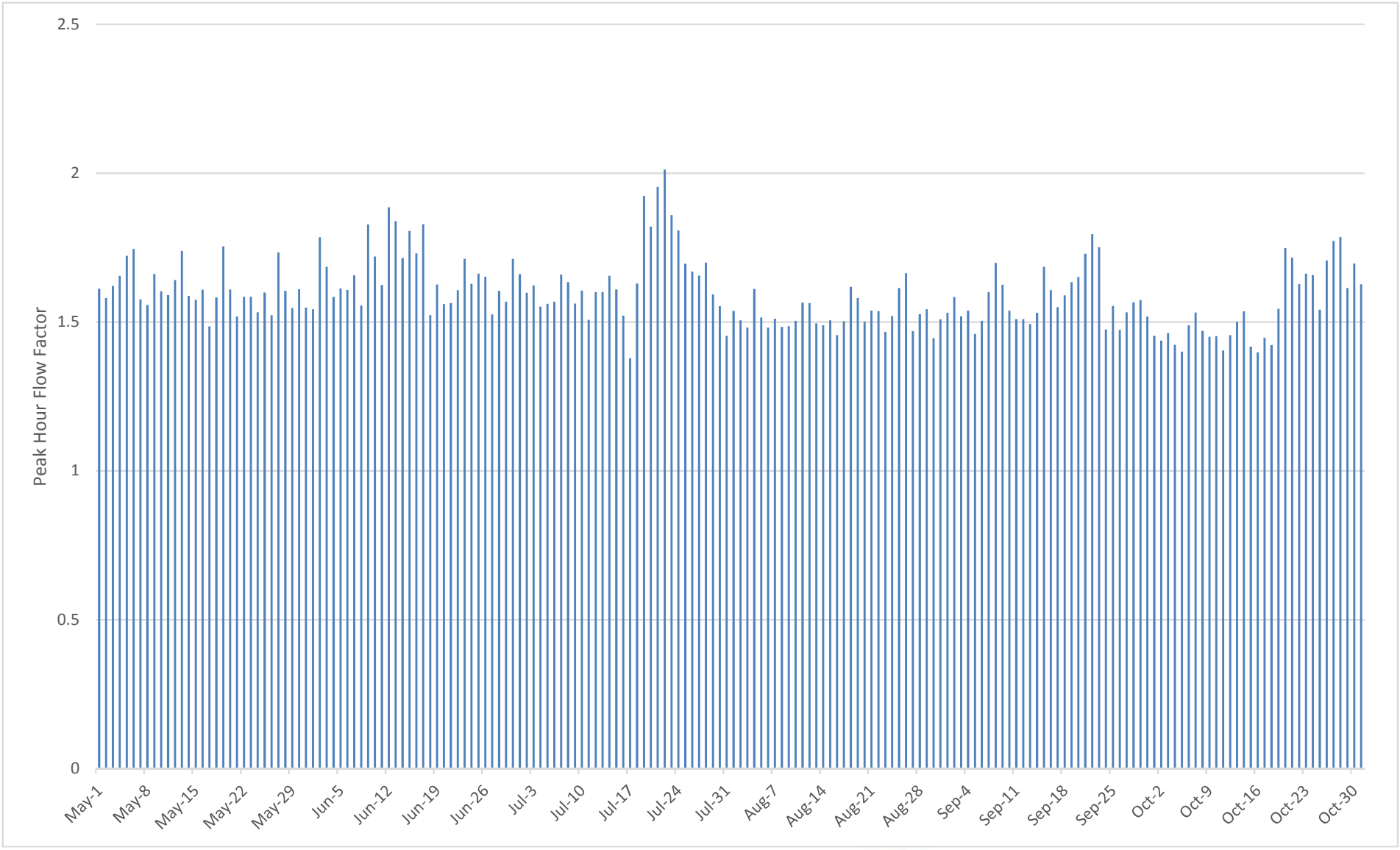


Figure 2-6
Synthesized Dry Weather
Peak Hour Flow Factors
May–October 2013

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

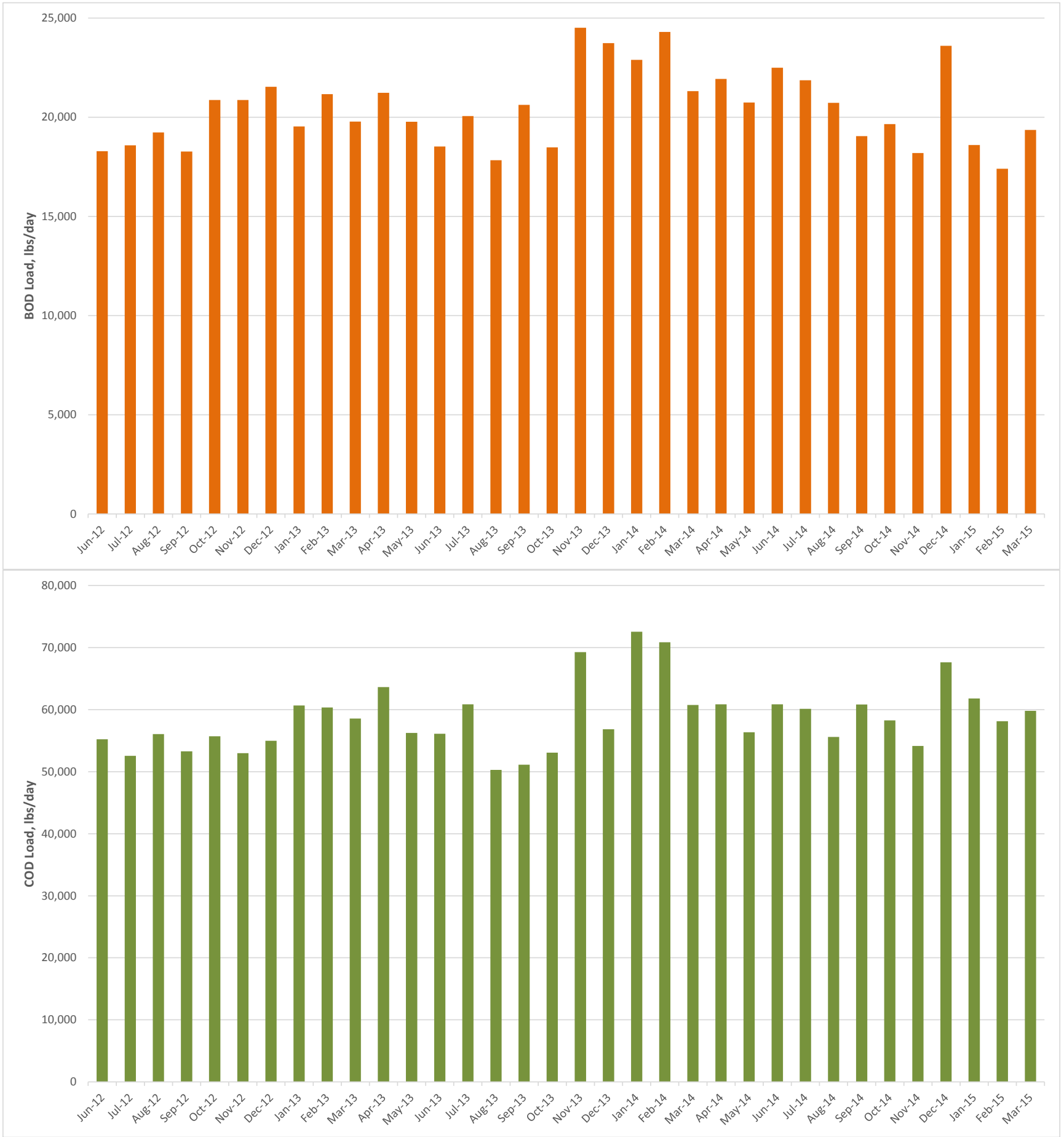


Figure 2-7
Monthly Average Influent Loads, 2012–2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Note: 12-month averages are backward-looking (e.g., the May 2013 value covers the period of June 2012 through May 2013).

WEST YOST ASSOCIATES

Figure 2-8
12-Month Running Average Influent Loads, 2013–2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

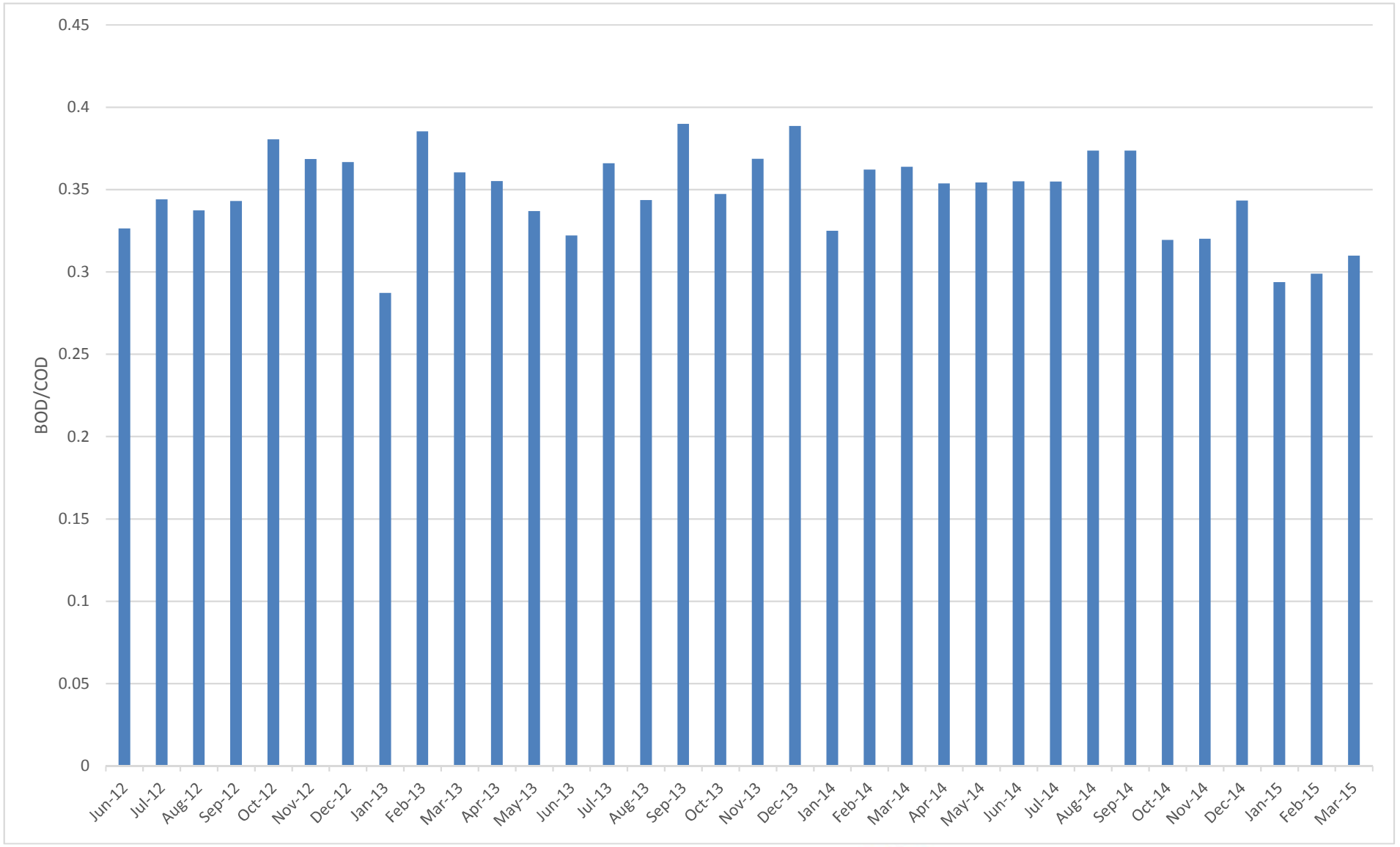


Note: 12-month averages are backward-looking (e.g., the May 2013 value covers the period of June 2012 through May 2013).

WEST YOST ASSOCIATES

Figure 2-9
12-Month Running Average Influent Concentrations, 2013–2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



WEST YOST ASSOCIATES

Figure 2-10
Influent BOD:COD Ratio,
June 2012 to March 2015

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

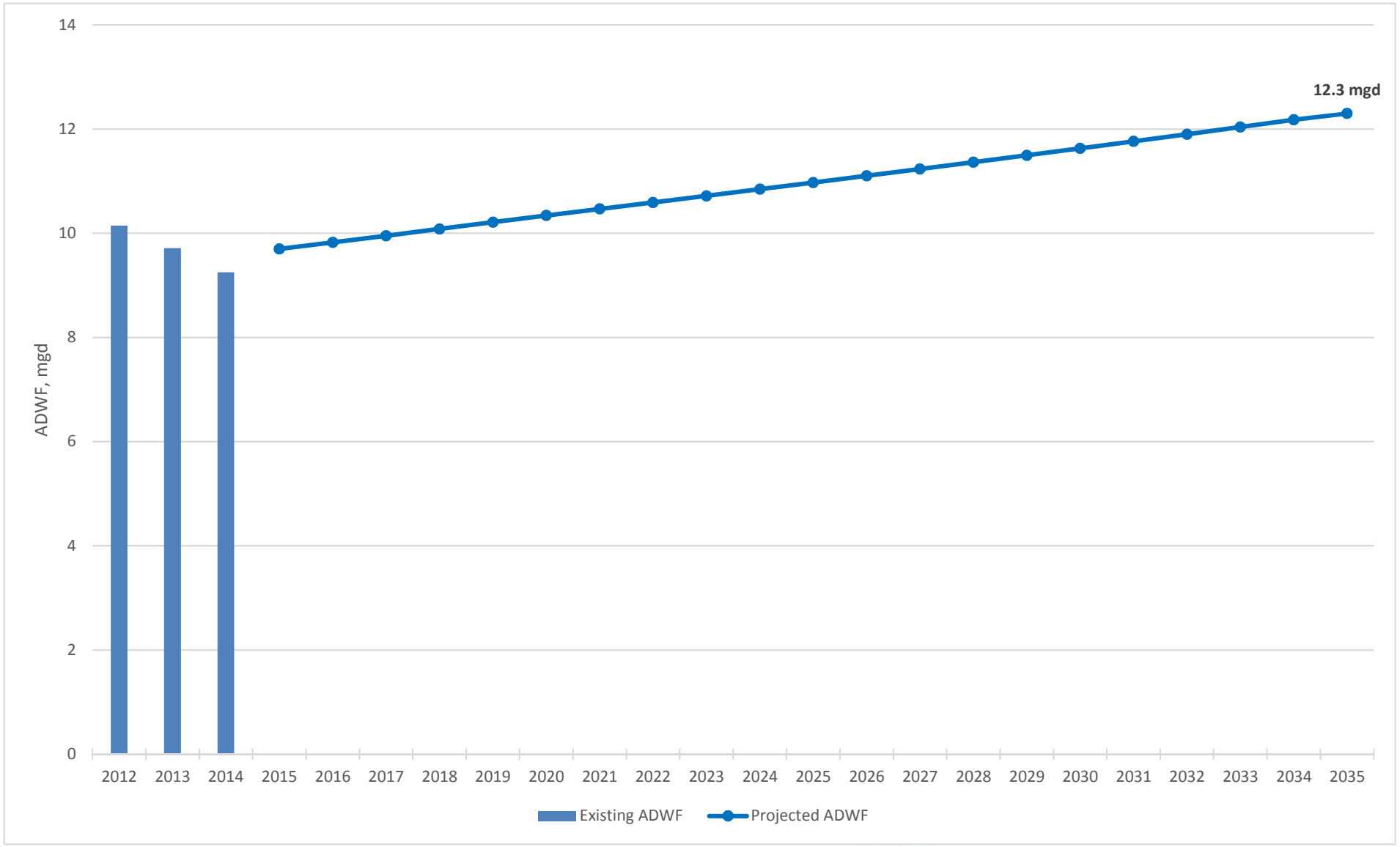


Figure 2-11
Existing and Projected ADWF, 2012–2035

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

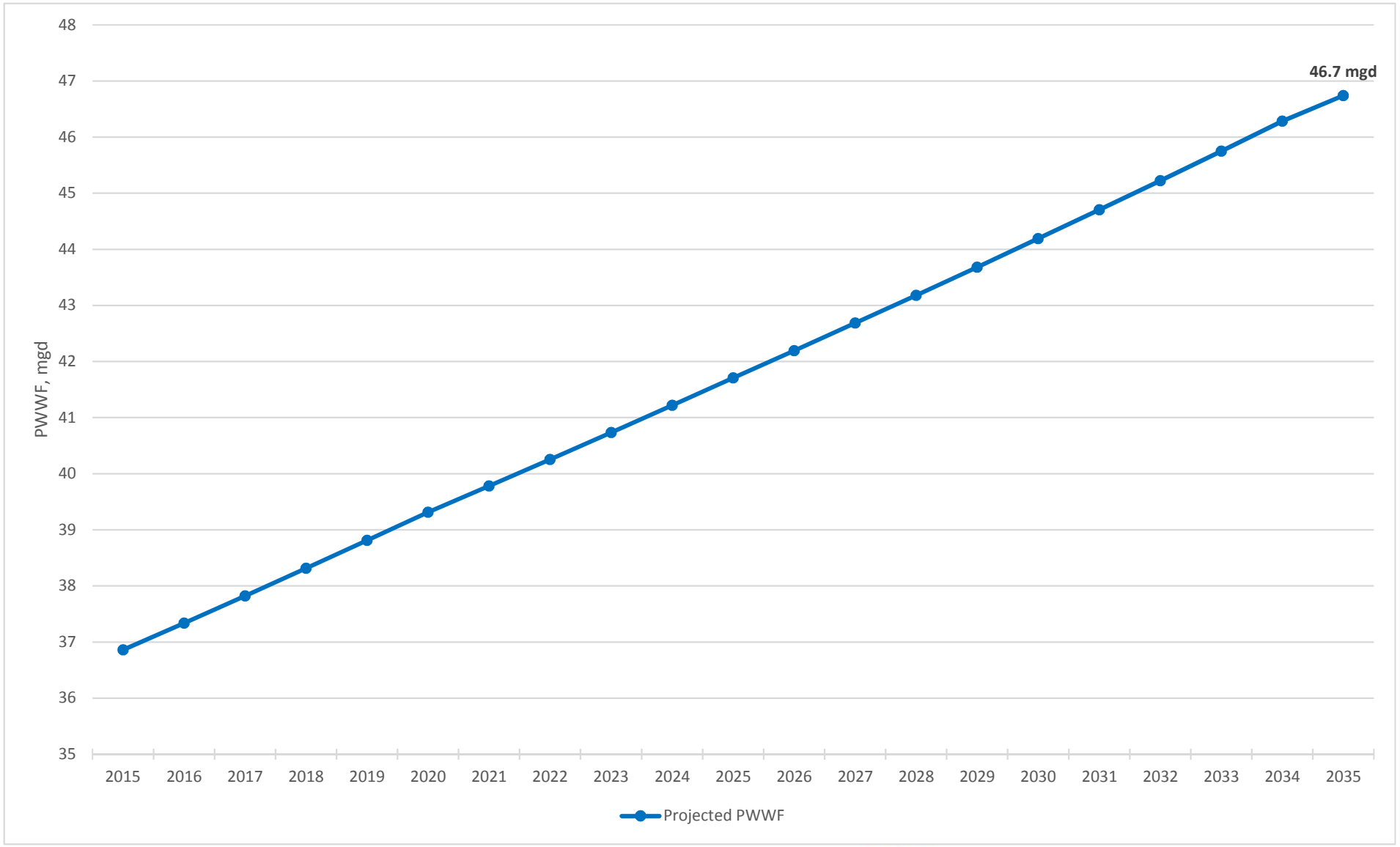


Figure 2-12
Projected PHWWF, 2015–2035

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

CHAPTER 3
Existing Facilities

This chapter presents an overview of the existing facilities at the District's WWTP. Also presented is an evaluation of the available capacity of these existing facilities as compared to the 2035 flow and loading conditions documented in Chapter 2 (Wastewater Flow and Loadings). A more detailed discussion of alternatives for addressing existing and/or future capacity and expansion needs is provided in later chapters of this Master Plan.

The major topics discussed in this chapter are as follows:

- Background Information
- Liquid Treatment Facilities
- Flow Equalization/Storage
- Solids Treatment and Disposal Facilities
- Recycled Water Treatment Facilities
- Ancillary Facilities

3.1 BACKGROUND INFORMATION

This section presents an overview of the existing facilities, discusses the documentation used to complete this chapter, and provides a plant construction history.

3.1.1 Facilities Overview

The WWTP includes two different sites. The main treatment plant area is located on an approximate 22-acre site that is south of Stoneridge Drive and east of Interstate 680. Practically all the Plant's treatment facilities are located at this site, including: all the liquid treatment and recycled water treatment facilities, a portion of the solids treatment facilities, and the cogeneration system facilities. The second treatment plant area is an approximately 162-acre site located across Stoneridge Drive from the main WWTP site. This second site includes six FSLs and the biosolids DLD area. In addition, the LAVWMA storage and pumping facilities are located adjacent to the DLD area.

A site plan of the main WWTP site, showing the locations of most of the facilities discussed in this Chapter, is presented in Figure 3-1. A site plan showing the locations of the treated biosolids storage and disposal facilities and the LAVWMA storage and pumping facilities is presented in Figure 3-2.

Process schematics that depict the function of each of the treatment facilities discussed this Chapter are presented in Figures 3-3 through 3-6, as follows:

- Figure 3-3: Liquid Treatment Train
- Figure 3-4: Storage Facilities
- Figure 3-5: Solids Treatment Train
- Figure 3-6: Recycled Water Treatment Train

Schematics that depict the function of the major cogeneration system facilities are discussed in Section 3.6 of this Chapter.

3.1.2 Documentation Used

The assessment presented herein was based on data and information presented in the following reports:

- *Wastewater Treatment Plant 1997 Master Plan Update Project Phase 1 Report* (Whitley Burchett & Associates and Brown & Caldwell, July 1997)
- *Wastewater Treatment Plant 1997 Master Plan Update Project Phase 2 Report* (Whitley Burchett & Associates and Brown & Caldwell, June 1998)
- *Coliform Limit Study* (Whitley Burchett & Associates, May 2000)
- *Effluent Transport Facilities Basis of Design Report* (Brown and Caldwell, 2002)
- *Odor Control Master Plan Update* (CH2M Hill, July 2004)
- *Plant Process Capacity Study: Dry Weather Capacity* (Brown and Caldwell, 2004)
- *Wet Weather Capacity Study* (Brown and Caldwell, 2005)
- *Final Report 2007 Wastewater Treatment Plant Expansion Planning Report* (Whitley Burchett & Associates, October 2007)
- *Secondary Effluent Wet Weather Capacity Evaluation* (RMC 2014).
- *DSRSD-EBMUD Recycled Water Authority Recycled Water Treatment Facilities Plan* (Carollo 2015)

Additional resources used in the development of this Chapter include: available record drawings of the existing facilities; water quality data collected by the District; the District's online Operations Manual; and information gathered from District staff during site visits and interviews that occurred on March 3rd, May 19th, June 4th, and June 25th of 2015, and February 25th, February 29th, and April 13th of 2016.

3.1.3 WWTP Construction History

The WWTP has been constructed in four primary stages that are referred to as the Stage 1 Project, Stage 2 Project, Stage 3 Project, and Stage 4 Project. As early as the completion of the Stage 2 Project in 1971, it was envisioned that the WWTP would ultimately serve a population of 300,000 – with an ADWF treatment capacity of 36 mgd. Over time, this vision has changed, and the most recent expansion project (i.e. the Stage 4 Project) provided for an ADWF capacity of 17 mgd, which was predicted in the District's 1997 Master Plan Update to occur by 2030.

A summary of the major system components that were constructed under each stage is provided in Table 3-1. As noted, the WWTP Holding Basins, Facultative Sludge Lagoons, and DLD sites were constructed during the same period as the Stage 3 Project, and are therefore included under the Stage 3 Project in Table 3-1. In addition, FSLs were expanded during the same time period as the Stage 4 Project completion, and are also included in Table 3-1 under the Stage 4 Project. Finally, although not indicated in the table, the Stage 2 Project also included the construction of three multi-media filters that were located west of the chlorine contact tank. These filters have since been removed.

Between the larger expansion projects shown in Table 3-1, various smaller scale plant rehabilitation and improvement projects were completed.

- 1966: Outfall pipe structure
- 1968: East Amador lift station
- 1971: Primary effluent diversion valve and piping (change order project)
- 1977: LAVWMA facilities
- 1982: Co-generation/maintenance facilities
- 1985: LAVWMA upgrade
- 1990: East Amador lift station rehabilitation
- 1991: Hypochlorite disinfection
- 1992: DAFT odor control
- 1993: Electrical power supply reconstruction and upgrade
- 1994: Wet weather storage improvements and lab expansion/operations building remodel
- 1995: Co-generation emission conversion and motor control center improvements
- 1996: Primary electrical metering and Facultative Sludge Lagoon Reodorant construction
- 1998: Clean Water Revival Project (microfiltration/reverse osmosis treatment)
- 1999: Supervisory Control and Data Acquisition (SCADA) system
- 1999: Odor control biofilter for DAFT

- 2000: Secondary sedimentation tanks odor control
- 2001: Gas tank relocation, trailer mounted gravity belt thickener and reclaimed water flushing systems at facultative sludge lagoons
- 2003: Effluent transport facilities, mechanical & electric instrumentation maintenance staff relocation
- 2003: DERWA Recycled Water Treatment Facility
- 2004: Treatment plant landscaping and emergency standby generator
- 2005: Grit washer/classifier and effluent monitoring station relocation
- 2007: Secondary Clarifier 2 rehabilitation
- 2008: Odor Reduction Tower rehabilitation and redundancy improvements
- 2008: Bar screen installation and screening conveyance system improvements
- 2010: Cogeneration system improvements and main electrical service relocation
- 2012: Secondary Clarifier 1 rehabilitation
- 2013: DAFT rehabilitation
- 2013: Underground Diesel Tank Replacement

Table 3-1. WWT&BF Major Capacity Expansion History

Parameter		Stage 1 Project	Stage 2 Project	Stage 3 and Related Projects[a]	Stage 4 and Related Projects[b]
Year Complete		1960	1971	1977, 1981, 1982, 1985	2000
ADWF Capacity Rating		unknown	5 mgd	9 mgd (11.5 mgd)[c]	17 mgd
Preliminary Treatment/ Influent Pumping	Headworks	Comminuters	Comminuters	Comminuters	2 existing bar screens
	Influent Pump Station	2 Pumps @ 16 mgd	2 Pumps @ 16 mgd	4 Pumps @ 15 mgd	3 Pumps @ 25 mgd 1 Pump @ 15 mgd
	Pre-aeration Grit Removal	Grit Tank 1	Grit Tank 2	-	Grit Tank 3
Primary Sedimentation		Tank 1	Tank 2	Tank 3	Tank 4
Secondary Treatment	Activated Sludge Reactors	Tanks 1 and 2	No change	Tank 3	Tanks 4 and 5
	Secondary Clarifiers	Small clarifier that was later converted to the dissolved air flotation thickener (DAFT)	Clarifier 1	Clarifier 2 and 3[d]	Clarifier 4
Chlorine Contact Tank		Original tank constructed	Expansion	Expansion to current size	-
Holding Basins		Holding Basin 1	-	Holding Basins 2, 3, and 4	
Solids Handling	Solids Thickening	None	-	DAFT	
	Anaerobic Digestion	Digester 1	Digester 2	-	Digester 3
	Facultative Sludge Lagoons	-[e]	-[e]	Lagoons 1 through 4	Lagoons 6 and 7
	Dedicated Land Disposal Site	-	-	58 acres	46 acres
Odor Reduction Tower		-	-	Constructed	-

[a] Includes the flow equalization and solids storage and disposal project completed in 1982.
[b] Includes the Facultative Sludge Lagoon expansion project that occurred in 1999.
[c] Stage 3A Project drawings say 9 mgd, but the Stage 4 Project drawings indicate 11.5 mgd. It is suspected that the WWTP was re-rated between the Stage 3 and Stage 4 projects.
[d] Includes the existing return activated sludge (RAS) pumping stations and the mixed liquor distribution box.
[e] The WWTP originally had two sludge lagoons that were located where Holding Basins 2 and 3 are now sitting.

3.2 LIQUID TREATMENT FACILITIES

This section presents an overview of the WWTP's liquid treatment facilities that are used for the production of disinfected secondary-treated effluent. The WWTP's tertiary treatment facilities are discussed in Section 3.5 (Recycled Water Treatment Facilities). The secondary treatment system liquid train facilities are addressed in the following sections under these headings:

- Preliminary Treatment/Influent Pumping
- Primary Treatment
- Secondary Treatment
- Secondary Effluent Disinfection
- Secondary Effluent Pumping
- Hydraulic Capacity

3.2.1 Preliminary Treatment/Influent Pumping

The WWTP preliminary treatment/influent pumping facilities include the following elements:

- Screening
- Influent Pump Station
- Plant Inner-Sewer Pump Station
- Influent Flow Measurements
- Pre-aeration Grit Removal System

3.2.1.1 Screening

Flows entering the WWTP from influent sewers includes the wastewater generated within the Cities of Pleasanton, Dublin, and San Ramon and the recycle flows streams from the facultative sludge lagoon (supernatant), the sand filters (backwash flows) and the microfiltration system (reject flows). In addition, the influent flows include occasional discharges to the waste hauler dump station and the recently constructed Vactor Receiving Station located in the southwest corner of Holding Basin 3.

The influent flows are screened via three mechanically-cleaned bar screens that have 3/8-inch openings, a channel width of 5.5 feet (ft), and a rated capacity of 25 mgd each. This results in a screening system firm capacity of 50 mgd and a total capacity of 75 mgd. There is also a manually cleaned emergency bypass bar screen which has a bar spacing of 1-inch and a channel width of 5.5 ft. A summary of the WWTP screening capacity is provided in Table 3-2. As shown, the firm capacity of the mechanical bar screens exceeds the 2035 PHWWF of 46.7 mgd that was presented in Chapter 2 (Wastewater Flow and Loadings).

Table 3-2. Bar Screen Rated Hydraulic Capacities			
Flow Condition	Number of Units	Firm Capacity, mgd	Total Capacity, mgd
Mechanical Bar Screen	3	50	75
Manual Bar Screen	1	--	--

Material removed by the screens is deposited on a conveyor which terminates in two washer compactor devices, which each have a capacity of 15 cubic feet per hour. The resulting washed and dewatered material is then placed in a dumpster for transport off site and final disposal.

3.2.1.2 Influent Pump Station

Screened wastewater flows first to the influent diversion structure and then into the influent distribution structure, which splits the flow into the two chambers of the Influent Pump Station wet well. While there is an isolation gate separating the two chambers, this gate is typically operated in the open condition to allow the chambers to operate as a single wet well. The purpose of the separation gate is to allow for one half of the pump station wet well to be isolated so the other half can be inspected and maintained.

Wastewater is pumped from the two chambers of the wet well by four centrifugal pumps driven by variable speed drives. The east chamber of the wet well has two, 200 horsepower (HP) pumps each with a maximum pumping capacity of 25 mgd and the west chamber of the wet well has one, 200 HP pump with a maximum capacity of 15 mgd and a second 200 HP pump with a maximum capacity of 25 mgd. The lower capacity pump in the second chamber has a trimmed impeller. Each pump discharges separately into the Raw Sewage Distribution Channel. In total, the Influent Pump Station has a firm capacity of 65 mgd and a total capacity of 90 mgd. The 15 mgd Influent Pump can be restored to 25 mgd capacity by changing the pump impeller to match the other three pumps. This modification would result in firm and total capacities of 75 and 100 mgd, respectively. A summary of the Influent Pump Station capacity is shown in Table 3-3. As shown, the firm capacity of the Influent Pump Station exceeds the 2035 PHWWF of 46.7 mgd that was presented in Chapter 2 (Wastewater Flow and Loadings).

Table 3-3. Influent Pump Station Capacity				
Flow Condition	Number of Units	Rated Capacity (each), mgd	Firm Capacity, mgd	Total Capacity, mgd
High Capacity Pumps	3	25	65	90
Low Capacity Pump	1	15		

3.2.1.3 Plant Inner-Sewer Pump Station

The plant inner-sewer pump station wet well is adjacent to the influent diversion structure mentioned above. The plant inner-sewer pump station wet well receives underflow from the DAFT, secondary scum from the secondary clarifiers, and flow that enters the WWTP sanitary and storm sewers. The plant inner-sewer pump station consists of two, 20 HP, 1.0 mgd submersible pumps. Flow from the pump station is discharged to the Grit Tank effluent channel (bypassing the grit removal system).

3.2.1.4 Influent Flow Measurement

Influent flows are measured via four magnetic flow meters that are installed on the discharge pipes from each pump in the Influent Pump Station. In this location, the influent flow meters record the total influent as the sum of the flow from each Influent Pump (including the return streams that enter the WWTP via the influent sewers), but do not record the flow discharged from the Plant Inner-Sewer Pump Station.

3.2.1.5 Pre-aeration Grit Removal System

The Pre-aeration grit removal system consists of the pre-aeration grit removal tanks (grit tanks) and the grit processing equipment. Flows are pumped from the Influent Pump Station wet well into the Raw Sewage Distribution Channel, which splits the flow between West and East Influent Distribution Channels. From these channels, the flows enter three aerated grit tanks through submerged baffles, where the east channel serves two grit tanks (Grit Tank 2 and 3) and the west channel serves one grit tank (Grit Tank 1).

3.2.1.5.1 Aerated Grit Tanks

Grit Tank 1 has an approximate volume of 33,000 gallons and Grit Tanks 2 and 3 have an approximate combined volume of 43,000 gallons. The tanks receive aeration from three Pre-Aeration Blowers that discharge to a common header that serves all grit tanks and are designed to provide a discharge pressure of 6 pounds per square inch (psi). One of these blowers has a maximum capacity of 360 cubic feet per minute (cfm) and the remaining two blowers have a maximum capacity of 275 cfm each, resulting in a firm capacity of 550 cfm and a total capacity of 910 cfm. The 275 cfm blowers are configured so they may each be replaced with a 360 cfm blower in the future, which would result in firm and total capacities of 720 and 1,080 cfm, respectively. These blowers are designed to provide 3 to 5 cfm per foot of grit tank wall length (cfm/ft of wall length), which is at the low end of the recommended design range of 3 to 8 cfm/ft of wall length (Metcalf and Eddy/AECOM, 2014).

Wastewater discharged from the grit tanks enters the grit tank effluent channels. These channels can be isolated from each other, such that all the flow from Grit Tank 1 is discharged to Primary Sedimentation Basin 1, the flow from Grit Tank 2 is discharged to Primary Sedimentation Basin 2, and the flow from Grit Tank 3 can enter Primary Sedimentation Basin 3 and/or 4. However, the District typically operates the grit tank effluent channels with all the gates open allowing the flow from the three grit tanks to be split by the flow hydraulics between the four primary sedimentation basins.

A summary of the grit removal system capacity based on typical design values is provided in Table 3-4. Assuming a recommended hydraulic detention time 3.5 minutes, Grit Tank 1 has a capacity of approximately 13.6 mgd and Grit Tanks 2 and 3 have a capacity of approximately 17.7 mgd each – providing a firm capacity of 31.3 mgd and a total capacity of 49.0 mgd. Also shown are the Pre-aeration blower capacities.

Table 3-4. Pre-aeration Grit Removal System Capacity

Facility	Number of Units	Typical Design Range	Selected Design Value	Capacity per Unit	Firm Capacity[a]	Total Capacity
Grit Tank 1	1	2 – 5 min @ peak flows	3.5 min	13.6 mgd	31.3 mgd	49.0 mgd
Grit Tanks 2 and 3	2			17.7 mgd		
Pre-aeration Blowers						
Larger Blower	1	N/A	N/A	360 cfm	550 cfm	910 cfm
Smaller Blowers	2	N/A	N/A	275 cfm		

[a] Available capacity with the largest unit out of service.

As shown in Table 3-4, the firm capacity of the aerated grit tanks is less than the 2035 PHWWF of 46.7 mgd presented in Chapter 2 (Wastewater Flow and Loadings); however, the firm capacity exceeds the 2035 PHDWF of 23.4 mgd. Moreover, at a less conservative design value (i.e., between 2 and 2.5 minutes of hydraulic detention time), the grit tank firm capacity would be adequate at the PHWWF. Nevertheless, if flows through the WWTP exceed the actual capacity of the aerated grit tanks, some grit would potentially be transferred to the anaerobic digesters via the primary sedimentation basins. Furthermore, flows exceeding the capacity of the grit tanks would be fairly short term during a PHWWF event, and normally all three grit tanks will be in service. Therefore, it is recommended that the grit tanks be evaluated based on their ability to accommodate a PHDWF flow event with one unit out of service, and a PHWWF event with all units in service. Given these recommendations, it is concluded that the existing grit removal tanks are adequately sized for the 2035 conditions.

3.2.1.5.2 Grit Processing Equipment

Grit slurry is removed from the grit tanks and pumped to two grit separators for dewatering via three, 10 HP, recessed impeller centrifugal pumps, which have a capacity of 200 gallons per minute (gpm) each. Each grit tank is serviced by a single grit pump. Grit Pump 1 discharges to one separator and Grit Pumps 2 and 3 discharge to a second separator. The two grit separators each have a capacity of 250 gpm. Liquid from the separators is diverted back to the grit tanks and grit is passed on to one of the two grit classifiers. The classifiers wash and dewater the grit, and the resulting material is then placed in a dumpster for transport off site and final disposal. Table 3-5 shows the rated capacities of the supporting grit processing equipment.

Table 3-5. Pre-aeration Grit Removal System Capacity

Facility	Number of Units	Capacity per Unit, gpm	Firm Capacity[(a)], gpm	Total Capacity, gpm
Grit Pumps	3	200	400	[(b)]
Grit Separators	2	250	250	500
Grit Classifiers	2	N/A	N/A	N/A

(a) Available capacity with the largest unit out of service.
(b) Due to capacity limitations in the Grit Separators, all three pumps cannot be operated simultaneously.

3.2.2 Primary Treatment

The primary treatment system consists of the primary sedimentation basins and their associated solids pumping systems.

3.2.2.1 Primary Sedimentation Basins

Effluent from the grit tanks enters the four, rectangular primary sedimentation basins via the grit effluent channels, as discussed previously. Primary effluent flows over notched weirs into a set of launders that convey effluent into the primary effluent channel. Flow from the primary effluent channel passes into the settled sewage channel where it is transported to the secondary activated sludge process.

All four sedimentation basins are 100 ft long, 19 ft wide, and have a 9 ft water depth, providing in each basin a surface area of 1,900 square ft and a volume of approximately 17,100 cubic feet (cu-ft). The plant is also configured to allow for two additional primary sedimentation basins to be constructed west of Basin 1.

A summary of the capacity of the existing primary sedimentation basins, based on typical design values, is shown on Table 3-6. As shown, the capacity of the sedimentation basins based on typical design values is less than the current WWTP flows presented in Chapter 2 (Wastewater Flow and Loadings) – let alone the 2035 flows. The average overflow rate between July 2011 and March 2015 was 1,400 gallons per day per square foot (gpd/sq-ft) and the detention time was approximately 1.2 hours, well outside normal design parameters of 800 to 1,200 gpd/sq-ft and 2 hours of hydraulic retention time (HRT).

Table 3-6. Primary Sedimentation Basin Capacity Based on Typical Design Values

Design Parameter	Number of Units	Typical Design Range[a,b]	Selected Design Value	Unit Capacity, mgd	Theoretical Firm Capacity, mgd	Theoretical Total Capacity, mgd
Detention Time at Average Flow	4	1.5 – 2.5 hour	2 hour	1.5	4.5	6.0
Overflow Rate at Average Flows		800-1,200 gpd/sq-ft	1,000 gpd/sq-ft	1.9	5.7	7.6
Overflow Rate at Peak Flows		2,000-3,000 gpd/sq-ft	2,500 gpd/sq-ft	4.8	14.4	19.2

[a] Metcalf and Eddy/AECOM, 2014.
[b] Historical data indicate actual overflow rates have ranged from 1,100 to 2,600 gpd/sq-ft, with an average of 1,400 gpd/sq-ft.

The primary sedimentation basins receive both the screened, raw influent and the underflow stream from the DAFT. To assess removal rate in the primaries, it is necessary to estimate the combined BOD, COD and TSS loadings from the two sources. BOD and COD concentrations and loads in the DAFT underflow are not monitored, so a mass balance on the DAFT was performed to estimate this portion of the loads to the primary sedimentation basins. Using the available TSS data, the DAFT underflow BOD and COD concentrations returned to the primary sedimentation basins were estimated by assuming the BOD to TSS and COD to TSS ratios of the plant influent and secondary effluent are representative of the BOD to TSS ratios of the primary solids and Waste Activated Sludge (WAS) portions of the DAFT underflow, respectively. While the actual ratios may be different, the combined DAFT underflow is estimated to contribute 9.5 percent of the total BOD load, 11.4 percent of the total TSS load, and 13 percent of the total flow to the sedimentation basins. Therefore, minor over or under estimation of the BOD to TSS and COD to TSS ratios would not have a significant impact on the primary sedimentation basin removal calculations.

A summary of the BOD, COD, and TSS removal rates based on available paired influent, primary effluent, and DAFT underflow concentration and flow data collected between June 2012 and March 2015 is presented in Table 3-7. Note that detention times shown in Table 3-7 are only for days when paired removal and detention time data were available. Average, maximum, and minimum detention times for the entire period of July 2011 through March 2015 are 1.19, 1.52, and 0.61 hours, respectively.

Table 3-7. Primary Sedimentation Basin Performance

Statistic	Percent Removal			Detention Time, hours
	BOD	COD	TSS	
Average	31%	40%	47%	1.04
Maximum	61%	55%	67%	1.20
Minimum	4%	19%	15%	0.81

A comparison of BOD and TSS removal rates observed in the primary sedimentation basins to the typical removal rates (Metcalf and Eddy/AECOM, 2014) is provided in Figures 3-7 and 3-8. Comparing Table 3-7 to Figures 3-7 and 3-8, the current average BOD removal of approximately 31 percent is slightly higher than the predicted BOD removal rates shown in Figure 3-7, but the current average TSS removal of approximately 47 percent is slightly lower than predicted TSS removal rates shown in Figure 3-8.

Ultimately, the appropriate sizing criteria for the primary sedimentation basins will depend on how much removal is desired in this process versus in the secondary treatment process. One advantage of removing BOD and TSS in the primaries is that the carbon associated with these parameters is directed to the anaerobic digesters where it is used to create energy, versus directing this carbon to the secondary reactors where it requires energy input to digest. In addition, increased loads to the secondary reactors will produce increased solids generation in this process, resulting in increased solids loads to the secondary clarifiers (and a need for additional secondary clarifiers). The optimum sizing for the primary sedimentation basins must account for these factors.

The performance curves in Figures 3-7 and 3-8 are based on typical design values (Metcalf and Eddy/AECOM, 2014). As shown, the WWTP's BOD removal rates could increase up to 35 percent (or possibly higher) and the TSS removal rates could increase up to 55 percent if additional primary sedimentation capacity was added to meet upper-end design criteria.

A summary of the primary sedimentation capacities using the upper limit of recommended process rating values is presented in Table 3-8. Also, shown in this table are the current average overflow rate and detention time, and what the primary sedimentation basin capacity rating would be if these values were selected as the recommended design values.

Table 3-8. Primary Sedimentation Basin Capacity Assuming Upper End of Typical Design Range Compared to Current Operating Conditions

Design Parameter	Number of Units	Typical Design Range[(a)]	Selected Design Value	Unit Capacity, mgd	Theoretical Firm Capacity, mgd	Theoretical Total Capacity, mgd
Recommended Criteria						
Detention Time at Average Flow	4	1.5 – 2.5 hour	1.5 hour	2.0	6.0	8.0
Overflow Rate at Average Flows		800 - 1,200 gpd/sq-ft	1,200 gpd/sq-ft	2.3	6.9	9.2
Overflow Rate at Peak Flows		2,000-3,000 gpd/sq-ft	3,000 gpd/sq-ft	5.7	17.1	22.8
Current Operating Conditions						
Average Detention Time	4	1.5 – 2.5 hour	1.2 hour	2.6	7.7	10.3
Average Overflow Rate		800 - 1,200 gpd/sq-ft	1,400 gpd/sq-ft	2.7	7.9	10.6

[(a)] Metcalf and Eddy/AECOM, 2014.

3.2.2.2 Sludge and Scum Pumping System

Each of the four primary sedimentation basins is served by a 10 HP, positive displacement sludge pump with a capacity of 90 gpm each at a design head of 35 ft. The primary sedimentation basins are also serviced by three, 10 HP, positive displacement scum pumps with a capacity of 50 gpm each at a design head of 35 ft. Basins 1 and 2 each have their own scum pump and Basins 3 and 4 share one pump, where scum from Sedimentation Basin 4 flows to the scum trough of Sedimentation Basin 3. All three scum pumps can send scum to the anaerobic digesters. Scum Pump 1 can also receive scum from Grit Tank 1 and Scum Pump 3 can also receive scum from Grit Tanks 2 and 3. The primary sludge and scum pumps discharge to the DAFT.

3.2.3 Secondary Treatment

The Secondary Treatment System consists of the following elements:

- Activated Sludge Reactors
- Mixed Liquor/WAS Pumping
- Secondary Clarifier Facilities

3.2.3.1 Activated Sludge Reactors

Primary effluent enters the activated sludge reactors (aeration tanks) from the Settled Sewage Channel, and return activated sludge (RAS) enters the tanks from a separate RAS channel. Aeration tank effluent passes through the mixed liquor channel to the mixed liquor distribution structure, which splits the flow between the four secondary clarifiers. A portion of the mixed liquor is wasted from the mixed liquor channel to control the mixed liquor concentrations and mean cell residence time (MCRT) in the activated sludge process.

The activated sludge reactors consist of five aeration tanks that can be operated in either a series or parallel operations mode. Each of the five aeration tanks is 210 ft long, 30 ft wide and has a 15.3 ft water depth, providing an approximate volume of 96,400 cu-ft each. In addition, Tank 3 is equipped with both submerged mixers and aeration equipment to allow it to be operated either as an anaerobic/anoxic selector or as an aerated basin. A sketch of process configuration as series or parallel mode is provided in Figure 3-9.

The aeration tanks are typically operated in series configuration during the summer months. While in the series configuration, all wastewater is directed to the middle aeration tank (Tank 3), and then is split between a northern flow path through Tank 2 then 1, and southern flow path through Tank 4 then 5. With this configuration, the activated sludge performs biological phosphorus removal by "selecting" for phosphate accumulating organisms. This operational strategy is used to improve the secondary sludge settleability.

During the winter months, the District shifts the configuration to parallel operations mode, where all five tanks are fed equally. In the parallel configuration, phosphorus levels in the primary solids sent to the digesters are reduced because biological phosphorus removal is not occurring. This operation is desired when phosphorus-rich cap water is seasonally returned from the Facultative Sludge Lagoons. When this operation has historically been coupled with biological phosphorus, the struvite precipitation caused significant problems in the District's solids handling processes.

Table 3-9 shows the activated sludge reactor operating conditions developed under the Stage 4 Improvements Design, which are based on maximum week conditions. Also shown are operating conditions based on the 2035 maximum 30-day influent conditions defined in Chapter 2 and a steady state model developed for this Master Plan using BioWIN software. Although the Stage 4 values are based on maximum week loads and the revised values from the BioWIN model are based on maximum 30-day loads, the analysis provides a reasonable comparison of the two sets of design conditions. As shown in Table 3-9, the projected maximum month 2035 loadings to the reactors are significantly lower than the maximum week loads defined under the Stage 4 Improvements Design. Therefore, the activated sludge reactors should have more than adequate capacity to handle the flows and loads anticipated at the 2035 conditions.

Table 3-9. Activated Sludge Reactor Design Operating Conditions (2035)

Flow Condition		Units	Stage 4 Value[a]	Revised Values from BioWIN Model[b]
Maximum BOD Loadings		lb BOD/day	34,800[c]	22,400[d]
Minimum Temperature		degree C	13	13
Mean Cell Residence Time (MCRT)		days	1.5 – 2.5	2.3
Hydraulic Retention Time (HRT) at ADWF (Stage 4)/2035 Maximum 30-day (Revised) Flow		hours	5.1	5.4
Mixed Liquor Suspended Solids @ Average Loading and 2.5 day MCRT	Volatile Suspended Solids	mg/L	2,300	1,500
	Total Suspended Solids	mg/L	Not Reported	2,000
Mixed Liquor Suspended Solids @ Maximum Week (Stage 4)/30-day (Revised) Loading and 2.5 day MCRT	Volatile Suspended Solids	mg/L	3,350	1,800
	Total Suspended Solids	mg/L	Not Reported	2,400

[a] Stage 4 values are based on maximum week conditions.
[b] Revised values from BioWIN Model are based on maximum 30-day conditions.
[c] Assumes a 44% reduction in BOD in the primaries.
[d] Assumes a 30% reduction in the primary sedimentation basins, which reflects current performance. Removal rates could be higher if more conservative primary sedimentation basin design criteria are applied.

There are three, 400 HP aeration blowers located in the blower building adjacent to the mixed liquor channel that provide aeration and mixing to the activated sludge reactors, the RAS channel, and the settled sewage channel. Each aeration blower has a capacity of 8,500 cfm. Four 9 HP mechanical mixers are also installed in Aeration Tank 3 for use when operating in the anoxic/anaerobic mode.

3.2.3.2 Mixed Liquor/WAS Pumping

The Mixed Liquor/WAS pumping facility consists of three, 20 HP, submersible pumps that each have a capacity of 835 gpm. Thus, the facility has a firm capacity of 1,670 gpm (or 2.4 mgd) and a total capacity of 2,505 gpm (or 3.6 mgd) (based on the stated capacity of the individual pumps). The Mixed Liquor/WAS pumps discharge via a common header to the DAFT.

3.2.3.3 Secondary Clarifier Facilities

The secondary clarifier facilities consist of the clarifiers, the associated RAS and scum pumping systems, and the RAS pipelines.

3.2.3.3.1 Clarifiers

Mixed liquor enters the four, circular secondary clarifiers from the mixed liquor distribution structure. Treated secondary effluent that overflows the secondary clarifier weirs can be distributed to one of the following locations:

- Secondary Effluent Disinfection System (discussed in Section 3.2.4)
- Facultative Sludge Lagoon Cap Water Pump Station (discussed in Section 3.4.4)
- Microfiltration Transfer Pump Station (discussed in Section 3.5.2.1)

Clarifiers 1, 2 and 3 are 90 ft in diameter, and Clarifier 4 is 110 ft in diameter. All four clarifiers have a side water depth of 14 ft. Capacity information for the secondary clarifiers, assuming the 2035 maximum 30-day loading mixed liquor suspended solids concentration of 2,400 mg/L shown in Table 3-9, is provided in Table 3-10.

As shown in Table 3-10, current average flows of 10.3 mgd put the secondary clarifiers at, or near, their available firm average flow capacity based on an evaluation of the overflow rates. However, because the mixed liquor concentrations in the activated sludge reactors is relatively low (even at the design conditions), the secondary clarifiers have more than enough average solids loading capacity to processes the estimated 2035 average flows of 13.5 mgd. Comparing the theoretical firm capacities shown in Table 3-10 to the Current and 2035 PHDWF values defined in Chapter 2 (Wastewater Flow and Loading) of 18.4 and 23.4 mgd, respectively, and comparing the total capacities shown in Table 3-10 to the Current and 2035 PHWWF values defined in Chapter 2 (Wastewater Flow and Loading) of 37.1 and 46.7 mgd further supports a conclusion that the clarifier facility may be at or near is available peak flow treatment capacity (while solids loading capacity is more than adequate).

Table 3-10. Secondary Clarifier Capacity Assuming Typical Design Values

Design Parameter	Number of Units	Typical Design Range[a]	Selected Design Value	Unit Capacity[b], mgd	Theoretical Firm Capacity, mgd	Theoretical Total Capacity, mgd
Overflow Rate at Average Flows	4	400-600 gpd/sq-ft	500 gpd/sq-ft	3.2/4.8	9.5	14.3
Overflow Rate at Peak Flows		1,000-1,200 gpd/sq-ft	1,100 gpd/sq-ft	7.0/10.5	21.0	31.4
Surface Loading Rate at Average Flows		0.8-1.2 lb/sq-ft-hr[c]	1 lb/sq-ft-hr[c]	7.6/11.4[d]	22.9[d]	34.3[d]
Surface Loading Rate at Peak Flows		2 lb/sq-ft-hr[c]	2 lb/sq-ft-hr[c]	15.3/22.8[d]	45.8[d]	68.6[d]

[a] Metcalf and Eddy/AECOM, 2014.
[b] Values are shown for three 90 foot clarifiers and the 110-foot clarifier separately.
[c] pounds per square foot per hour.
[d] Based on a mixed liquor concentration of 2,400 mg/L, which is the calculated mixed liquor concentration at the 2035 maximum 30-day load conditions and an MCRT of 2.5 days.

To better understand the capacity limitations of the existing secondary clarifiers, the clarifiers were modeled using a state point analysis tool. The state point analysis model provides a graphical means of predicting clarifier performance under stressed conditions using a gravity flux curve approach, and allows for site-specific factors like Mixed liquor suspended solids (MLSS) concentrations, sludge settleability (measured as the sludge volume index, or SVI), and hydraulic flows into and out of the clarifiers to be simultaneously considered.

The WWTP's SVI data collected between January 2014 through September 2015 is shown in Figure 3-10. As shown, SVI values are higher during the January to April than they are during the April to July period. This change in SVI is expected because the WWTP activated sludge reactors are operated with an anaerobic selector during the summer months, which improves settleability. The maximum recorded SVI is 300 milliliters per gram (mL/g), and there are several occurrences of SVI values between 250 and 300 mL/g during the January to April period. The overall average SVI is approximately 170 mL/g and the overall average in the January to April period is approximately 190 mL/g.

Table 3-11 shows the maximum flow that is estimated to be reliably treatable in the secondary clarifiers using the state point analysis model, assuming at SVI conditions ranging between 170 and 300 mL/g and the following:

- Maximum RAS flow = 12.2 mgd (see discussion below on RAS pumping capacities)
- MLSS Concentration = 2,400 mg/L

Table 3-11. State Point Analysis Results				
Operating Assumption	Peak Flow Without Failure at Different SVI Values, mgd			
	SVI = 300 mL/g	SVI = 250 mL/g	SVI = 190 mL/g	SVI = 170 mL/g
Largest Unit Out of Service	13.0	15.0	24.0	26.0
All Units Online	20.0	23.0	35.0	38.0

As shown in Table 3-11, the state point analysis predicts that the maximum flow that can be reliably treated through the secondary clarifiers with the largest unit out of service ranges between 13.0 and 26 mgd, and the maximum flow that can reliably treated when all the clarifiers are online ranges between 20 and 38 mgd.

Because secondary clarifiers are a reliable treatment process, and major repairs can typically be scheduled during dry-weather periods such that a catastrophic failure does not occur during peak wet weather flow periods, a reasonable approach to clarifier peak flow design is to provide adequate firm capacity to reliably treat PHDWF and provide adequate total capacity to reliably treat PHWWF. Comparing the largest unit out of service values in Table 3-11 to the Master Plan PHDWF of 18.4 mgd (current) and 23.4 mgd (2035), and comparing the all units online values in Table 3-11 to the Master Plan PHWWF of 37.1 mgd (current) and 46.7 mgd (2035), reveals that that that clarifiers currently do not have adequate capacity to reliably treat current peak flows when the SVI reaches approximately 200 mL/g. Moreover, as flows increase toward the 2035 conditions, the peak wet weather treatment capacity would not be adequate even under historic average SVI conditions of 170 mL/g.

3.2.3.3.2 RAS and Scum Pumping

There is a dedicated RAS pump for each of the secondary clarifiers. In addition, the RAS pumping facilities are configured so that the RAS pump associated with Clarifier 1 can also be used to pump RAS from Clarifier 2 (and vice versa), and the RAS pump associated with Clarifier 3 can also be used to pump RAS from Clarifier 4 (and vice versa). The RAS pumps are centrifugal pumps with variable drives. The pumps that service Clarifiers 1 and 2 each have a maximum capacity of 6.1 mgd and 60 HP, and the pumps that service Clarifiers 3 and 4 have a maximum capacity of 6.1 mgd and 8.5 mgd, with 60 HP and 125 HP motors, respectively. Each set of pumping facilities have a firm capacity of 6.1 mgd, for a combined firm capacity of 12.2 mgd and a total capacity of 26.8 mgd. The firm capacity represents a combined RAS flow that is approximately 50 percent of the 2035 maximum day flow provided in Chapter 2 (Wastewater Flow and Loadings), and should generally be adequate for the 2035 conditions.

Secondary Clarifiers 1, 2 and 4 are each serviced by a 5 HP, centrifugal scum pump, which have capacities of 150 gpm. Scum is pumped from these three clarifiers to the Inner-Sewer Pump Station wet well, or to the DAFT. Secondary Clarifier 3 contains a scum overflow trough where scum can flow via gravity to the plant Inner-Sewer Pump Station wet well, from where it is returned upstream of the grit tanks.

3.2.3.4 RAS Pipelines

RAS can be returned to the aeration basins via two pipelines, a 24-inch diameter pipeline and a 14-inch diameter pipeline. The locations of these pipelines are shown on the aerial layout on Figure 3-11. The 24-inch diameter line is the preferred line that is used for day to day operations, and the 14-inch line has generally been out of service for several years. The 24-inch diameter pipeline returns activated sludge to the RAS channel, while the 14-inch diameter pipeline returns activated sludge to the settle sewage channel.

Due to pipe integrity issues, the District has recently slip lined the 24-inch pipe with cured-in-place pipe (CIPP), which reduced the pipe diameter by 3 inches to 21 inches. Similarly, the District is planning to slip line the existing 14-inch pipe, which would reduce its diameter to 11-inches. An analysis to determine whether the reduced sized RAS pipelines could handle the projected 2035 RAS flows demonstrated the following:

- For day to day operations, the velocity in the 21-inch RAS pipe should be less than 5 feet per second (fps), which would equate to approximately 7.8 mgd of capacity. For day to day operations, RAS flows are typically set to 50 percent of the influent flow. At 50 percent of the 2035 annual average influent flow, the RAS flow would be 6.8 mgd. Therefore, the reduced-sized 21-inch pipeline should be adequate for day to day operations.
- At sustained maximum flow conditions, the velocity in the RAS pipeline should generally not exceed 7 fps. For the 21-inch line, the maximum flow that can be conveyed while maintaining this desired velocity is 10.9 mgd. The expected sustained maximum RAS flow condition is approximately 50 percent of the 2035 maximum 30-day flow, or 8.0 mgd. Therefore, the reduced-sized 21-inch pipeline should be adequate for sustained maximum flow operations.
- For short-term maximum flow conditions, the velocity in the pipeline should generally not exceed 10 fps. For the 21-inch line, the maximum flow that can be conveyed while maintaining this velocity is 15.5 mgd. The expected short-term maximum RAS flow conditions is approximately 50 percent of the 2035 maximum daily influent flow, or 12.9 mgd. Therefore, the reduced-sized 21-inch pipeline should be adequate for sustained maximum flow operations.
- With both a 21-inch and 11-inch diameter pipe in operation, the flow capacity while maintaining a 10 fps velocity would be 19.8 mgd. The firm capacity of the RAS pumping system is 18.3 mgd. Thus, two RAS pipelines should be adequate to accommodate any short-term peaks that might occur.

Based on this analysis, it is recommended that the District perform CIPP on both the existing RAS pipes and continue to rely on larger RAS pipe for day to day conditions. The existing 24-inch pipeline should be adequate under all analyzed flow conditions; however, the 11-inch pipe could accommodate larger flows that the anticipated maximum short-term flows.

3.2.4 Secondary Effluent Disinfection

Disinfection is accomplished through the use of sodium hypochlorite and consists of the following components:

- Chlorine contact tank
- Sodium hypochlorite storage and chemical metering pump system

3.2.4.1 Chlorine Contact Tank (CCT)

Secondary effluent enters the chlorine contact tank (CCT) downstream of the Microfiltration Transfer Pump Station and Facultative Sludge Lagoon Cap Water Pump Station, both of which can divert secondary effluent flow away from the chlorine contact tank year-round. Flow leaving the chlorine contact tank is either discharged to the LAVWMA system or to Holding Basin 4. In addition, effluent flows can be diverted to Holding Basin 1 in the event the effluent water quality is not adequate for surface water discharge.

The CCT consists of a conventional, two-pass contact tank. Each pass is 305 ft in length, 10 ft in width, and has a water depth at ADWF of 10.4 ft, for a total volume of 0.47 million gallons (Mgal). The volume of the CCT increases slightly by 0.05 Mgal, to a total of 0.52 Mgal, at peak flows when the CCT water level increases. The CCT was sized to provide 60 minutes of theoretical contact time at flows of 11.5 mgd, and 30 minutes of theoretical contact time at flows of 23 mgd.

As detailed in the *Coliform Limit Study* (Whitley Burchett and Associates, 2000), an additional 0.32 Mgal contact time is available in the 4,000 ft of 42- to 48-inch diameter pipeline between the CCT and the existing LAVWMA Junction Structure, which is located near the LAVWMA holding ponds. When added to the CCT volume, the total volume available for chlorine contact ranges from 0.79 Mgal at average flow conditions to about 0.84 Mgal at peak flow conditions.

The *Coliform Limit Study* also included the performance of tracer studies, which indicated that the CCT has a modal contact time measured at around 90 percent of the theoretical contact time, and that the pipeline to the LAVWMA junction structure has nearly ideal plug flow conditions, with modal contact times approaching 100 percent of the theoretical contact time.

Since the *Coliform Limit Study* was completed, the DERWA project was brought online, which included the installation of the Effluent Diversion Structure approximately 950 ft downstream of the CCT. During periods when the DERWA facilities are operated, flows are diverted from the LAVWMA pipeline into Holding Basin 4, and the stored disinfected secondary effluent is then pumped from Holding Basin 4 either to the DERWA facilities for tertiary treatment or back to the LAVWMA pipeline for discharge. Given the variable inflow/outflow conditions of the DERWA system, it would be difficult to include Holding Basin 4 in a predictable way to calculate the detention times. Therefore, assuming that only the length of discharge pipeline between the CCT and the Effluent Diversion Structure are available for added detention time, the combined volume of the CCT and the 42-inch discharge pipeline ranges from about 0.54 Mgal under average flows conditions to 0.59 Mgal at peak flow conditions.

As documented in the *Coliform Limit Study*, studies completed by others in the region have demonstrated that an average chlorine residual of 2 mg/L and a target CT value of 100 to 120 milligram-minutes per liter (mg-min/L) is needed to reliably achieve a 500 Most Probable Number of coliform per 100 Milliliter (MPN/100 mL) fecal coliform effluent limit. However, since the completion of the *Coliform Limits Study*, a maximum monthly mean enterococcus limit of 240 MPN/100 mL has been added to the District's permit. Although an evaluation of the CT needed to meet the enterococcus limit has not been performed, it is expected that the CT values required for fecal coliform removal are more than adequate because the maximum enterococcus concentrations measured in the WWTP effluent have been well below the prescribed limits.

Assuming a residual of 2 mg/L, the modal contact time in the CCT must be at least 50 to 60 minutes to achieve the recommended CT of 100 to 120 mg-min/L. The *Coliform Limit Study* did not provide a recommended residual or minimum CT for peak flows. Shorter contact times resulting from higher flows require higher chlorine dose concentrations to achieve the target CT value.

A summary of the theoretical and modal contact volumes for the CCT alone, and the CCT in conjunction with the downstream LAVWMA pipeline, is provided in Table 3-12. Also shown is the maximum allowable flow that can be treated given these volumes and the need to provide a modal contact time range of 50 to 60 minutes at average flows and 25 to 30 minutes at peak flows.

Table 3-12. Flows Corresponding to Modal Contact Times Ranging from 25 to 60 minutes

Facilities	Units	CCT Alone	CCT Plus Pipeline to the Effluent Diversion Structure	CCT Plus Pipeline to the LAVWMA Junction Structure
Average Flow Conditions				
Theoretical contact volume[a]	Mgal	0.47	0.54	0.79
Modal contact volume[b]	Mgal	0.42	0.48	0.71
Average flow that allows for a 60-minute modal contact time	mgd	10.2	11.6	17.1
Average flow that allows for a 50-minute modal contact time	mgd	12.2	14.0	20.5
Peak Flow Conditions				
Theoretical Contact Volume[c]	Mgal	0.52	0.59	0.84
Modal Contact Volume[b]	Mgal	0.47	0.53	0.76
Peak flow that allows for a 30-minute modal contact time	mgd	22.5	25.4	36.3
Peak flow that allows for a 25-minute modal contact time	mgd	27.0	30.5	43.6

[a] CCT water depth of 10.4 ft.
[b] Modal contact time to theoretical contact time ratio of 0.9 is assumed for both the CCT and the pipeline, which is slightly conservative for the pipeline based on the *Coliform Limit Study* (Whitley Burchett and Associates, 2000).
[c] CCT depth of 11.4 ft

As shown in Table 3-12, the volume provided by the CCT combined with the LAVWMA pipeline between the CCT and the Effluent Diversion Structure provides 60 minutes of modal contact time for flows of 11.6 mgd and 50 minutes of modal contact at flows of 14.0 mgd. With the 2035 average flows at 13.5 mgd, the recommended minimum CT of 100 to 120 mg-min/L can be reliably achieved under average flow conditions if a chlorine residual of 2 mg/L is met at Effluent Diversion Structure.

Table 3-12 also demonstrates that during the peak flow periods (when flows are less likely to be diverted into Holding Basin 4), the volume provided by the CCT and the LAVWMA pipeline between the CCT and the LAVWMA Junction Structure provides 25 minutes of modal contact time for flows up to 43.6 mgd, which exceeds the 2035 PHWWF condition of 42.7 mgd. Therefore, the recommended minimum CT of 100 to 120 mg-min/L can be reliably achieved under peak flow conditions if a chlorine residual of 2 mg/L is met at LAVWMA Junction Structure.

Currently, the District can monitor residual chlorine at the LAVWMA Junction Structure. However, this monitoring data is not currently used to verify chlorine dose requirements at the CCT are adequate to meet a 2 mg/L residual at the LAVWMA Junction Structure. Nevertheless, discussions with District staff indicate that effluent fecal coliform limits are reliably achieved even without the use of this added control point. If improved chlorination system control is needed in the future, the District would need to modify the chlorine dosing control strategy to rely on both feedback from the current residual monitoring equipment at the contactor, augmented by a chlorine analyzer located at the LAVWMA Junction Structure to provide chlorine residual concentration data. The first on-line analyzer's target chlorine residual should be adjusted to modify the current chlorine dose if the second on-line analyzer at the LAVWMA structure indicates that the current chlorine residual is approaching the required 2 mg/L residual. Similarly, a new secondary chlorine residual monitoring site at the Effluent Diversion Structure for use when effluent is first diverted to Holding Basin 4.

3.2.4.2 Sodium Hypochlorite Storage and Chemical Metering Pump System

The sodium hypochlorite storage facility consists of a covered open air structure with four hypochlorite storage tanks with a capacity of 7,600 gallons each, for a firm capacity of 22,800 gallons and a total capacity of 30,400 gallons. These tanks supply six chemical metering pumps. The three post-chlorination metering pumps, which are used to supply hypochlorite solution to the chlorine contact tank, have a rated capacity of 1,200 gpd each. Two RAS chlorination pumps, with a capacity of 200 gpd each are used optionally for RAS chlorination. One of these pumps is currently not operational. The sixth metering pump has a maximum capacity of 420 gpd, and is used to pump hypochlorite solution to the discharge of the Microfiltration Transfer Pump Station.

The post-chlorination pumps deliver sodium hypochlorite to a chamber just upstream of the chlorine contact tank. A motor driven propeller type chemical induction unit provides mixing and creates a vacuum assist to the post-chlorination pumps.

3.2.5 Secondary Effluent Pumping

There are two secondary effluent pump stations located at the WWTP: Final Effluent Pump Station No. 1 and Final Effluent Pump Station No. 2.

3.2.5.1 Final Effluent Pump Station No. 1

Final Effluent Pump Station No. 1 is located immediately downstream of the CCT and can pump flows either to the LAVWMA system or to Holding Basin 4. Final Effluent Pump Station No. 1 consists of three, 250 HP vertical turbine pumps that have variable speed drives. For pumping to Holding Basin 4, each pump has a rated maximum capacity of 25 mgd (at a design head of 40 ft). Thus, this pump station has a firm capacity of 50 mgd and an estimated total capacity of 75 mgd (based on the stated capacity of each pump). For pumping to LAVWMA, this pump station has a rated capacity of up to 35 mgd and an estimated total capacity of 53 mgd (based on an assumed capacity of 17.5 mgd for each pump).

The *Effluent Transport Facilities Basis of Design Report* (Brown and Caldwell, 2002) documents that treated effluent can also flow by gravity to Holding Basin 4 when flows are less than approximately 30 mgd and the water surface elevation in Holding Basin 4 is less than 320 ft. An updated hydraulic evaluation presented the *Secondary Effluent Wet Weather Capacity Evaluation* (RMC 2014) indicated that the peak gravity flow from the CCT to Holding Basin 4 (with 1 inch of free overflow at the secondary clarifier weirs) is approximately 32 mgd with a water surface elevation in Holding Basin 4 of 318 ft.

3.2.5.2 Final Effluent Pump Station No. 2

Final Effluent Pump Station No 2 pumps flow from Holding Basin 4 to the LAVWMA system. Final Effluent Pump Station No. 2 consists of five, vertical mixed flow pumps that are operated with variable speed drives. Two pumps are 75 HP and have a maximum capacity of 10 mgd with a design head of 23 ft, and the remaining three pumps are 250 HP and have a maximum capacity of 19 mgd with a design head of 50 ft. Thus, this pump station has a firm capacity (i.e. capacity with the largest unit out of service) of 58 mgd and a total capacity of 77 mgd (based on the stated capacity of each pump).

3.2.6 Hydraulic Capacity

The hydraulic profile completed as part of the Stage 4 Project documents that the hydraulic capacity through the treatment facilities (assuming three aerated grit tanks, four primary sedimentation basins, five aeration tanks and four secondary clarifiers) was adequate to convey flows up to 60.7 mgd (the equalized PHWWF defined under the Stage 4 Project). This capacity exceeds the project PHWWF of 46.7 mgd presented in this Master Plan.

However, as documented herein, a number of improvements have occurred since the Stage 4 Project was complete that have the potential to impact the hydraulic profile through the WWTP. District staff observed water levels encroaching on the top of the secondary clarifier weirs during peak flow events, suggesting that the conveyances between the secondary clarifiers and Effluent Pump Station No. 1 are not adequate to convey the current PHWWF, let alone the equalized PHWWF of 60.7 mgd defined for the Stage 4 Project.

An evaluation of the liquid treatment process hydraulic capacity between the secondary clarifiers and Effluent Pump Station No. 1 was recently completed, and documented in the *Secondary Effluent Wet Weather Capacity Evaluation* (RMC 2014). Based on the assessment presented in the 2014 report, the current maximum flow capacity downstream of the secondary clarifiers is estimated to be approximately 38 mgd.

3.3 FLOW EQUALIZATION/STORAGE

Four holding basins are available for storage of raw, primary, secondary and/or tertiary treated wastewater. Holding Basins 1, 2 and 3 can receive raw wastewater, primary effluent, and secondary treated flows. Holding Basin 4 is hydraulically isolated from the other three basins and can only receive secondary treated effluent. Holding Basin 4 is predominantly used to hold secondary treated effluent prior to its treatment in the DERWA facilities or its discharge to the LAVWMA system. (Note, Holding Basin 4 serves as the wet well for the Tertiary Influent Pump Station described in Section 3.5.1.1 and Effluent Pump Station No. 2 described in Section 3.2.5.2.) A schematic of the storage facilities is shown on Figure 3-4.

Water surface elevation (WSEL) versus volume curves for the four holding basins are shown on Figures 3-12 through 3-15. An overview of the holding basin operations and available storage capacity is provided in the following sections. A more detailed assessment of the storage needs and availability at the WWTP is provided in TM No. 1 Wastewater Storage Assessment, which is included at **Appendix C**.

3.3.1 Holding Basin Operations

Holding Basins 1, 2 and 3 are available for equalization of raw wastewater, screened wastewater, or primary effluent. All four of the basins are available for equalization of secondary effluent. Equalization is used at the Plant when either 1) the treatment or conveyance capacity of the facilities downstream of the Primary Sedimentation Basins is exceeded or impaired, or 2) flows exceed the allowable discharge capacity to the LAVWMA system. For the former, the preferred practice is equalization of primary effluent. For the latter, storage of secondary effluent is preferred. Finally, all four basins can be used for emergency storage of secondary effluent in the event that the treated effluent does not adequately meet the water quality requirements for surface water discharge. The available pathways for filling and draining each of the holding basins is described below.

3.3.1.1 Holding Basin 1

Flow may be directed to Holding Basin 1 from the following locations:

- Raw wastewater can be received from the influent diversion structure. Filling from this location would require backing up flows into the headworks, flooding the influent screening area and upstream sewer, and would only be practiced in the event of an issue with the Influent Pump Station.
- Primary effluent can be received from the primary effluent channels. From this location, Holding Basin 1 can be filled to its capacity (i.e., WSEL 326.3 ft).
- Secondary effluent (either chlorinated or unchlorinated) can be received from the chlorine contact tank. From this location, Holding Basin 1 can only be filled to a WSEL of approximately 320 ft.
- Flows from Holding Basin 2 via the storage distribution station located between these two basins. Other than receiving flow from Holding Basin 1, Holding Basin 2 can receive secondary effluent from Holding Basin 4 or raw/screened wastewater from Holding Basin 3. Holding Basin 1 can be filled to capacity from Holding Basin 2 if the WSEL in Holding Basin 2 is at 326.3 ft. However, there is a risk of Holding Basin 1 overflowing if being filled from Holding Basin 2 and the WSEL in Holding Basin 2 exceeds 326.3 ft.
- Thickener underflow can also be received from the DAFT (discussed in Section 3.4.2), although this is not a typical practice.

Wastewater held in Holding Basin 1 can be discharged back to the preliminary treatment facilities via the influent diversion structure or through Holding Basin 2 and the Pleasanton Junction Structure. If water is stored in Holding Basin 1 that meets the water quality requirements for either surface water discharge or diversion to the recycled water facilities, the stored water may be returned to the downstream end of the chlorine contact tank (and the outfall to LAVWMA) via the Basin Lift Station wet well.

3.3.1.2 Holding Basin 2

Flows can be directed to Holding Basin 2 via the following four pathways:

- Primary effluent can be diverted to Holding Basin 2 via the Holding Basin 1 Bypass Pipeline (see Figure 3-4). From this diversion point, Holding Basin 2 can be filled to capacity (assuming the hydraulic connection to Holding Basin 1 is removed).
- Flows from Holding Basin 1 via the storage distribution station. However, because the maximum WSEL of Holding Basin 1 is 326.3 ft, Holding Basin 2 will have a freeboard of approximately 0.8 ft when filled from this location.

- Flows from Holding Basin 3 via Flow Diversion Structure 2/3. As discussed further below, other than receiving flows from Holding Basin 2, Holding Basin 3 can receive either raw wastewater from the Dublin Trunk, or screened wastewater from downstream of the Influent Pump Station. Therefore, this pathway only provides for raw/screened wastewater storage, which is not a preferred practice. Holding Basin 2 can be filled to capacity from this location if Holding Basin 3 is at its maximum capacity, which can only occur for screened wastewater.
- Flows from Holding Basin 4 to Holding Basin 2 via the transfer pump located in Holding Basin 4. Because Holding Basin 4 can only be used to store secondary effluent and the sand filter reject flows, flow entering Holding Basin 2 from this location will always be treated through the secondary process. Holding Basin 2 can be filled to capacity from Holding Basin 4. (Although the capacity of the transfer pump station is unknown.)
- Finally, digested sludge from the anaerobic digesters may also be directed to Holding Basin 2 in an emergency condition. A hose can also be attached to the outlet of the pipeline that conveys sludge from the anaerobic digesters to Holding Basin 2 so the sludge can also be sent to a tanker truck.

Wastewater held in Holding Basin 2 can be discharged to Holding Basin 1, from where it can be directed to the preliminary treatment facilities or the LAVWMA outfall. Wastewater held in Holding Basin 2 can also be discharged to Holding Basin 3 from where it can be directed to the Dublin Trunk. Finally, wastewater in Basin 2 can also be sent directly to the preliminary treatment facilities via the Pleasanton Junction Structure.

3.3.1.3 Holding Basin 3

Flows can be directed to Holding Basin 3 via the following four pathways:

- Raw wastewater from the Dublin Trunk may be directed to Holding Basin 3 via the Holding Basin 3 Drainage/Diversion Structure. Filling of Holding Basin 3 from this location is limited by the available water surface elevation in the Dublin Trunk, and an assessment of this filling capacity is beyond the scope of this Master Plan.

  Note that the Stage 4 Improvements Project drawings indicate the top of two manholes upstream of the Drainage/Diversion Structure as being 328.50 ft and 328.96 ft. It is unclear, however, whether these two elevations are the control points for the upstream sewers. Presumably, the maximum WSEL would be well below these two elevations unless the plant was experiencing an emergency condition.
- Screened wastewater may be directed to Holding Basin 3 from the Raw Sewage Distribution Channel downstream of the Influent Pump Station and upstream of the grit tanks. From this location, Holding Basin 3 can be able to be filled to its maximum WSEL.

- Flows from Holding Basin 2 via Flow Diversion Structure 2/3. As noted above, other than receiving flow from Holding Basin 3, Holding Basin 2 may be filled with from Holding Basin 1 (raw wastewater, primary effluent, or secondary effluent), from the Holding Basin 1 Bypass Pipeline (primary effluent), or from Holding Basin 4 (secondary effluent). If being filled from Holding Basin 1, the maximum WSEL in Holding Basin 2 is 236.3 ft. Holding Basin 2 may be filled to its maximum WSEL of 237.1 ft if filled from the Holding Basin 1 Bypass Pipeline or from Holding Basin 4. Because the maximum WSEL in Holding Basin 3 is 328.3 ft, the storage capacity in Holding Basin 3 when being filled from Holding Basin 2 is limited to the capacity available with a 1.2 ft freeboard. If Holding Basin 2 has only been filled from Holding Basin 1, the storage capacity in Holding Basin 3 when being filled from Holding Basin 2 is limited to the capacity available with a 2 ft freeboard.

Water held in Holding Basin 3 can be discharged to Holding Basin 2 or through the Dublin Trunk via the Holding Basin 3 Drainage/Diversion Structure.

3.3.1.4 Holding Basin 4

Secondary effluent either flows by gravity or is pumped to Holding Basin 4 from the chlorine contact tank effluent channel. As presented in the *Secondary Effluent Wet Weather Capacity Evaluation* report (RMC 2014), flows up to approximately 32 mgd can be conveyed by gravity to Holding Basin 4, when the WSEL in Holding Basin 4 is less than 318 ft. Above 32 mgd, or if Holding Basin 4 WSEL is above 318 ft, secondary effluent is pumped from Final Effluent Pump Station No. 1 to Holding Basin 4. Holding Basin 4 can be filled to capacity if flows are pumped to the basin. The maximum WSEL in Holding Basin 4 is 329.9 ft.

Off-spec water from the sand filters UV system effluent channel, reject water from the sand filters reject water pump station, off-spec water from sand filters filter effluent channel, and overflows from the rapid mix and flocculation basin bypass channel can also be discharged to Holding Basin 4.

Water held in Holding Basin 4 can either be pumped to the sand filtration facilities via the Tertiary Influent Pump Station, or the LAVWMA system via Final Effluent Pump Station No. 2. There is also a transfer pump that is available to convey flows from Holding Basin 4 to Holding Basin 2. The District installed this pump facility but does not have records on pump capacity or type.

3.3.2 Holding Basin Storage Capacity

As noted above, the available storage capacity of the holding basins varies for each type of stored flow, as follows:

- Raw Wastewater: Raw Wastewater can be diverted to Holding Basin 1 via the Influent Diversion Structure or to Holding Basin 3 from the Dublin Trunk. Diversion from either of these locations is limited by how much flow can be backed up into the upstream facilities. For purposes of this analysis it is assumed that the maximum filling volume is limited by the maximum elevation of Holding Basin 1. Therefore, raw wastewater storage capacity is maximized when Holding Basin 1 has no freeboard, Holding Basin 2 has a 0.8 ft. freeboard, and Holding Basin 3 has a 2 ft freeboard.

  However, it should be noted that the influent screening area would be completely flooded if Holding Basin 1 was filled to its maximum WSEL. Therefore, diversion of raw wastewater should only occur in emergency situations.

- Screened Wastewater: Screened wastewater can be diverted to Holding Basin 3. From this diversion point, Holding Basin 3 can be filled to capacity. From Holding Basin 3, Holding Basin 2 can be filled to capacity. However, if filling Holding Basin 1, the maximum WSEL in Holding Basin 2 cannot exceed the maximum WSEL of Holding Basin 1. Therefore, screened wastewater storage capacity is maximized when Holding Basin 1 has no freeboard, Holding Basin 2 has a 0.8 ft. freeboard, and Holding Basin 3 has no freeboard.

- Primary Effluent: Primary effluent can be diverted to Holding Basin 1 and Holding Basin 2. Both of these basins can be filled to capacity from the diversion location. Holding Basins 3 can be filled up to the maximum WSEL of Holding Basin 2. Therefore, primary effluent storage capacity is maximized when Holding Basin 1 has no freeboard, Holding Basin 2 has no freeboard, and Holding Basin 3 has a 1.2 ft freeboard.

- Secondary Effluent: Holding Basin 4 can be filled to capacity with secondary effluent via Effluent Pump Station No. 1. Holding Basin 2 can be filled to capacity from Holding Basin 4 via the transfer pump discussed above. However, if filling Holding Basin 1, Holding Basin 2 WSEL cannot exceed 326.3 ft. The maximum WSEL at the CCT diversion point to Holding Basins 1 is approximately 323 ft. Therefore, secondary effluent storage capacity is maximized when Holding Basin 4 has no freeboard, Holding Basin 1 has no freeboard, Holding Basin 2 has a 0.8 ft. freeboard, and Holding Basin 3 has a 1.2 ft freeboard.

Table 3-13 summarizes the available storage capacities for the types of flow discussed above assuming the minimum amount of freeboard is maintained as possible. However, in practice, District staff will likely operate the storage facilities with some freeboard. As demonstrated in Table 3-13, it is not possible to operate Basins 1, 2 and 3 with the same freeboard because they are hydraulically connected. However, it is possible to operate the basin that controls the maximum water surface elevation with a minimum freeboard. For purposes of this Master Plan, it is assumed that the District would likely operate the controlling basin water level with somewhere between a half a foot, and a foot of freeboard. Table 3-13 also summarizes the available storage capacities for the types of flow discussed above assuming that the pond that controls the water surface elevation has at least one foot of freeboard. The actual storage capacity available for each water source therefore is somewhere between the two values shown in Table 3-13.

Table 3-13. Holding Basin Storage Capacities for Different Types of Flow

	Raw Wastewater		Screened Wastewater		Primary Effluent		Secondary Effluent	
	Freeboard ft	Storage, Mgal	Freeboard ft	Storage, Mgal	Freeboard ft	Storage, Mgal	Freeboard ft	Storage, Mgal
Operating with Minimum Freeboard Possible								
Basin 1	0	2.3	0	2.3	0	2.3	0	2.3
Basin 2	0.8	4.8	0.8	4.8	0	5.2	0.8	4.8
Basin 3	2.0	5.6	0	6.8	1.2	6.1	1.2	6.1
Basin 4	Not Used		Not Used		Not Used		0	7.2
Total Storage, Mgal	**12.7**		**13.9**		**13.6**		**20.4**	
Operating with Minimum of One Foot Freeboard								
Basin 1	1	2.0	1	2.0	1	2.0	1	2.0
Basin 2	1.8	4.4	1.8	4.4	1	4.7	1.8	4.4
Basin 3	3.0	5.0	1	6.2	2.2	5.5	2.2	5.5
Basin 4	Not Used		Not Used		Not Used		1	6.7
Total Storage, Mgal	**11.4**		**12.6**		**12.2**		**18.6**	

3.4 SOLIDS TREATMENT AND DISPOSAL FACILITIES

This section presents an overview of the WWTP's solids treatment facilities under the following headings:

- Plant-Wide Solids Balance
- Solids Thickening
- Anaerobic Digestion
- Facultative Sludge Lagoons
- FSL Harvesting System
- Dedicated Land Disposal

3.4.1 Plant-Wide Solids Balance

A plant-wide solids balance was developed to estimate the mass of solids (measured as TSS, and volatile suspended solids, or VSS) that are generated under the following four WWTP operating conditions:

- Current Average
- Current Maximum 30-Day
- 2035 Average
- 2035 Maximum 30-Day

These solids balances are provided in Figures 3-16 through 3-19. Table 3-14 provides a summary of the pertinent TSS and VSS loading data for each of the solids treatment process areas.

Table 3-14. Predicted Solids Loading Rates, lb/day

Influent Conditions	DAFT		Anaerobic Digesters		Facultative Sludge Lagoons		Dedicated Land Disposal	
	TSS	VSS	TSS	VSS	TSS	VSS	TSS	VSS
Current								
Average	30,200	23,300	28,500	22,000	13,800	7,300	9,600	3,500
Maximum 30-Day	35,300	27,700	33,300	26,100	15,800	8,600	10,900	4,100
2035								
Average	38,700	29,900	36,500	28,200	17,600	9,300	12,300	4,400
Maximum 30-day	45,300	35,400	42,700	33,400	20,300	11,000	14,100	5,200

3.4.2 Solids Thickening

Primary and secondary solids are combined in a common line and thickened together in the DAFT. The DAFT has a diameter of 65 ft, with a tank water surface diameter of 60 ft, and a side water depth of 8 ft. A summary of the capacity of the existing DAFT, based on typical design values, is shown on Table 3-15. As shown, the DAFT should be capable of processing the full range of solids loadings conditions that were presented in Table 3-14.

Table 3-15. DAFT Allowable Solids Loading Rate at Typical Design Conditions

Design Parameter	Number of Units	Typical Design Range[(a)]	Selected Design Value	Theoretical Allowable Loading Rate, lb/day
Loading Rates	1	0.6-0.8 lb/sq-ft-hr	0.7 lb/sq-ft-hr	47,500

(a) Metcalf and Eddy/AECOM, 2014.

Two DAFT pressurization pumps, which are 125 HP and 150 HP centrifugal pumps with a capacity of 1,600 gpm each, deliver air-enriched flow to the DAFT. The DAFT is equipped with a double disc bottom sludge pump that is used to intermittently pump sludge that has settled to the bottom of the DAFT to the aerated grit tanks. The bottom sludge pump is a 25 HP centrifugal pump with a capacity of 100 gpm. The DAFT is also equipped with two, 25 HP progressive cavity thickened sludge pumps with a capacity of 200 gpm that pump thickened sludge to the anaerobic digesters.

The District also has a trailer mounted gravity belt thickener (GBT) that is available for thickening when the DAFT is offline. The GBT has a 1.5-meter belt, and is sized to accommodate up to 300 gpm, or 0.4 mgd, at 0.3 to 0.6 percent solids. To maintain an MCRT of 2.5 days, the wasting rate should be at least 1.5 mgd.

3.4.3 Anaerobic Digestion

Three mesophilic, high-rate anaerobic digesters are located at the WWTP. A summary of the size and capacity of these three digesters is provided in Table 3-16. The firm capacity of the digesters needs to be adequate to handle the anticipated maximum 30-day solids generation rates. The firm capacity of the digesters is 23,200 lb VSS/day. In addition, given an average solids content in the thickened solids discharged to the digesters of 4.5 percent and the firm hydraulic capacity of 77,300 gallons presented in Table 3-16, the allowable total solids loading rate is approximately 26,000 lb/day.

The current maximum 30-day solids loadings rates are shown in Table 3-14. The current maximum 30-day VSS loading is 26,100 lb/day while the corresponding solids loading rate is 33,300 lb/day. It is apparent that the existing digesters are not adequately sized to accommodate the current maximum 30-day loading rates. This problem is exacerbated when comparing the firm capacity of the digesters to the 2035 projected maximum 30-day values. The projected 2035 maximum 30-day VSS generation rate is 33,400 lb/day and the solids loading rate is 42,700 lb/day (as shown in Table 3-14). Both of these values are significantly more than the current firm capacity.

Table 3-16. Anaerobic Digester Volatile Solids Loading Capacities at Original Design Condition

Facility	Diameter, ft	Depth, ft	Volume, cubic ft	Solids Loading Rate, lb VSS/ 1,000 cu-ft-day[a]		Capacity, lb VSS/ day[b]	Minimum Retention Time, day		Capacity, gal/day[c]
				Typical Design Range	Selected Value		Typical Design Range	Selected Value	
Digester 1	55	32	77,400	100-300	150	11,600	15-20	15	38,600
Digester 2	55	31	77,600			11,600			38,700
Digester 3	70	33	138,200			20,700			68,900
Total Capacity			**293,200**			**43,900**			**146,200**
Firm Capacity			**155,000**			**23,200**			**77,300**

[a] Pounds of VSS per 1,000 cubic feet per day.
[b] Pounds of VSS per day.
[c] Gallons per day.

Each anaerobic digester has mechanical mixing systems. Anaerobic Digesters 1 and 2 each have a single draft tube mixer each with a capacity of 12,500 gpm. Anaerobic Digester 3 has three mixers with a capacity of 7,200 gpm each for a total capacity of 21,600 gpm and a firm capacity of 14,400 gpm. Each anaerobic digester also has a centrifugal 15 HP circulating sludge pump with a capacity of 600 gpm each.

Each anaerobic digester has a heat exchanger. Anaerobic Digester 1 and 2 exchangers can provide up to 0.77 million British Thermal Units per hour (MM BTU/hr) (each) and the Anaerobic Digester 3 exchanger can provide up to 1.24 MM BTU/hr, for a total of 2.78 MM BTU/hr. The heating demand anticipated for the 2035 maximum 30-day sludge production rate is estimated to be 3.23 MM BTU/hr. Therefore, additional heat exchange capacity is needed to meet the 2035 heat demand.

Digested sludge from the anaerobic digesters is pumped by three, 25 HP, facultative sludge pumps, which are equipped with variable frequency drives, to the Facultative Sludge Lagoons. Two of the three pumps are progressing cavity type and the remaining pump is a recessed impeller type. Each pump has a 200 gpm capacity, for a total capacity of 600 gpm and a firm capacity of 400 gpm.

3.4.4 Facultative Sludge Lagoons (FSLs)

There are six FSLs. Lagoons 1, 2, 4, and 6 have a total surface area of approximately 3 acres and Lagoons 3 and 7 have a total surface area of approximately 4 acres. A summary of the surface area and volumes of each lagoon based on a four-foot water cap and three-foot freeboard is shown in Table 3-17. Also shown are the lagoons' capacities based on typical design total solids loading rates (which are based on total volume) and the capacities based on the District-defined design VSS loading (which are based on surface area).

Table 3-17. Facultative Sludge Lagoon Volatile Solids Loading Capacity

FSL No.	Area[a], acres	Volume[b], 1,000 cu-ft	Typical Solids Loading Design Criteria Range[c], lb/cu-ft-year[d]	Selected Value, lb/cu-ft-year[d]	Capacity, lb TSS/day	Design VSS Loading Rate[e], lb VSS/1,000 sq-ft-day[f]	Capacity lb VSS/day
1	3.3	1,000	2.2 - 2.4	2.3	6,400	15	2,200
2	2.9	900			5,600		1,900
3	3.8	1,200			7,600		2,500
4	3.3	1,000			6,400		2,200
6	3.3	800			5,000		1,700
7	4.2	1,200			7,400		2,800
Total Capacity	**20.8**	**6,100**			**38,400**		**13,300**
Firm Capacity	**16.6**	**4,900**			**30,800**		**10,500**

[a] Surface area below water cap and freeboard.
[b] Assumes all basins have an average depth of 15 ft.
[c] Metcalf and Eddy/AECOM, 2014.
[d] Pounds per cubic feet per year.
[e] Recommended loading rates range from 20 to 50 lb VSS/1,000 sq-ft-day. However, to minimize odor generation potential, the District relies on a loading rate of 15 lb VSS/1,000 sq-ft-day as a basis of design.
[f] Pounds per thousand square feet per day.

Comparing the firm TSS loading capacity values provided in Table 3-17 to the calculated solids generation rates (17,600 lb of TSS at the 2035 average and 20,300 lb of TSS at the 2035 maximum 30-day condition), the FSLs will be under loaded for TSS at the 2035 conditions. However, comparing the firm VSS loading capacity values provided in Table 3-17 to the calculated solids generation rates (9,300 lb of VSS at the 2035 average and 11,000 lb of VSS at the 2035 maximum 30-day condition), the FSLs are expected to be loaded near their rated capacity by 2035.

Cap water is provided to the FSLs from the Facultative Lagoon Cap Water Pump Station. This station consists of two centrifugal 75 and 10 HP pumps with a capacity of 1,000 GPM and 500 GPM, respectively.

Each facultative sludge lagoon has a 5 HP brush aerator, which is intended to help keep the water cap that sits on top of the digested solids aerobic (for odor control purposes).

3.4.5 FSL Harvesting System

Biosolids are harvested annually from the facultative sludge lagoons using a dredge. One lagoon is harvested every summer, allowing five years for the solids to stabilize between harvesting periods. Operating data for the harvesting period indicates that the dredge is operated at an average flowrate of approximately 350 gpm and up to 470 gpm. Harvested biosolids are discharged to the harvested sludge pipeline and conveyed to the DLD site for disposal via injection.

3.4.6 Dedicated Land Disposal Site

Biosolids from the FSLs are injected into the soil on the DLD site, which is a 46-acre area used for biosolids injection. The original design capacity of the DLD was established at 100 dry tons per acre per year (dt/ac-yr), assuming the biosolids are 4 to 6 percent solids by weight. However, a review of the District's DLD operations data indicates that the average solids concentration is about half this amount, and averages approximately 2.7 percent. In addition, discussions with District staff indicate that the DLD is often too saturated to continue applications near the end of the injection season. Thus, it appears the capacity of the DLD site is limited by the water content of the applied biosolids.

An analysis of the historic operations was completed to assess the DLD capacity and determine if solids are accumulating in the FSLs. For this analysis, the following three VS reduction conditions were evaluated:

- **53 Percent Reduction**: Historic VS reduction data collected by the District. This case represents the best estimate of current operations.
- **45 Percent Reduction:** Low end of literature values for VS reduction. This case represents a worst-case scenario with respect to accumulation in the FSLs.
- **64 Percent Reduction:** Represents the volatile reduction that would be needed to achieve zero accumulation.

The results of this evaluation are provided in Table 3-18. As shown, the evaluation completed indicates that approximately 1,210 dry tons per year (dt/yr) of solids have been applied to the DLD in the last five years. Therefore, this is the estimated capacity of the DLD site. This results in a capacity of approximately 6,600 lb/day, which is significantly less than the 2035 annual average generation rate of 12,300 lb/day defined through the mass balance (see Table 3-14).

Table 3-18. Solids Accumulation within the FSLs

Year	Total Annual Solids to FSLs[a], dt/yr	Total Annual Volatile Solids to FSLs[a], dt/yr	Total Solids Reduced in FSLs, dt/yr	Total Solids Applied to DLD[b], dt/yr	Accumulated Solids[c], dt/yr		
					Historic VS Reduction Data	Low-End of Literature Values	No Accumulation
2009	1,981	1,400	742	1,006	233	345	80
2010	2,318	1,624	861	1,303	154	284	(23)
2011	2,327	1,615	856	1,613	(141)	(12)	(317)
2012	2,613	1,849	980	1,062	571	719	370
2013	2,770	1,967	1,043	1,071	657	814	442
2014	2,599	1,839	975	1,180	(351)	(204)	(552)
Average	**2,430**	**1,720**	**910**	**1,210**	**190**	**320**	**0**

[a] Total annual volatile solids applied to FSLs calculated from total digested sludge flow to FSLs (gal), average digested sludge percent solids, and average digested sludge percent volatile solids data obtained from the District.

[b] Total annual solids applied to DLD site were obtained from the District's Annual Biosolids Monitoring Reports.

[c] Accumulated solids are calculated by subtracting the total solids reduced in the FSLs and the total solids applied to the DLD from the total solids in the FSLs.

The analysis provided in Table 3-18 also shows that the FSLs are accumulating solids at an average rate of approximately 190 dt/yr. However, this value is based on limited historic VS reduction data for the FSLs. Therefore, the accuracy of this analysis may need to be adjusted once more VS reduction data is available. Nevertheless, for the analysis presented in this Mater Plan, this condition is assumed to be an accurate depiction of the FSL solids accumulation rates.

3.5 RECYCLED WATER TREATMENT FACILITIES

This section presents an overview of the WWTP's recycled water treatment facilities under the following headings:

- Sand Filtration with UV Disinfection (SF/UV)
- Microfiltration with UV Disinfection (MF/UV)
- Recycled Water Pump Station R1
- MF Product Water Pump Station

The capacity requirements for the WWTP recycled water facilities are driven by increasing recycled water demands, which are currently being evaluated under a separate effort. Therefore, capacity requirements for the recycled water system are not addressed as part of this Master Plan.

3.5.1 Sand Filtration with UV Disinfection (SF/UV)

The SF/UV system was installed as part of the DERWA project in 2003 and consists of the following components:

- Tertiary Influent Pumping
- Band Screening
- Chemical Addition/Flocculation
- Sand Filtration
- UV Disinfection

The District is in the process of constructing a ballasted flocculation process, which along with a new band screen facility, will provide pretreatment of the secondary effluent flow upstream of the tertiary filters. The new band screen will have a production capacity of 22.2 mgd, and the ballasted flocculation process will have a production capacity of 18.0 mgd. These facilities are expected to be online by Spring 2018. The existing filtration pretreatment system discussed below will be retained to serve as a back up to the new ballasted flocculation process.

3.5.1.1 Tertiary Influent Pumping

There are three tertiary influent pumps, including two duty pumps and one standby. These 50 HP pumps are vertical propeller mixed-flow type pumps with rated capacity of 6.2 mgd, for a firm capacity of 12.3 mgd and a total capacity of 18.5 mgd. The Tertiary Influent Pump Station has space for two additional pumps.

The District will expand the tertiary influent pump station as part of the ballasted flocculation project that is coming online in Spring 2018. With this improvement, the pump station will be equipped with four, 50 hp pumps, each with a capacity of 6.9 mgd. In addition, the pump station will be equipped with one, 20 hp pump that has a capacity of 2.0 mgd. The 20 hp pump will only run by itself and not at the same time as the 50 hp pumps. Therefore, following this project the firm capacity of the tertiary influent pump station will be 20.7 mgd, and the total capacity will be 27.6 mgd.

3.5.1.2 Band Screening

Tertiary influent travels from the tertiary influent pumps to the band screen facility, which was designed to remove fruit labels and other items which can pass through the sand filters and clog distribution system strainers. The band screen has 2 mm openings. The District staff tested the band screen shortly after it was installed and determined that the system headloss was acceptable with flows of up to 12 mgd.

3.5.1.3 Chemical Addition / Flocculation

Two rapid mix tanks are used for coagulant addition with a volume of approximately 950 gallons each. The two tanks can operate in parallel, but only one basin is in service at a time. Each tank is serviced by a 20 HP mixer with a maximum mixer velocity gradient of 1,000 feet per second per foot (ft/sec per ft). The coagulant is stored in one of two storage tanks with a capacity of 4,500 gallons each. Currently, the coagulant being used consists of a 50 percent aluminum sulfate solution of polyaluminum chloride. The coagulant is pumped to the rapid mix tanks by one of two diaphragm metering pumps with a capacity of 17 gallons per hour (gph). A summary of the rapid mix tank hydraulic loading capacities compared to typical design criteria are presented in Table 3-19. As shown, the facility detention time is slightly lower than recommended values.

Table 3-19. Rapid Mix Tank Hydraulic Loading Capacities at Original and Selected Design Conditions

Loading Condition	Number of Units	Typical Design Range, sec[(a)(b)]	Selected Design Value, sec[(b)]	Firm Capacity, mgd	Total Capacity, mgd
Detention Time	2	5-30	17.5	4.7	9.3

(a) Metcalf and Eddy/AECOM, 2014.
(b) Seconds.

Effluent from the rapid mix tanks then travels to two flocculation basins with an approximate volume of 18,750 gallons each, which can be operated singly, in series, or bypassed. Each tank is serviced by a 2 HP flocculator with a maximum flocculator velocity gradient of 100/sec. Liquid polymer is added to the flocculation basins to help bring neutralized particles together to form larger particles. The polymer is stored in two 250-gallon storage totes for a firm capacity of 250 gallons and a total capacity of 500 gallons. Polymer is pumped to the flocculation basins through one of two diaphragm feed pumps with a capacity of 1 gph.

3.5.1.4 Sand Filtration

The sand filtration facility consists of continuous backwash, upflow deep bed filters with monomedia sand. There are six filters cells and each filter cell has six individual filtration modules with wash chambers which are used to continuously clean the sand. Each cell has a filtration area of 300 square feet (sq-ft) for a total of 1,800 sq-ft of filtration area. The media depth is 80 inches resulting in a total media volume of 2,000 cu-ft per cell and 120,000 cu-ft overall.

The filters were designed for a loading rate of 4.7 gpm per square foot (gpm/sq-ft), including a 3.3 percent backwash flow through the filters. However, the filters have generally been operated between 2 and 3 gpm/sq-ft, and have not been operated at higher loading rates except for short periods of time. The District recently decided that a loading rate of 3.4 gpm/sq-ft, which includes a 10 percent backwash flows through the filters, would be used for facility planning purposes. This results in a total capacity of 7.9 mgd and a firm capacity of 6.6 mgd. A summary of the sand filter capacities is presented in Table 3-20.

Table 3-20. Sand Filter Hydraulic Loading Capacities

Loading Condition	Number of Units	Design Hydraulic Loading Rate, gpm/sq-ft	Firm Capacity, mgd	Total Capacity, mgd
Original Design	6	4.7	9.1	11.0
Current Planning Value		3.4	6.6	7.9

With the new ballasted flocculation system installed, it is expected that the filters will be able to process flows at a hydraulic loading rates that meet, and likely exceed, the original design value shown in Table 3-20. Testing, along with confirmation from the State Water Board Division of Drinking Water, will be required following the completion of this project to confirm the allowable filtration system loading. However, loading rates of 7 gpm/sq-ft ore higher are likely achievable, putting the entire system capacity at approximately 16 mgd (or more).

Filter reject water is pumped to the plant influent sewers via two, 10 HP submersible pumps that have a 350 gpm capacity at a total dynamic head of 20 ft. Reject water flows may also be directed to Holding Basin 4.

3.5.1.5 UV Disinfection

Filtered water from the sand filters flows to a dedicated UV disinfection system. The system consists of two UV disinfection channels, where each channel contains five banks of lamps (four duty and one standby). Each bank consists of six modules and there are 20 lamps per module, for a total of 1,200 lamps. Each channel is designed with removable walls so that two additional modules can be added to each bank to increase the total number of lamps to 1,600.

The UV system is operated at a standard flow-pace control mode, where the number of lamps that is activated at any time is adjusted based on flow and UV Transmittance (UVT). The existing system has a permitted capacity of 9.7 mgd based on the bioassay that was conducted in 2006 (Carollo, 2015). In accordance with the specifications for a UV system that follows a sand filtration process provided in the *Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, Third Edition (NWRI,* August 2012), hereinafter referred to as the UV Guidelines, the UV system is designed to provide a minimum UV dose of 100 millijoules per square centimeters (mJ/cm^2) at the 9.7 mgd design flow when the UVT is at 55 percent.

A new bioassay is currently being conducted for the UV system to determine system capacity based on some system changes that have been made. The revised system rating will reflect a revised bioassay approach that is detailed in the UV Guidelines.

3.5.2 Microfiltration with UV Disinfection

The Microfiltration with UV Disinfection (MF/UV) system has a rated capacity of 3 mgd and is used during low demand periods in the winter and as backup for the SF/UV system. The MF/UV system components are as follows:

- Microfiltration Transfer Pump Station
- Strainers
- Microfiltration Feed Pumps
- Microfiltration
- UV Disinfection

3.5.2.1 Microfiltration Transfer Pump Station

The Microfiltration Transfer Pump Station consist of two, 40 HP centrifugal pumps, each with a capacity of 3.7 mgd. Sodium hypochlorite is occasionally added downstream of the transfer pumps to minimize biofouling on the MF filters.

3.5.2.2 Strainers

Two mechanical strainers with a screen size of 500 microns strain out larger debris prior to the microfilters to prevent damage to the filter cartridges.

3.5.2.3 Microfiltration Feed Pumps

After the strainers, effluent is pumped to the microfiltration units by four, 40 HP microfiltration feed pumps, which have a capacity of 1.7 mgd. This provides a firm capacity of 5.1 mgd and a total capacity of 6.9 mgd for the pump station.

3.5.2.4 Microfiltration

Five microfiltration units are provided that have a total design capacity of 3.75 mgd, with a net capacity of 3.35 mgd with an 89 percent recovery. Each unit houses 90 cartridges, and each cartridge contains 18,000, 0.5 millimeter diameter with 0.2 millimeter pore size, hollow fiber membranes.

3.5.2.5 UV Disinfection

Filtered water from the microfiltration system flows to a dedicated UV disinfection system separate from the UV system dedicated to the sand filtration system. The system consists of one UV disinfection channel, with ten modules (nine duty and one standby) and 40 lamps per module for a total of 400 lamps. The lamps are low pressure, low output. The system has a stated capacity of 3 mgd. In accordance with the specifications for a UV system that follows a membrane filtration process provided in the UV Guidelines, the UV system is designed to provide a minimum UV dose of 80 mJ/cm^2 at the 3.0 mgd design flow when the UVT is at 65 percent.

The UV system has several control modes: a standard flow-paced control mode (where the number of banks are controlled by flow and measured UVT); manual mode; and a third "flow control" option that is based on flow alone. The "flow control" mode assumes the UVT is always at 65 percent, and adjusts the number of banks operating based on flow alone. In addition, this operating mode relies on the use of a redundant bank that is on at all times so that failure of a bank will not lead to a reduction in dose below the required limit. The "flow control" mode is the primary mode that is utilized for operation of this facility.

3.5.3 Recycled Water Pump Station R1

Effluent from both tertiary treatment systems are pumped to the recycled water distribution system via the Recycled Water Pump Station R1. Recycled Water Pump Station R1 consists of three 450 HP vertical turbine pumps, each with a capacity of 4.9 mgd. The pump station has a nominal firm capacity of 9.7 mgd and a total capacity of 14.6 mgd. The pump station has space for three additional 450 HP pumps.

As part of the ballasted flocculation project that will come online in Spring 2018, the District is installing two additional recycled water pumps, bringing the Recycled Water Pump Station R1 firm capacity to 19.5 mgd and the total capacity to 24.5 mgd.

To reduce surge pressure changes on pump start-up and pump shut down, an air chamber surge tank is provided at the discharge manifold to dampen surge pressures. The surge control system consists of a 5,000-gallon surge tank with two air compressors at a capacity of 20 cfm.

Sodium hypochlorite is fed into the wet well by one of 2 vacuum ejector chlorinators with a capacity of 440 gpd each. The sodium hypochlorite is stored in one 6,000-gallon tank.

3.5.4 MF Product Water Pump Station

There is a Product Water Pump Station that was originally constructed to pump RO product water from the treatment facilities to the designated groundwater injection site. The Product Water Pump Station includes three, 60 HP vertical turbine pumps. Each pump has a capacity of 1.2 mgd, for a firm capacity of 2.4 mgd and a total capacity of 3.6 mgd. Because the Clean Water Revival Project was never implemented, the Product Water Pump Station has never been used for its intended purpose. Instead, flows from the MF/UV system are directed to the Recycled Water Pump Station R1 for distribution to the irrigation customers.

3.6 ANCILLARY FACILITIES

Ancillary facilities are the facilities located at the Plant site that are not directly used for treatment purposes, but instead support the treatment functions. This section addresses the following ancillary facilities:

- Odor Control Facilities
- Ferric Chloride System
- Plant Water Systems
- Plant Air Systems
- Cogeneration System

Additional ancillary facilities that are not addressed in this WWTP Master Plan include: the electrical and instrumentation/control systems, buildings and storage facilities (and associated functions), the plant air systems, fire protection, site lighting, site access and security, site drainage and landscaping, and parking and plant ingress/egress. The Plant's instrumentation/control system facilities were evaluated under a separate SCADA Master Planning effort.

3.6.1 Odor Control Facilities

The odor control system is composed of three processes:

- One Odor Reduction Tower
- Two Soil Bed Biofilters

An overview of these existing facilities is provided below. A more detailed evaluation of the existing odor control facilities and recommendations for future improvements is presented in a technical memorandum that was prepared by Environmental Management Consulting in support of this Master Plan. This memorandum is provided in **Appendix D**.

3.6.1.1 Odor Reduction Tower (ORT)

The odor reduction tower (ORT) is used to reduce odors generated in the influent pumping room, the old bar screen channels, the grit building, the influent diversion structure, and the aerated grit tanks. The ORT functions as a trickling filter type treatment system with plastic media for biofilm attachment. The ORT foul air collection system was rehabilitated in 2008 to improve odor capture and to reduce the foul air flow from 21,000 cfm before the project to 13,000 cfm after the project. This project reduced fan horsepower by 50 percent and improved hydrogen sulfide removal from 64 percent to 96 percent.

The ORT interior dimensions are 17.3 ft long by 12 ft wide for a cross sectional area of 208 sq-ft. The plastic media depth is 20 ft for a packed volume of 4,160 cu-ft. Unchlorinated secondary effluent is pumped via two, 10 HP centrifugal pumps that are located at the southwest end of the primary sedimentation basins to the top of the media to keep the biofilm wet. Each pump has a capacity of 380 gpm, for a total flow rate of 760 gpm. One pump serves as a redundant pump, so the resulting downward liquid velocity through the media is 1.3 fpm.

Foul air enters the bottom of the ORT and is scrubbed as it passes upward past biofilm coated media. Treated air is discharged out the top of the ORT. Current total foul air flow rate is 13,000 cfm with an empty bed residence time of 19.2 seconds. A masking odorant (or reodorant) system sprays chemical into the treated air outlets on top of the ORT to diminish residual odors.

3.6.1.2 Soil Bed Biofilters

There are two soil bed biofilters. One is used to reduce odors generated in the headworks screenings conveyors and screenings compaction and storage room, and the other is used to reduce odors generated in the DAFT.

The headworks biofilter has three beds for a total surface area of 10,000 sq-ft and a media depth of 4 ft for a total volume of 40,000 cu-ft. Fans draw air from the screenings conveyors, compactor and screenings storage room via foul air ducting and discharge beneath the biofilter. Foul air is treated by bacteria as it passes through the media. The fan air flow rate at design is 15,000 cfm, and a 15,000 cfm air passing through 40,000 cu-ft results in an Empty Bed Residence Time (EBRT) of 160 seconds. Actual contact time with the bacteria is much less because of the volume taken up by the media. Since media volume is difficult to measure precisely, EBRT is typically used as a design criterion. A sprinkler system irrigates the biofilter media to provide moisture to keep the bacteria alive.

The DAFT and clarifier biofilter consists of a single bed. The total surface area of this bed is 1,160 sq-ft with a media depth of 5 ft for a total media volume of 5,800 cu-ft. A fan draws air from the DAFT and clarifiers 1 and 2 and discharge to the biofilter. Foul air is treated by bacteria as it passes through the media. The fan air flow rate at design is 1,000 cfm. 1,000 cfm air passing through 5,800 cu-ft results in an EBRT of 348 seconds. Actual contact time with the bacteria is much less because of the volume taken up by the media. Since media volume is difficult to measure precisely, EBRT is typically used as a design criterion. A sprinkler system irrigates the biofilter media to keep bacteria alive.

3.6.2 Ferric Chloride System

The WWTP has a ferric chloride system that consists of a 5000 gallon storage tank and two, 0.16 HP metering pumps. This system can be used to send ferric chloride to the anaerobic digesters to control hydrogen sulfide levels in the digester gas. The system can also be used to add ferric chloride upstream of the primary clarifiers to improve solids settling and help control odors in the primary clarification process.

3.6.3 Plant Water Systems

Each of the WWTP's process water systems (1W, 2W, and 3W) are discussed below.

3.6.3.1 1W System

Potable water from the City of Pleasanton is provided to the WWTP to supply the 1W system for use throughout the plant. These uses include emergency showers and eyewash stations by the DAFT building, hypochlorite building and chemical building. Additionally, 1W is used to service the Operations and Cogeneration Building. 1W becomes 2W at a booster pump station located inside the Operations and Cogeneration Building.

3.6.3.2 2W System

Non-potable water (2W) is supplied from the 1W system (originating from the City of Pleasanton) to serve clean water demands for mechanical seal flushing and other clean water demands in process areas. 2W is delivered for use throughout the plant via a water pressure booster system in the operations and cogeneration building. The 2W pump booster pump station consists of three, 7.5 HP, centrifugal pumps, each with a capacity of 10 gpm each, for a total capacity of 30 gpm and a firm capacity of 20 gpm.

3.6.3.3 3W System

Disinfected secondary effluent is the source of 3W for the WWTP. The 3W pumping facility is located adjacent to the CCT and shares a wet well with the Final Effluent Pump Station No. 1. The 3W pump system consists of three, 75 HP submersible pumps with a capacity of 350 gpm - for a total pumping capacity of 1,050 gpm and a firm capacity of 700 gpm. There is also a 75 HP vertical turbine, high-pressure 3W washdown pump, which has a capacity of 685 gpm.

3.6.4 Plant Air Systems

Filtered air is compressed and sent through air receiver tanks before it is distributed to one of the plant air systems. Compressed air can be:

- Sent directly to the DAFT,
- Sent to the service air system, or
- Pass through a desiccant dryer before entering the instrument air system.

The service air system connects to several utility stations and corresponding piping throughout the WWTP. The service air system extends through the Influent Pump Station, influent distribution structure, influent diversion structure, headworks, RAS pumping facilities, several of the secondary clarifiers, digesters, Holding Basin 1, and Building D.

The instrument air system delivers desiccant dried air to various instruments and equipment throughout the WWTP, including electropneumatic actuators and a bubbler. The instrument air system extends through, the headworks, bar screen building, grit processing area, blower building, aeration tanks, Mixed Liquor/WAS pumping facility, Clarifier 4, the digesters, Operations Building, Cogeneration Building, and Building D.

3.6.5 Cogeneration System

The existing cogeneration system consists of three reciprocating engine generators, which are connected to the Plant's electrical power distribution system; digester gas (DG) and natural gas (NG) fuel systems; and an engine heat recovery and hot water distribution system.

In 2008, two lithium carbonate-based fuel cells with 300 kilowatt (kW) capacity each were brought online. A digester gas treatment system to remove hydrogen sulfide, water, and siloxanes was installed as part of the fuel cell project. After the five-year fuel cell warranty expired, it was determined that the fuel cells were too costly to operate. In 2013, the fuel cells were removed from the WWTP site. However, the digester gas treatment system remains online and is used in concert with the cogeneration engines.

3.6.5.1 Cogeneration Engines

The original cogeneration system was installed in 1982 and consisted of two engine generators rated at 200 kW each. By the time of the Stage 4 Improvements Project, the cogeneration engine, had been changed out to two, 450 kW engines with heat recovery. Since 2002, there have been incremental projects that increased the capacity of the two engines to 500 kW each. In 2010, DSRSD staff purchased, installed and upgraded a used cogeneration engine that has a capacity of 400 kW. This brought the total installed cogeneration capacity to 1,400 kW.

The three engine generator systems are located in a cogeneration room in the south side of the Administration Building. Engines 1 and 3 are rich burn engines, meaning there is less than 3 percent oxygen in the exhaust, and Engine 2 is a lean burn engine, meaning there is more than 9 percent oxygen in the exhaust. For air permitting reasons, the DG is preferentially utilized in cogeneration Engine 2, the lean burn engine. Excess DG above Engine 2 needs can be burned in either of the other two engines when blended with NG.

Each cogeneration engine produces electrical power and heat energy. The heat energy can be beneficially used in the treatment plant for digester heating and building needs as discussed in Section 3.6.5.3. The current design information for the three engine generators is summarized in Table 3-21.

Table 3-21. Existing Cogeneration Reciprocating Engine Design Summary

Engine No. (Air Permit Source No.)	Engine Make/Model	Emission Control	Manufacturer Continuous Duty Rating, kW at 1200 rpm[a]	Heat Recovery at Rating[b], MM Btu/hr	Typical Output[c], kW	Heat Recovery at Typical Output, MM Btu/hr
No. 1 (S-13)	Waukesha VHP3521 GSI	Exhaust 3-way catalytic converter	500	3.19	400	2.63
No. 2 (S-22)	Waukesha VHP3521 GLD	Lean burn	500	2.76	400	2.28
No. 3 (S-21)	Waukesha VHP28953 GSI	exhaust 3-way catalytic converter	400[d]	2.59	400	2.59
		Totals	**1,400**	**8.54**	**1,200**	**7.50**

[a] Revolutions per minute.
[b] Heat is recovered from each engine by heat exchange from engine systems to the circulating hot water system as discussed in Section 3.6.5.3. Heat recovery at duty rating is from manufacturer's published information.
[c] Typical output reflects limits established by plant staff to keep engine running reliably and to meet air permit annual gas fuel limits contained in the BAAQMD Permit to Operate (PTO) for the three engines.
[d] Engine 3 has a 500 kW rated generator, but the engine continuous operation duty rating is 400 kW.

3.6.5.2 Engine Fuel Systems

The cogeneration engines are fueled by anaerobic DG and utility NG. DG is collected off all three of the existing anaerobic digesters discussed in Section 3.4.3. NG gas is delivered to the WWTP site at approximately 50 pounds per square inch gage (psig) at the meter near the East Amador Lift Station. NG is purchased through the State of California, Department of General Services. As of October 2015, DSRSD is purchasing NG at approximately $0.31 per therm. This section addresses the gas piping systems and the DG treatment system.

3.6.5.2.1 Gas Piping Systems

The DG collection piping from the digesters is schematically shown on Figure 3-20. Figure 3-21 represents a piping schematic for the DG treatment system and the DG/NG conveyance system to the cogeneration engines.

3.6.5.2.2 DG Treatment

Low pressure DG that is produced from the digesters flows to a gas treatment system originally installed as part of the fuel cell project discussed previously. The treatment steps are as follows:

- The first treatment step is hydrogen sulfide removal. Initially, chemical addition was used to enhance iron sponge removal of hydrogen sulfide. However, the iron sponge media was later replaced with a proprietary media called SulfaTreat that does not need the chemical addition.
- In the next treatment step, the DG pressure is raised to 30 to 50 psig in an oil lubricated rotary lobe compressor. The higher pressure DG is then treated to remove water vapor in a series of heat exchangers and finally in a refrigerated glycol heat exchanger to achieve a 38°F dew point.
- In the third step, the DG is warmed about 25 degrees above the dew point in a cold-to-warm gas heat exchanger. Heating the DG enhances siloxane bed life and eliminates the possibility of liquid water droplets getting into the engine intake, which can destroy the engine. Removing the water in DG also raises the effective energy content of the fuel, which helps the engines run better.
- The final step is siloxane removal, which occurs through absorption onto carbon media in a siloxane treatment vessel.

There are a number of deficiencies with the gas treatment system. First, the capacity was designed to provide 180 cfm of gas at an outlet pressure of 30 psig. However, actual operation is unable to achieve more than 140 cfm of gas delivery. An undersized gas compressor, pipes being too small, and heat exchangers being too small are suspected causes. Additional deficiencies related to the energy use of the gas treatment system have also been identified, not the least of which includes the fact that the system delivers gas at a pressure and quality that is much higher than what is required for use in Engine 2 (the lean burn engine) and at a higher pressure than what is required for use in Engine 3. Finally, the digester gas cleaning system is undersized for treating all the available DG, and the District sometimes is required to flare DG that cannot be processed through the system. For these reasons, a wholesale review of the District's DG treatment needs is recommended, and this evaluation is presented in the Energy Management Plan Chapter of this Master Plan.

3.6.5.3 Heat Recovery System

The cogeneration engines provide additional energy in the form of hot water. This hot water is distributed to provide heat energy for the anaerobic digesters and for heating and cooling the Administration Building and the Maintenance Building (shops and office areas). The heat recovery system piping schematic is illustrated on Figure 3-22.

Heat is recovered off each engine by exchanging engine generated heat to a plant hot water loop that distributes the heat energy to the use areas. The engine sources of heat that are recovered include the engine lubricating oil cooler, the engine jacket water (the heat that normally goes to an engine radiator) and the exhaust gas flow. These sources are hot enough to heat the hot water loop to approximately 170°F to 180°F.

From the total engine fuel used, the approximate amount of the fuel energy that goes to heat instead of electric power varies by engine. For the two rich burn engines, the percent of fuel energy is approximately 4.5 percent to the lube oil, 31 percent to the jacket water and 21 percent to the exhaust. For the lean burn engine No. 2, the input energy distribution is approximately 4.4 percent to the lube oil, 23 percent to the jacket water and 19 percent to the exhaust. This means the rich burn engines can recover up to 56 percent of the fuel energy as heat and the lean burn can recover 46 percent of the fuel energy as heat.

Heat energy is used in the plant to heat the sludge being pumped in to the digesters up to operating temperature and to provide the digester energy lost through the tank walls and roofs to the ambient conditions. Heat energy is also utilized to provide building Heating, Ventilation and Air Conditioning (HVAC) for the Administration Building and the Maintenance Building. In winter the hot water is used directly for building heating and to heat domestic hot water for bathroom and shower facilities. In summer the hot water is used to drive absorption chillers that use the heat to make chilled water for air conditioning purposes. Table 3-22 summarizes the current heat energy uses in the plant.

Table 3-22. Current Average Hot Water Heat Demand Summary[(a)]

Parameter	Units	Current Demand @ 10.8 mgd	
		Summer	Winter
Digester Heat Demand			
Sludge Pumped to Digesters	gpd	99,900	99,900
Heat for Pumped Sludge	MM Btu/hr	0.90	1.07
Digester 1, 2 and 3 Convective Loss	MM Btu/hr	0.34	0.98
Building Heat Demand			
Administrative Building HVAC (chiller in Summer)	MM Btu/hr	1.27	0.99
Maintenance Building (chiller in Summer)	MM Btu/hr	1.01	0.64
Total Demand	**MM Btu/hr**	**3.52**	**3.68**

(a) Heat demands are based on six month averages, not design day values.

At current engine operating limits shown in Table 3-21, the three engines can recover approximately 7.5 MM Btu/hr. As shown in Table 3-22, the average plant heat energy use is approximately 3.52 MM Btu/hr in summer and 3.68 MM Btu/hr in winter. Therefore, the average heat demand is more than one engine output (2.28 to 2.63 MM Btu/hr) and less than two engines (4.87 to 5.22 MM Btu/hr). Moreover, if all three engines are running, there is 3.8 to 4.0 MM Btu/hr excess heat energy available, but it is not being used.

During Spring and Fall seasons, the beneficial heat demands are lower, and at times drop to almost zero. This excess energy is exchanged to plant 3W to "waste" the heat in two shell and tube heat exchangers, located just outside of the engine room. Since demand for beneficially used heat energy varies seasonally, the heat rejection system is designed to provide engine cooling over a wide range of heat up to cooling all the engine heat generation.

The hot water distribution system is a series of main circulation loops with multiple sub-loops off the main loop systems. Pumps are used to drive the flow in the main loop and there are pumps in each sub-loop system. The main hot water loop starts in the Administration Building and it provides heat to the anaerobic digester systems. One sub-loop extends to the Maintenance Building. Other sub loops direct heat to operations areas like the old bar screen room. Most of the unoccupied plant buildings no longer use heating or cooling for the ventilation air.

The hot and cold water distribution systems are shown on the following figures:

- Figure 3-23 shows the approximate routing of the main hot water loop through the Administration Building.
- Figure 3-24 shows the hot and chilled water piping schematic for the Administration Building main loop and sub-loops.
- Figure 3-25 shows the main hot water loop at the anaerobic digesters.
- Figure 3-26 shows some of the piping details of the outside portion of the Maintenance Building main loop, which is just outside the southwest side of the shop building.
- Figure 3-27 and Figure 3-28 shows details of the Administration Building chilled water circulation system.
- Figure 3-29 shows details of the hot water piping at specific areas like the domestic hot water heater and Air Handling Unit (AHU) 2 for the Administration Building.
- Figure 3-30 shows the hot water piping details at the pump room in the Administration Building.

3.7 DESIGN CRITERIA SUMMARY

Tables 3-23 through 3-28 provide a summary of the available treatment capacity of the major treatment and conveyance facilities.

Table 3-23. Liquid Treatment Facilities Capacity Summary

Facility/Design Basis	Design Criteria		Number of Components	Capacity		
	Criteria	Units		Firm[a]	Total[b]	Units
Preliminary Treatment/Influent Pumping						
Screening						
Mechanical Bar Screens	N/A	--	3	50	75	mgd
Screenings Washer/Compactor	N/A	--	2	15	30	cfh
Influent Pumps	N/A	--	4	65	90	mgd
Plant Inner-Sewer Pumps	N/A	--	2	1	2	mgd
Grit Removal						
Grit Tank Peak Flow Based on Recommended Detention Time [c]	3.5	minutes	3	31.3	49.0	mgd
Grit Pre-Aeration Blowers	N/A	--	3	550	910	cfm
Grit Pumps	N/A	--	3	400	[d]	gpm
Grit Seperators	N/A	--	2	250	500	gpm
Primary Treatment						
Primary Sedimentation Basins						
Average Flow Based on Recommended Detention Time [c]	1.5	hours		6.0	8.0	mgd
Average Flow Based on Recommended Overflow Rate [c]	1,200	gpd/sq-ft	4	6.9	9.2	mgd
Peak Hourly Flow Based on Recommended Overflow Rate [c]	3,000	gpd/sq-ft		17.1	22.8	mgd
Primary Sedimentation Basin 1-4 Sludge Pumps [e]	N/A	--	4	--	90.0	gpm
Primary Sedimentation Basin 1-4 Scum Pumps (Basins 3 and 4 Share One Pump) [e]	N/A	--	3	--	50.0	gpm
Secondary Treatment						
Secondary Reactor						
Tank Volume	N/A	--		385,600	482,000	cf
Maximum Week BOD Loading at Original Design Condition	N/A	--	5	27,840	34,800	lb BOD/day
HRT at 2035 Maximum Month Flow	N/A	--		4.3	5.4	hours
Aeration Blowers	N/A	--	3	17,000	25,500	cfm
Mixed Liquor/WAS Pumps	N/A	--	3	1,670	2,505	gpm
Secondary Clarifiers						
Average Hourly Flow Based on Recommended Overflow Rate [c]	500	gpd/sq-ft		9.5	14.3	mgd
Peak Hourly Flow Based on Recommended Overflow Rate [c]	1,100	gpd/sq-ft		21.0	31.4	mgd
Average Hourly Flow Based on Recommended Solids Loading Rate [c]	1.0	lb/sq-ft-hr	4	22.9	34.3	mgd
Peak Hourly Flow Based on Recommended Solids Loading Rate [c]	2.0	lb/sq-ft-hr		45.8	68.6	mgd
Peak Hourly Flow at SVI 170 mg/L and MLSS of 2,400 mg/L	[f]	--		26	38	mgd
Peak Hourly Flow at SVI 300 mg/L and MLSS of 2,400 mg/L	[f]	--		13	20	mgd
Secondary Clarifier 1-2 RAS Pumps	N/A	--	2	6.1	12.2	mgd
Secondary Clarifier 3-4 RAS Pumps	N/A	--	2	6.1	14.6	mgd
Secondary Clarifier 1 Scum Pump	N/A	--	1	--	150	gpm
Secondary Clarifier 2 Scum Pump	N/A	--	1	--	150	gpm
Secondary Clarifier 4 Scum Pump	N/A	--	1	--	150	gpm
Secondary Effluent Disinfection						
Chlorine Contact Tank						
Average Flow Based on Recommended Modal Contact Time to Effluent Diversion Structure [g]	60	minute	1	--	11.6	mgd
	50	minute		--	14.0	mgd
Peak Flow Based on Recommended Modal Contact Time to LAVWMA Junction Structure [g]	30	minute		--	36.3	mgd
	25	minute		--	43.6	mgd
Post-Chlorination Pumps	N/A	--	3	1,200	3,600	gpd
RAS-Chlorination Pump	N/A	--	1	--	200	gpd
MF Metering Pump	N/A	--	1	--	420	gpd
Secondary Effluent Pumping						
Final Effluent Pump Station No. 1 Pumps						
Flow When Pumping to Holding Basin 4	N/A	--	3	50	75	mgd
Flow When Pumping to LAVWMA	N/A	--		35	53	mgd
Final Effluent Pump Station No. 2 Pumps	N/A	--	5	58	77	mgd

[a] Available capacity with the largest unit out of service.

[b] When multiple pumps discharge to a common header, pump station total capacities are estimated based on rated unit capacities defined for the design condition. The actual capacity may be lower due to increased head losses at higher flows, depending on how the pumps were originally rated (i.e., all pumps running or one on standby).

[c] Wastewater Engineering Treatment and Resource Recovery, Fifth Edition, Metcalf & Eddy, (2014).

[d] Due to capacity limitations in the Grit Separators, all three pumps cannot operated simultaneously.

[e] Capacity is for each pump. Each pump is dedicated to either a single basin, or in the case of the scum pump for Basins 3 and 4, to a pair of basins. No installed pumping redundancy is provided.

[f] Based on State Point Analysis model results.

[g] Dublin San Ramon Services District Coliform Limit Study,Whitley Burchett & Associates, (2000).

Table 3-24. Flow Equalization/Storage Facilities Capacity Summary

Facility Design Basins	Number of Components	Capacity		
		Firm[a]	Total	Units
Sump Pump from Holding Basin 4 to Holding Basin 2	1	Unknown	Unknown	mgd
Holding Basin 1				
Storage Volume Based on Maximum WSEL (Elevation = 326.3 ft)	1	--	2.3	Mgal
Storage Volume Based on 2-Foot Freeboard WSEL (Elevation = 324.3 ft)		--	1.7	Mgal
Storage Volume Filled by Gravity from CCT (Elevation = 323.0 ft)		--	1.3	Mgal
Holding Basin 2				
Storage Volume Based on Maximum WSEL (Elevation = 327.1 ft)	1	--	5.1	Mgal
Storage Volume Based on 2-Foot Freeboard WSEL (Elevation = 325.1 ft)		--	4.3	Mgal
Storage Volume Filled by Gravity from CCT (Elevation = 323.0 ft)		--	3.3	Mgal
Holding Basin 3				
Storage Volume Based on Maximum WSEL (Elevation = 328.3 ft)	1	--	6.8	Mgal
Storage Volume Based on 2-Foot Freeboard WSEL (Elevation = 326.3 ft)		--	5.6	Mgal
Storage Volume Filled by Gravity from CCT (Elevation = 323.0 ft)		--	3.8	Mgal
Holding Basin 4				
Storage Volume Based on Maximum WSEL (Elevation = 329.9 ft)	1	--	7.2	Mgal
Storage Volume Based on 2-Foot Freeboard WSEL (Elevation = 327.9 ft)		--	6.0	Mgal
Storage Volume Filled by Gravity from CCT (Elevation = 318.0 ft)		--	0.9	Mgal
Holding Basins 1, 2, 3 and 4				
Storage Volume for Raw/Primary Wastewater [b]	4	--	11.5	Mgal
Storage Volume for Secondary Wastewater Filled by Gravity from CCT [c]		--	9.3	Mgal
Storage Volume for Secondary Wastewater [d]		--	17.6	Mgal

[a] Available capacity with the largest unit out of service.
[b] Holding Basins 1-3 Only.
[c] Holding Basins 1-4.
[d] Holding Basins 1-4; Requires pumping from Holding Basin 4 to Holding Basin 2.

Table 3-25. Solids Treatment Facilities Capacity Summary

Facility/Design Basis	Design Criteria		Number of Components	Capacity		
	Criteria	Units		Firm[a]	Total[b]	Units
Solids Thickening						
DAFT Maximum Month Solids Loading Based on Recommended Solids Loading Rate [c]	0.7	lb/sq-ft-hr	1	--	47,500	lb/day
Pressurization Pumps	N/A	--	2	1,600	3,200	gpm
Bottom Sludge Pump	N/A	--	1	--	100	gpm
Thickened Sludge Pumps	N/A	--	2	200	400	gpm
Gravity Belt Thickener at 0.3 to 0.6 Percent Solids	N/A	--	1	--	0.4	mgd
Anaerobic Digestion						
Anaerobic Digesters						
Maximum Month Solids Load Based on Recommended Solids Loading Rate [c]	150	lb VSS/ 1,000 cu-ft-day	3	23,200	43,900	lb VSS/ day
Average Flow Based on Lowest Recommended Retention Time [c]	15	days		77,300	146,200	gal/day
Anaerobic Digester 1 Mechanical Mixer	N/A	--	1	--	12,500	gpm
Anaerobic Digester 2 Mechanical Mixer	N/A	--	1	--	12,500	gpm
Anaerobic Digester 3 Mechanical Mixers	N/A	--	3	14,400	21,600	gpm
Anaerobic Digester 1 Sludge Circulation Pump	N/A	--	1	--	600	gpm
Anaerobic Digester 2 Sludge Circulation Pump	N/A	--	1	--	600	gpm
Anaerobic Digester 3 Sludge Circulation Pump	N/A	--	1	--	600	gpm
Digested Sludge Facultative Sludge Lagoon pumps	N/A	--	3	400	600	gpm
Facultative Sludge Lagoons						
Facultative Sludge Lagoons						
Average Solids Loading Based on Recommended Solids Loading Rate [c]	2.3	lb/cu-ft-year	7	30,800	38,400	lb TSS/ day
Average Volatile Solids Loading Based on Recommended Peak Month Loading Rate [d]	15	lb VSS/ 1,000 sq-ft-day		10,500	13,300	lb VSS/ day
Facultative Sludge Lagoon Cap Water Pumps	N/A	--	2	500	1,500	gpm
Dedicated Land Disposal Site						
Average Solids Loading at 2-3 Percent Solids Based on Recommended Solids Loading Rate [e]	50	dt/ ac-yr	1	--	2,300	dt/ yr

[a] Available capacity with the largest unit out of service.

[b] When multiple pumps discharge to a common header, pump station total capacities are estimated based on rated unit capacities defined for the design condition. The actual capacity may be lower due to increased head losses at higher flows, depending on how the pumps were originally rated (i.e., all pumps running or one on standby).

[c] *Wastewater Engineering Treatment and Resource Recovery, Fifth Edition* , Metcalf & Eddy, (2014).

[d] District operations manual.

[e] Original design criterion was 100 dt/ac-yr at 4 to 6 percent solids. Listed design criterion is based on District experience applying at 2 to 3 percent solids concentration.

Table 3-26. Tertiary Plant Facilities Capacity Summary

Facility/Design Basis	Design Criteria		Number of Components	Capacity		
	Criteria	Units		Firm[a]	Total[b]	Units
Sand Filtration with UV Disinfection						
Tertiary Influent Pumps	N/A	--	3	6.8	10.2	mgd
Band Screens	N/A	--	1	--	12.0	mgd
Rapid Mix Tank Peak Flow Based on Recommended Detention Time [c]	17.5	sec		4.7	9.3	mgd
Flocculation Basin Peak Flow Based on Recommended Detention Time [d]	5.3	minutes	2	5.1	10.2	mgd
Sand Filter Peak Flow Based on Hydraulic Loading Rate [e]	3.4	gpm/sq-ft		6.6	7.9	mgd
Sand Filter Reject Water Pumps	N/A	--	2	350	700	gpm
UV Disinfection Maximum Flow for UV Dose of 100 mJ/cm^2 at 55% Transmittance [e]	N/A	--	3	4.9	9.7	mgd
Microfiltration with UV Disinfection						
Microfiltration Transfer Pumps	N/A	--	2	3.7	7.4	mgd
Microfiltration Feed Pumps	N/A	--	4	5.1	6.9	mgd
Microfiltration Units	N/A	--	5	2.68	3.35	mgd
UV Disinfection Maximum Flow for UV Dose of 80 mJ/cm^2 at 65% Transmittance [f]	N/A	--	1	--	3.0	mgd
Recycled Water Pumping						
Recycled Water R1 Pumps	N/A	--	3	9.7	14.6	mgd
Recycled Water Pump Station Wet Well Chlorinators	N/A	--	2	440	880	gpd

[a] Available capacity with the largest unit out of service.

[b] When multiple pumps discharge to a common header, pump station total capacities are estimated based on rated unit capacities defined for the design condition. The actual capacity may be lower due to increased head losses at higher flows, depending on how the pumps were originally rated (i.e., all pumps running or one on standby).

[c] *Wastewater Engineering Treatment and Resource Recovery, Fifth Edition* , Metcalf & Eddy, (2014).

[d] Recycled Water Treatment Project design criteria.

[e] *DSRSD-EBMUD Recycled Water Authority Recycled Water Treatment Facilities Plan* , Carollo, (2015).

[f] District operations manual.

Table 3-27. Odor Control System Capacity Summary

Facility/Design Basis	Design Criteria		Number of Components	Capacity		
	Criteria	Units		Firm[a]	Total[b]	Units
Odor Reduction Tower Foul Air Scrubbing Rate Based on Design Detention Time [c]	13.1	sec	1	--	18,010	cfm
Odor Reduction Tower Pumps	N/A	--	2	360	720	gpm
Headworks Soil Bed Media Filters Foul Air Scrubbing Rate Based on Design Detention Time [c]	160	sec	1	--	15,000	cfm
DAFT Soil Bed Media Filter	348	sec	1	-	1,000	cfm

[a] Available capacity with the largest unit out of service.

[b] When multiple pumps discharge to a common header, pump station total capacities are estimated based on rated unit capacities defined for the design condition. The actual capacity may be lower due to increased head losses at higher flows, depending on how the pumps were originally rated (i.e., all pumps running or one on standby).

[c] Stage 4 design Criteria.

Table 3-28. Plant Water Systems Capacity Summary

Facility/Design Basis	Design Criteria		Number of Components	Capacity		
	Criteria	Units		Firm[a]	Total[b]	Units
2W Booster Pumps	N/A	--	3	20	30	gpm
3W Washdown Pump	N/A	--	1	-	685	gpm
3W Pumps	N/A	--	3	700	1,050	gpm

[a] Available capacity with the largest unit out of service.

[b] When multiple pumps discharge to a common header, pump station total capacities are estimated based on rated unit capacities defined for the design condition. The actual capacity may be lower due to increased head losses at higher flows, depending on how the pumps were originally rated (i.e., all pumps running or one on standby).

3.8 REFERENCES

Brown and Caldwell. (2002). *Effluent Transport Facilities Basis of Design Report.*

Brown and Caldwell. (2004). *Plant Process Capacity Study: Dry Weather Capacity*

Brown and Caldwell (2005). *Wet Weather Capacity Study*

CH2M Hill. (2004). *Odor Control Master Plan Update.*

Carollo. (2015). *DSRSD-EBMUD Recycled Water Authority Recycled Water Treatment Facilities Plan.*

Metcalf & Eddy/AECOM. (2014). *Wastewater Engineering Treatment and Resource Recovery Fifth Edition.*

NWRI. (2012). *Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, Third Edition.*

RMC. (2014). *Secondary Effluent Wet Weather Capacity Evaluation.*

West Yost Associates. (2005). *NPDES Permit Compliance Action Plan*.

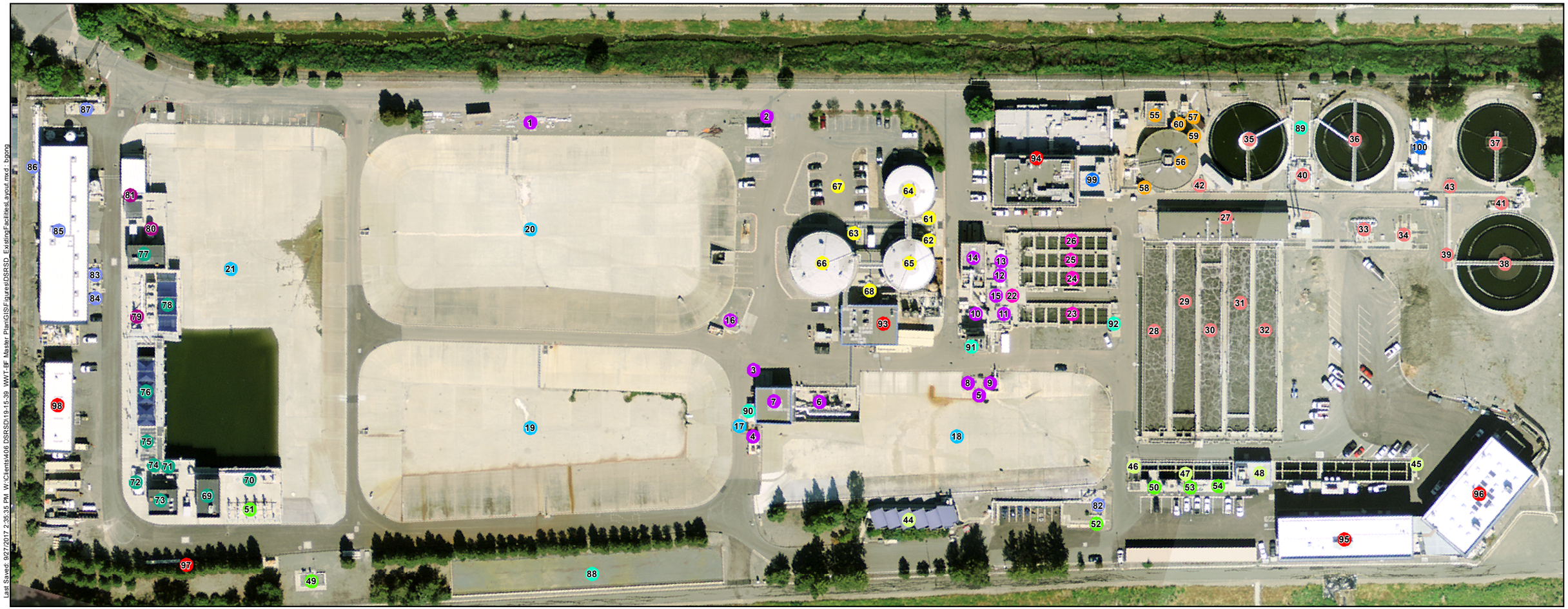
West Yost Associates. (2005). *Phase 1 Feasibility Study.*

Whitley Burchett & Associates and Brown & Caldwell. (1997). *Wastewater Treatment Plant 1997 Master Plan Update Project CIP 96CH142 Phase 1 Report.*

Whitley Burchett & Associates and Brown & Caldwell. (1998). *Wastewater Treatment Plant 1997 Master Plan Update Project CIP 96CH142 Phase 2 Report.*

Whitley Burchett & Associates. (2000). *Dublin San Ramon Services District Coliform Limit Study.*

Whitley Burchett & Associates. (2007). *Final Report 2007 Wastewater Treatment Plant Expansion Planning Report.*



LEGEND

- PRE-TREATMENT
1. Holding Basin 3 Drainage/Diversion Structure
2. East Amador Trunk Sewer Pump Station (2 Pumps)
3. Dublin Sewer Distribution Box
4. Pleasanton Junction Structure
5. Basin Lift Station (Not Used)
6. Bar Screens
7. Screenings Washer/Compactor
8. Influent Diversion/Distribution Structure
9. Plant Inner-Sewer Pump Station
10. Influent Pump Station (4 Pumps)

11.-13. Pre-aeration Grit Removal Tank 1 through 3

14. Grit Separators and Classifier
15. Equip Gallery Room (3 Grit Pumps)
16. Vactor Station (New)
- HOLDING BASINS
17. Storage Distr. Station (Lift Station Not Used)

18.-21. Holding Basin 1 through 4

- PRIMARY TREATMENT
22. Equip Gallery Room (3Prim Scum/4Prim Sludge Pumps)

23.-26. Primary Sedimentation Basin 1 through 4

- SECONDARY TREATMENT
27. Blower Building

28.-32. Aeration Tank 1 through 5

33. Mixed Liquor WAS Pump Station (3 Pumps)
34. Mixed Liquor Distribution Structure

35.-38. Secondary Clarifier 1 through 4

39. Tank 4 Secondary Scum Pump Station (1 Pump)
40. Secondary Clarifier 1 and 2 RAS Pump Stations (2 Pumps)
41. Secondary Clarifier 3 and 4 RAS Pump Stations (2 Pumps)
42. Dewatering Pump Station for Secondary Clarifiers 1-3
43. Dewatering Pump Station for Secondary Clarifier 4
- DISINFECTION
44. Hypochlorite Storage
45. RAS Chlorination Diffuser
46. Chlorine Influent Junction Box
47. Chlorine Contact Tank
48. Motor Control Center/Sample Building
- EFFLUENT PUMPING
49. Effluent Diversion Structure
50. Final Effluent Pump Station 1 (3 Pumps)
51. Final Effluent Pump Station 2 (5 Pumps)
52. Facultative Sludge Lagoon Cap Water Pump Station (2 Pumps)
53. 3 Water Pump Station (2 Pumps)
54. Washdown Pump Station (1 Pump)
- DISSOLVED AIR FLOATATION THICKENER (DAFT)
55. DAFT Motor Control Center
56. DAFT
57. DAFT Pressurization Pump Station (2 Pumps)

58.-59. Thickened Sludge Pump Station 1 through 2 (2 Pump)

60. Bottom Sludge Pump Station (1 Pump)
- DIGESTION

61.-63. Digester Mixing Pump 1 through 3

64.-66. Anaerobic Digester 1 through 3

67. Future Anaerobic Digester Location
68. Facultative Sludge Lagoon Pump Station (3 Pumps)
- TERTIARY (SAND FILTRATION/ULTRAVIOLET (UV))
69. Pump Motor Control Center
70. Tertiary Influent Pump Station (3 Pumps)
71. Band Screen
72. Poly Aluminum Chloride Storage
73. Chemical Feed Equipment
74. Rapid Mix Tanks
75. Flocculation Basins
76. Sand FIlters
77. UV Motor Control Center
78. UV Disinfection Channels (For Sand Filters)
- TERTIARY (GENERAL)
79. Pump Station R1 (3 Pumps)
80. Surge Tank
81. Chemical storage
- TERTIARY (MICROFILTRATION (MF)/UV)
82. MF Transfer Pump Station (2 Pumps)
83. MF Feed Pump Station (4 Pumps)
84. MF Strainers
85. MF and UV Building
86. UV Disinfection Channel (For MF)
87. Product Water Pump Station (Not Used)
- ODOR CONTROL
88. Biofilter 1
89. Biofilter 2
90. Foul Air Blower
91. Odor Reduction Tower
92. Odor Reduction Tower Pump Station (2 Pumps)
- BUILDINGS
93. Building D
94. Operations Building
95. Maintenance Building
96. Maintenance Building
97. Future Maintenance Building Location
98. Maintenance Storage
- COGENERATION
99. Cogeneration Building
100. Fuel Cell Facility (Abandoned)

0 55 110

Scale in Feet

WEST YOST ASSOCIATES

Figure 3-1

Wastewater Treatment Plant Site Plan

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



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0 300 600

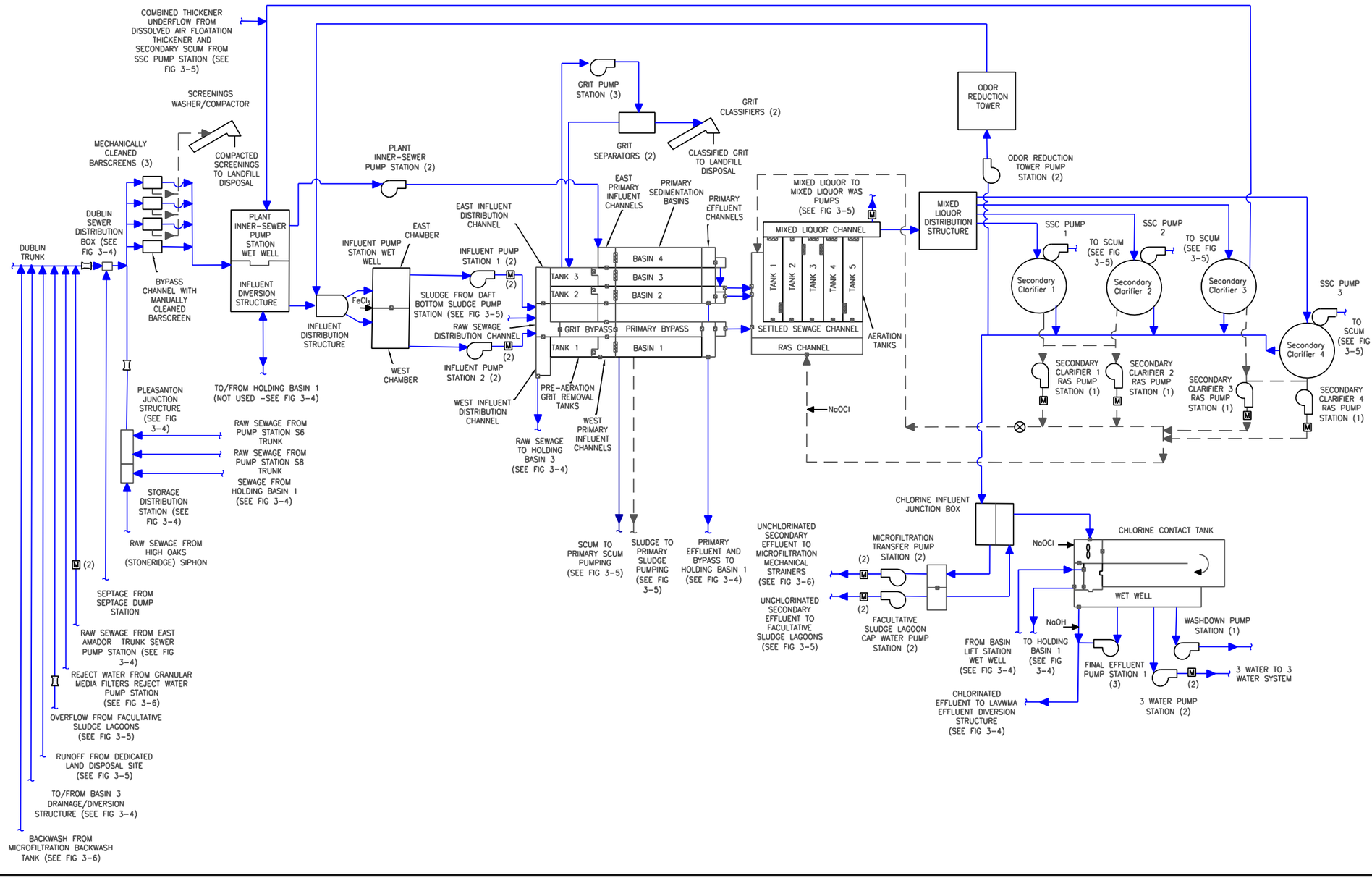
Scale in Feet

WEST YOST ASSOCIATES

Figure 3-2

Biosolids and LAVWMA Facilities Site Plan

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



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LEGEND

- Magmeter
- Flume
- Valve (Generic)
- Gate (Generic)
- Liquid Stream
- Solid Stream

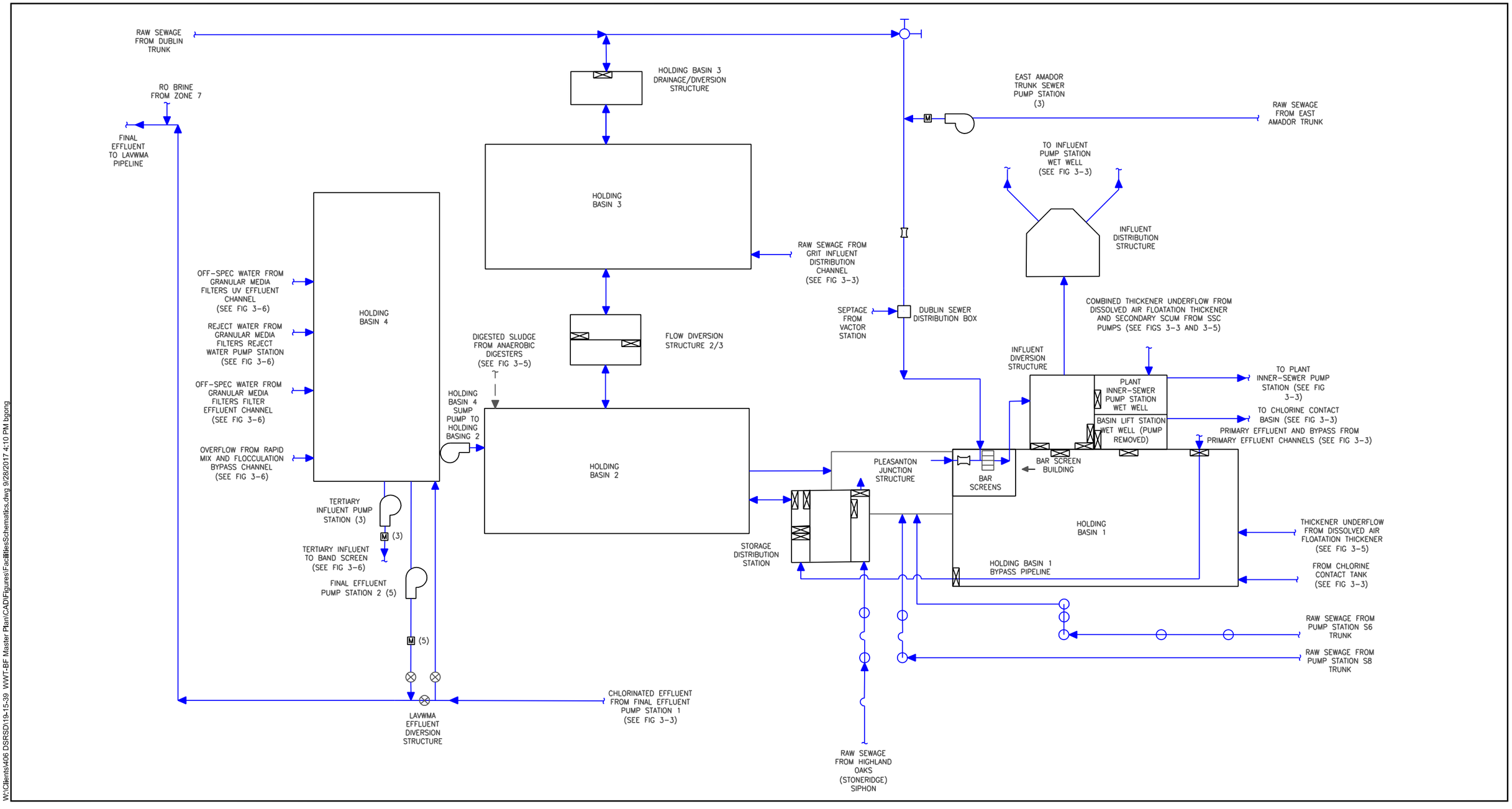
WEST YOST ASSOCIATES

Figure 3-3

Liquid Treatment Train Facilities Schematic

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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LEGEND

- Magmeter
- Flume
- Valve (Generic)
- Gate (Generic)
- Liquid Stream

Notes:
In 2018, the District will add a ballasted flocculation process downstream of the tertiary influent pumps. As part of this project two additional tertiary influent pumps will be installed.

WEST YOST ASSOCIATES

Figure 3-4

Storage Facilities Schematic

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

SCUM FROM PRIMARY SEDIMENTATION BASINS (SEE FIG 3-3)

SLUDGE FROM PRIMARY SEDIMENTATION BASINS (SEE FIG 3-3)

MIXED LIQUOR FROM AERATION TANKS MIXED LIQUOR CHANNEL (SEE FIG 3-3)

UNCHLORINATED SECONDARY EFFLUENT FROM FACULTATIVE SLUDGE LAGOON CAP WATER PUMP STATION (SEE FIG 3-3)

SCUM FROM SSC PUMPS 1,2 AND 3 (SEE FIG 3-3)

PRIMARY SCUM PUMPING (3)

PRIMARY SLUDGE PUMPING (4)

MIXED LIQUOR WAS PUMP STATION (3)

M

THICKENED SLUDGE PUMP STATION 2 (1)

DISSOLVED AIR FLOTATION THICKENER

PRESSURIZATION PUMP STATION (2)

THICKENER UNDERFLOW TO HOLDING BASIN 1 (SEE FIG 3-4)

DIGESTER GAS TO COGENERATION FACILITIES

DIGESTER CIRCULATION PUMP 1

$FeCl_2$

ANAEROBIC DIGESTER 1

DIGESTER CIRCULATION PUMP 2

$FeCl_2$

ANAEROBIC DIGESTER 2

DIGESTER CIRCULATION PUMP 3

$FeCl_2$

ANAEROBIC DIGESTER 3

THICKENED SLUDGE PUMP STATION 1 (1)

SECONDARY SCUM AND THICKENER UNDERFLOW TO INNER-SEWER PUMP STATION WET WELL (SEE FIGS 3-3 AND 3-4)

DAFT BOTTOM SLUDGE PUMP STATION (1)

SLUDGE TO PREARATION GRIT REMOVAL TANKS (SEE FIG 3-3)

FACULTATIVE SLUDGE LAGOON PUMP STATION (3)

(3)

DIGESTED SLUDGE TO HOLDING BASIN 2 (SEE FIG 3-4)

FACULTATIVE SLUDGE LAGOONS (FSL) (6)

FSL 7

FSL 3

FSL 4

FSL 1

FSL 2

FSL 6

OVERFLOW TO DUBLIN TRUNK (SEE FIG 3-3)

RUNOFF TO DUBLIN TRUNK (SEE FIG 3-3)

TRUCK TO DEDICATED LAND DISPOSAL SITE

DEDICATED LAND DISPOSAL SITE

LEGEND

- M — Magmeter
- Brush Aerator
- Valve (Generic)
- Liquid Stream
- Solid Stream
- Gas Stream

WEST YOST ASSOCIATES

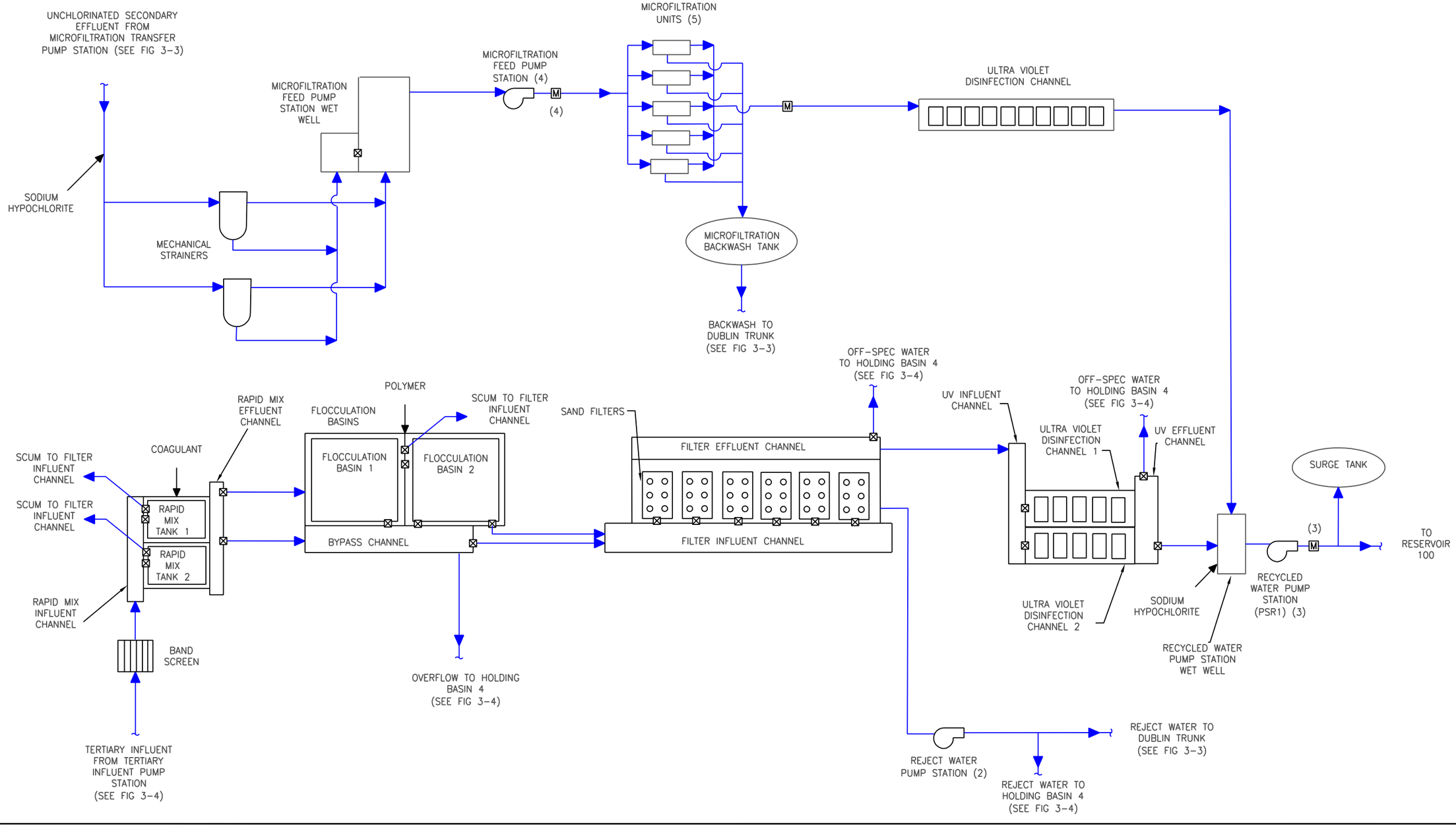
Figure 3-5

Solids Treatment Train Facilities Schematic

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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LEGEND

- M — Magmeter
- ⊠ — Gate (Generic)
- ——— Liquid Stream

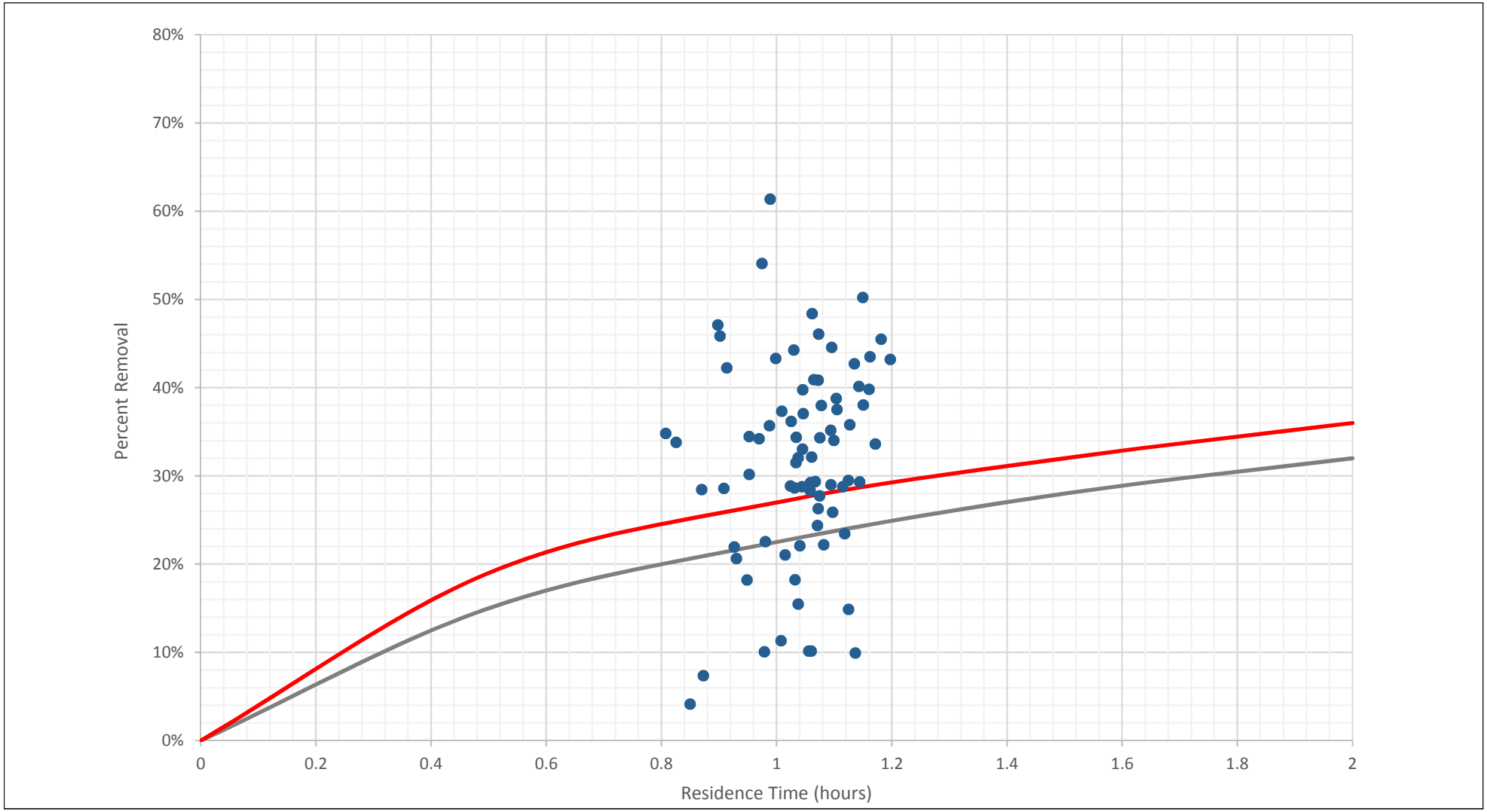
Note:
In 2018, the District will add a ballasted flocculation process between the tertiary influent pump station and the band screen. For this project, two additional tertiary influent pumps, a new band screen, and the ballasted flocculation process will be constructed. In addition two additional pumps will be installed at the recycled water pump station (PSR1) and one additional UV module will be installed in each UV bank in both UV channels.

WEST YOST ASSOCIATES

Figure 3-6

Recycled Water Treatment Train Facilities Schematic

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

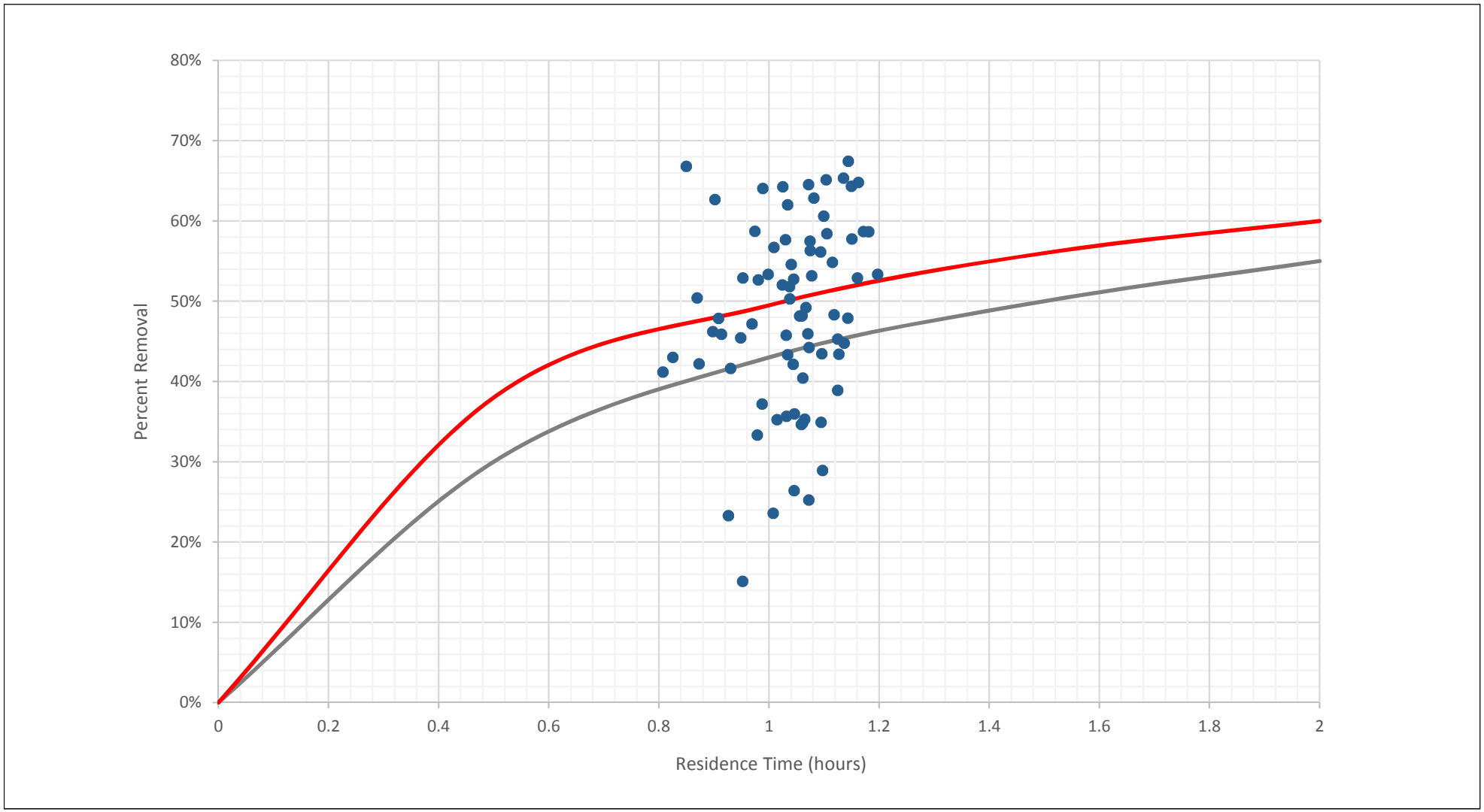


WEST YOST ASSOCIATES

Figure 3-7

Primary BOD Removal

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



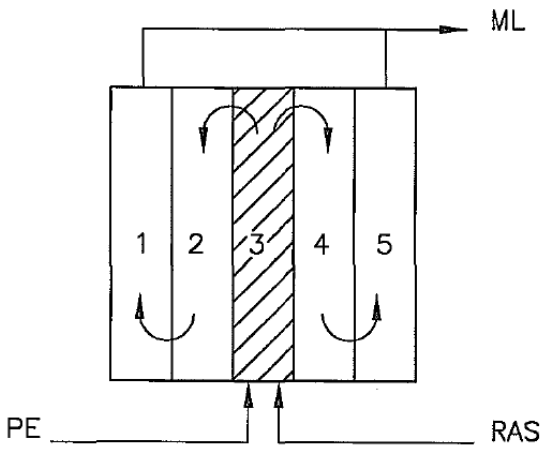
WEST YOST ASSOCIATES

Figure 3-8

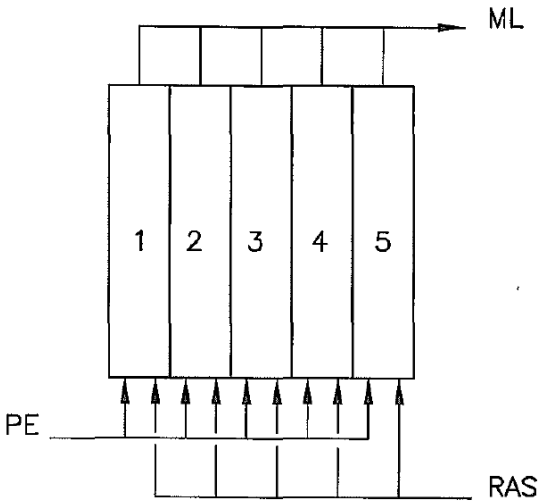
Primary TSS Removal

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

Series Configuration:
No FSL Supernatant Return



Parallel Configuration:
FSL Supernatant Return



LEGEND:

AEROBIC

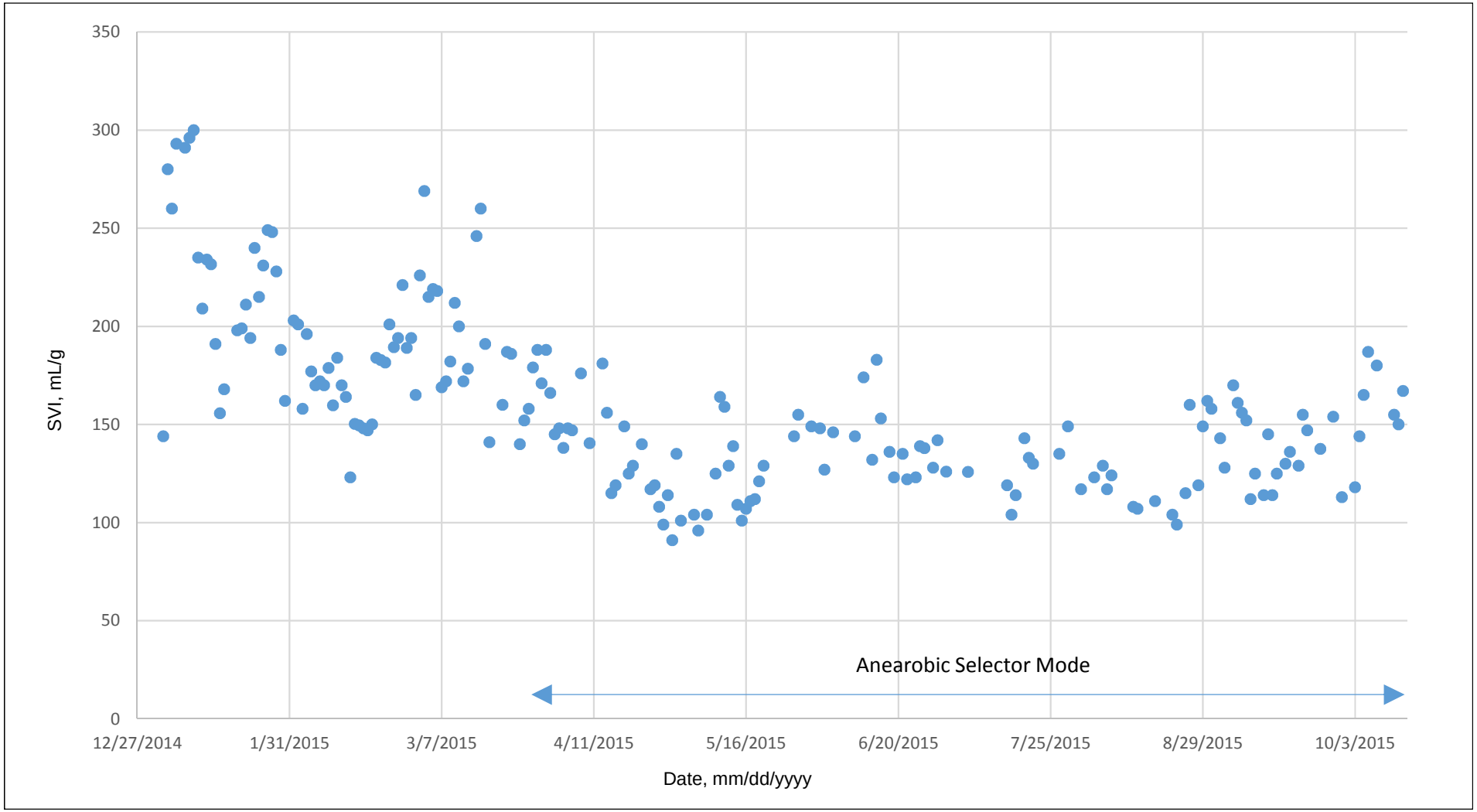
ANAEROBIC (MECHANICALLY MIXED)

Figure 3-9

Dublin San Ramon Services District
Wastewater Treatment Plant Master Plan

ACTIVATED SLUDGE REACTOR SEASONAL CONFIGURATIONS

WEST YOST ASSOCIATES
Consulting Engineers



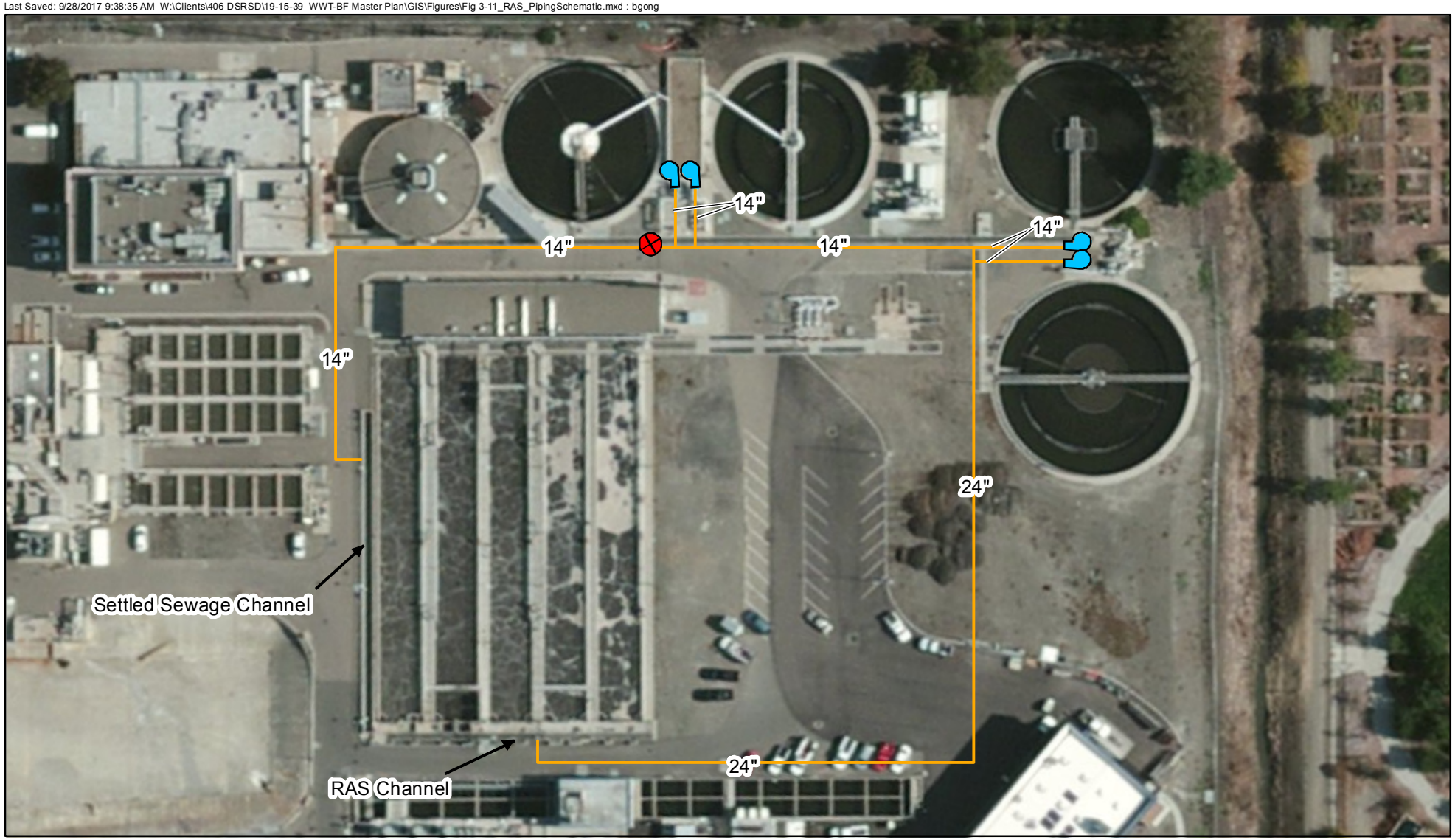
WEST YOST ASSOCIATES

Figure 3-10

Seasonal SVI Data

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- Valve
- Pump
- 14" Return Activated Sludge with Pipe Diameter

N

0 40 80

Scale in Feet

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Figure 3-11

Existing Return Activated Sludge Piping

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

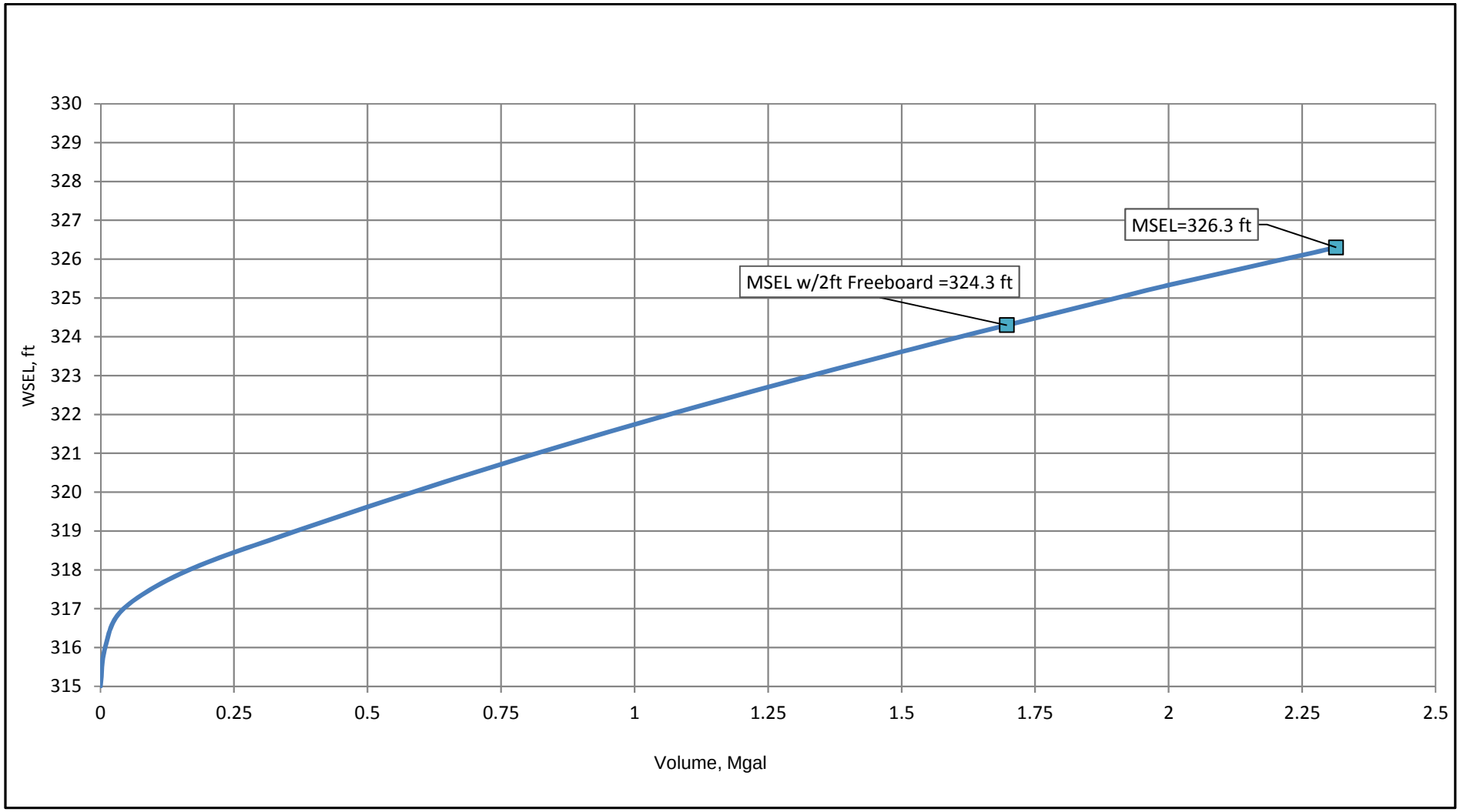
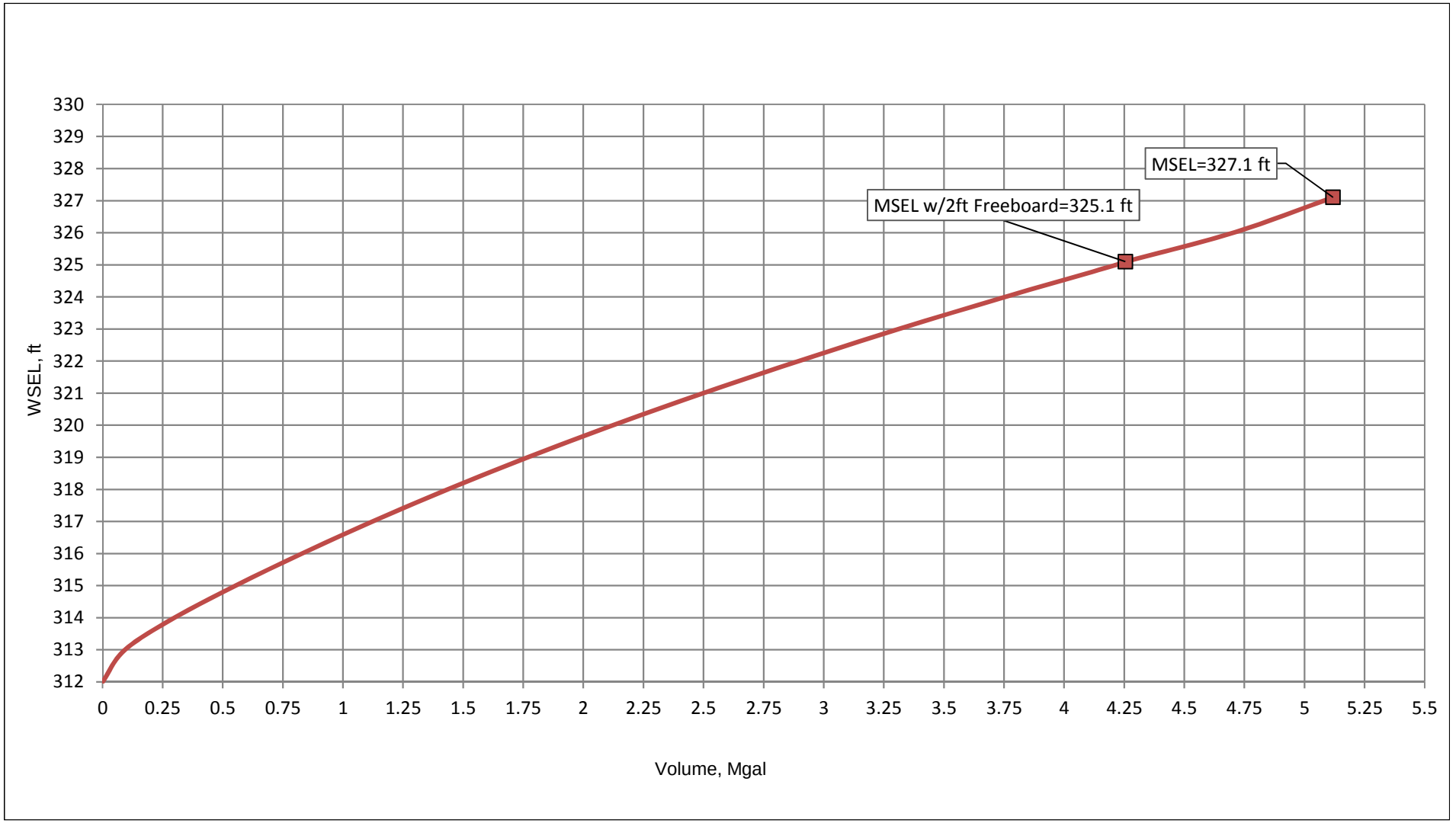


Figure 3-12

Basin 1 Stage Storage Curve

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

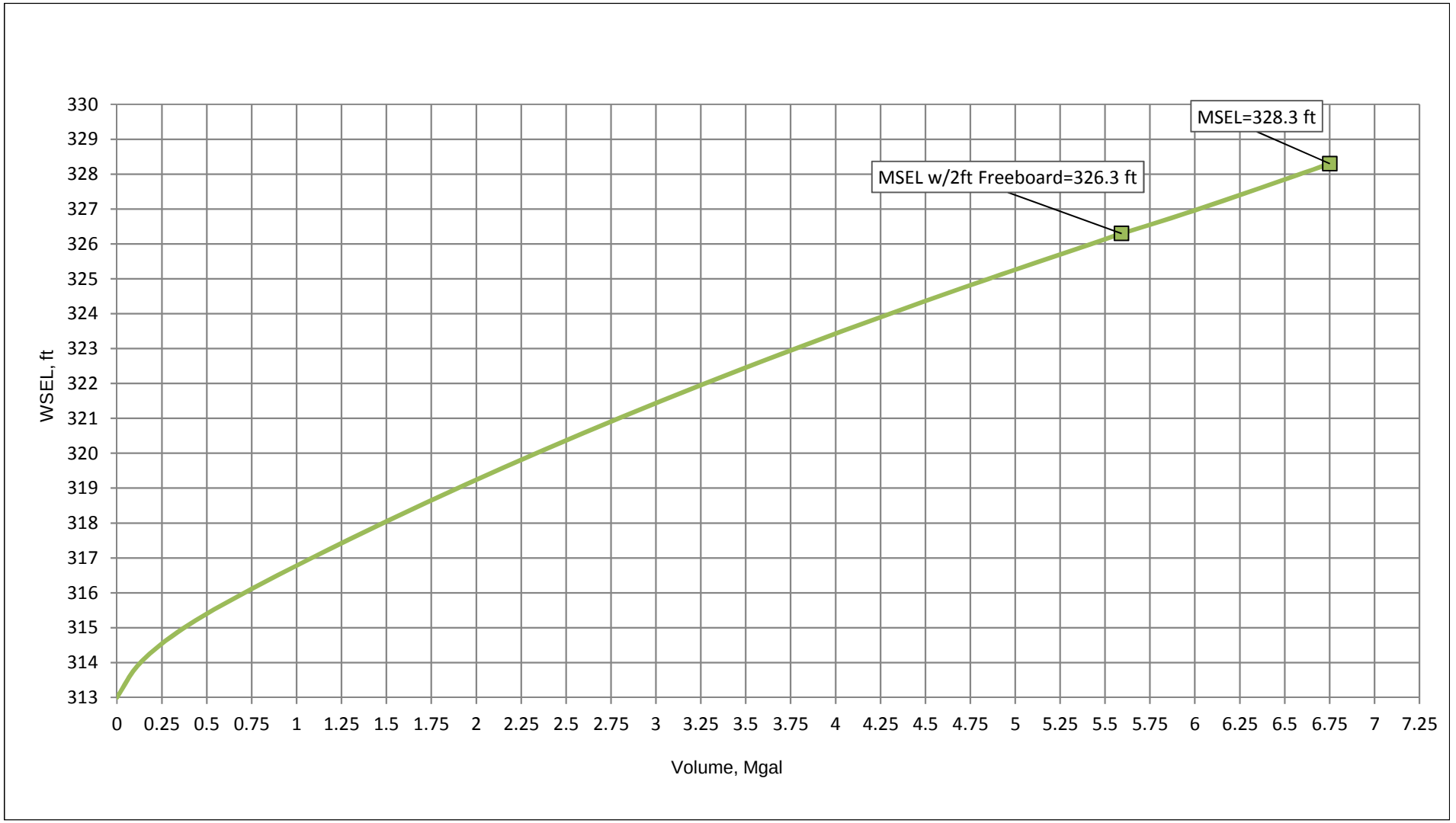


WEST YOST ASSOCIATES

Figure 3-13

Basin 2 Stage Storage Curve

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

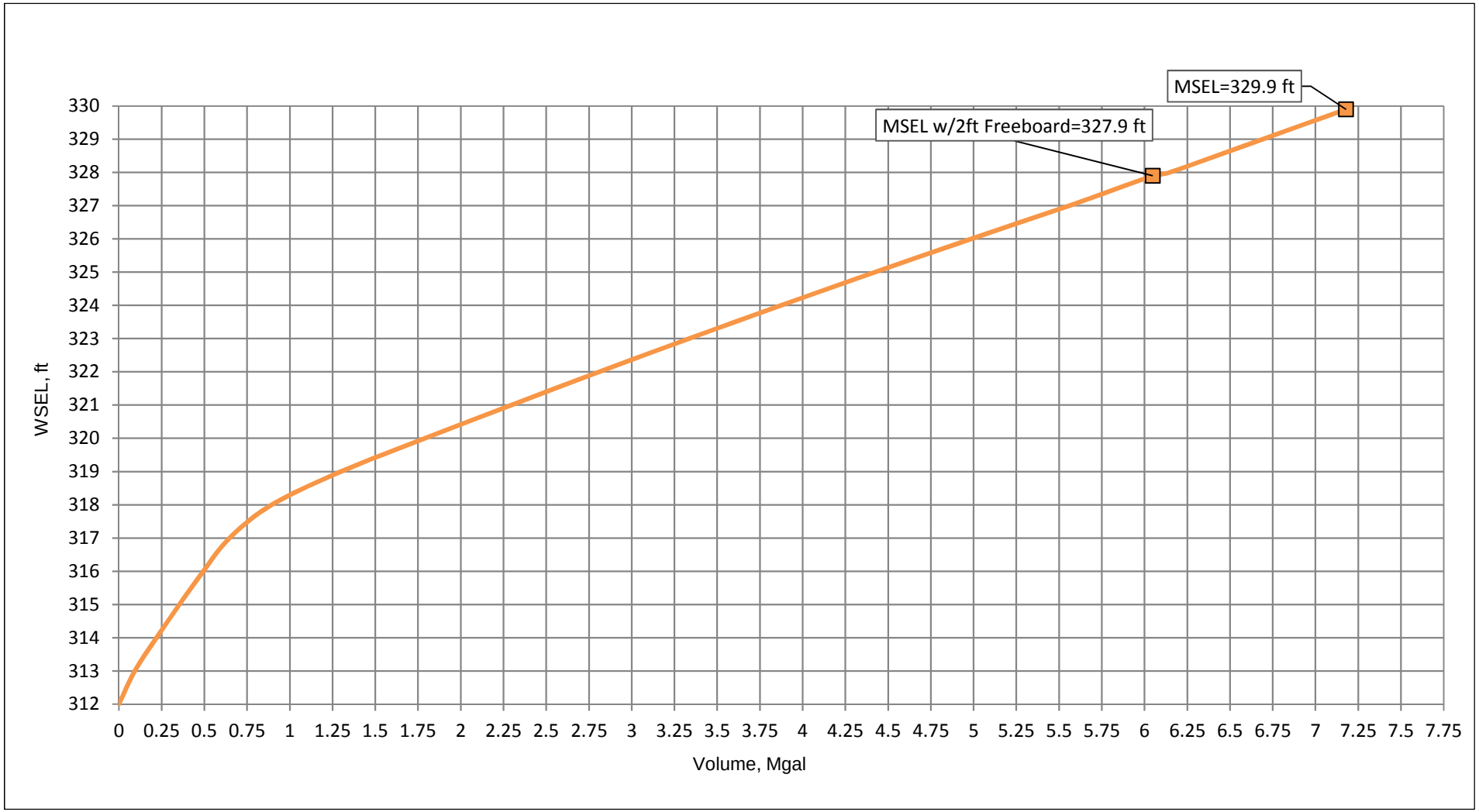


WEST YOST ASSOCIATES

Figure 3-14

Basin 3 Stage Storage Curve

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

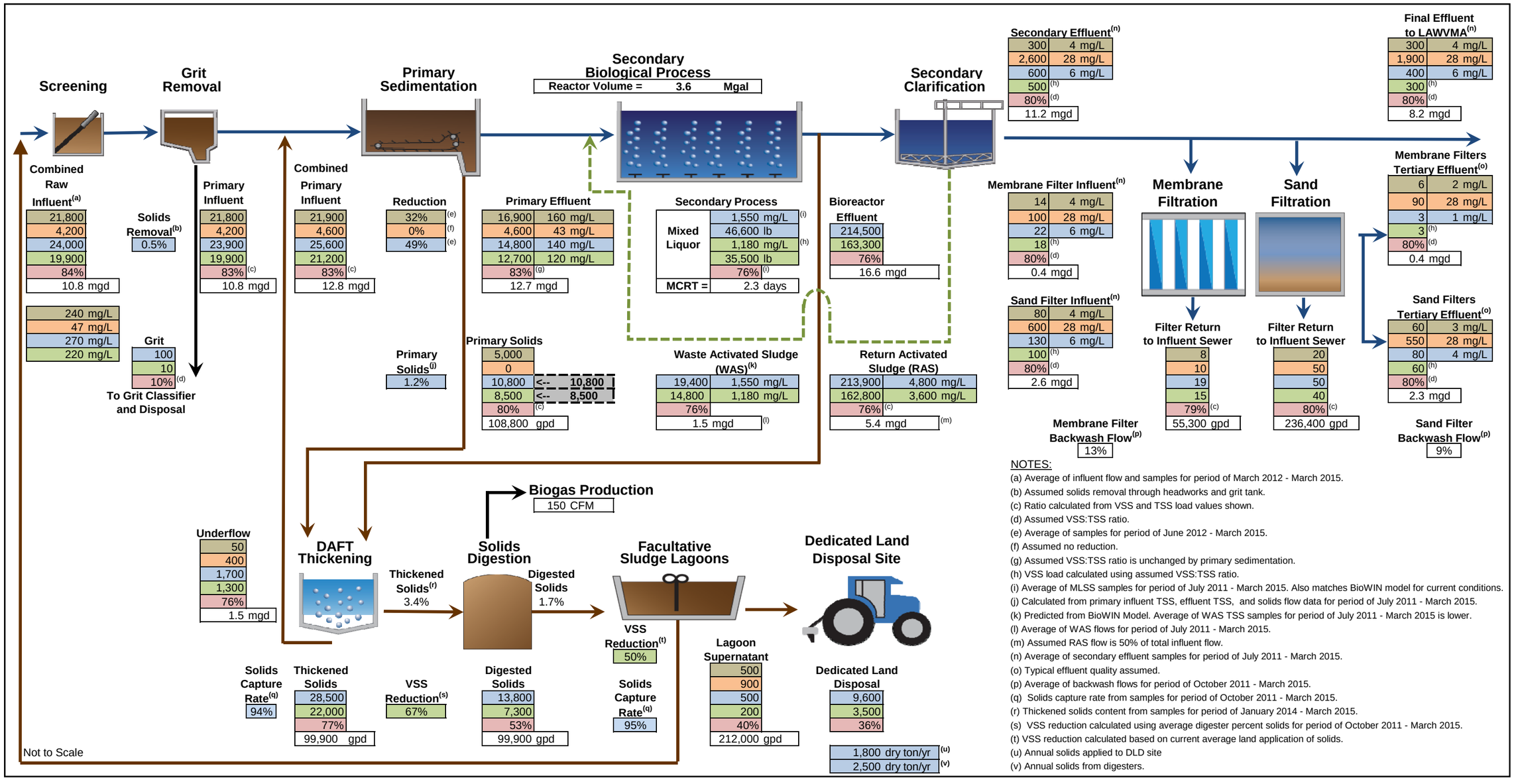


WEST YOST ASSOCIATES

Figure 3-15

Basin 4 Stage Storage Curve

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



NOTES:

(a) Average of influent flow and samples for period of March 2012 - March 2015.
(b) Assumed solids removal through headworks and grit tank.
(c) Ratio calculated from VSS and TSS load values shown.
(d) Assumed VSS:TSS ratio.
(e) Average of samples for period of June 2012 - March 2015.
(f) Assumed no reduction.
(g) Assumed VSS:TSS ratio is unchanged by primary sedimentation.
(h) VSS load calculated using assumed VSS:TSS ratio.
(i) Average of MLSS samples for period of July 2011 - March 2015. Also matches BioWIN model for current conditions.
(j) Calculated from primary influent TSS, effluent TSS, and solids flow data for period of July 2011 - March 2015.
(k) Predicted from BioWIN Model. Average of WAS TSS samples for period of July 2011 - March 2015 is lower.
(l) Average of WAS flows for period of July 2011 - March 2015.
(m) Assumed RAS flow is 50% of total influent flow.
(n) Average of secondary effluent samples for period of July 2011 - March 2015.
(o) Typical effluent quality assumed.
(p) Average of backwash flows for period of October 2011 - March 2015.
(q) Solids capture rate from samples for period of October 2011 - March 2015.
(r) Thickened solids content from samples for period of January 2014 - March 2015.
(s) VSS reduction calculated using average digester percent solids for period of October 2011 - March 2015.
(t) VSS reduction calculated based on current average land application of solids.
(u) Annual solids applied to DLD site
(v) Annual solids from digesters.

LEGEND:

1,000	BOD Value (lb/day, unless noted otherwise)
1,000	NH_4-N Value (lb/day, unless noted otherwise)
1,000	TSS Value (lb/day, unless noted otherwise)
1,000	VSS Value (lb/day, unless noted otherwise)
1,000	Assumed VSS percentage of TSS
1,000	Flow (units as indicated)
1,000	Values used for iterative process

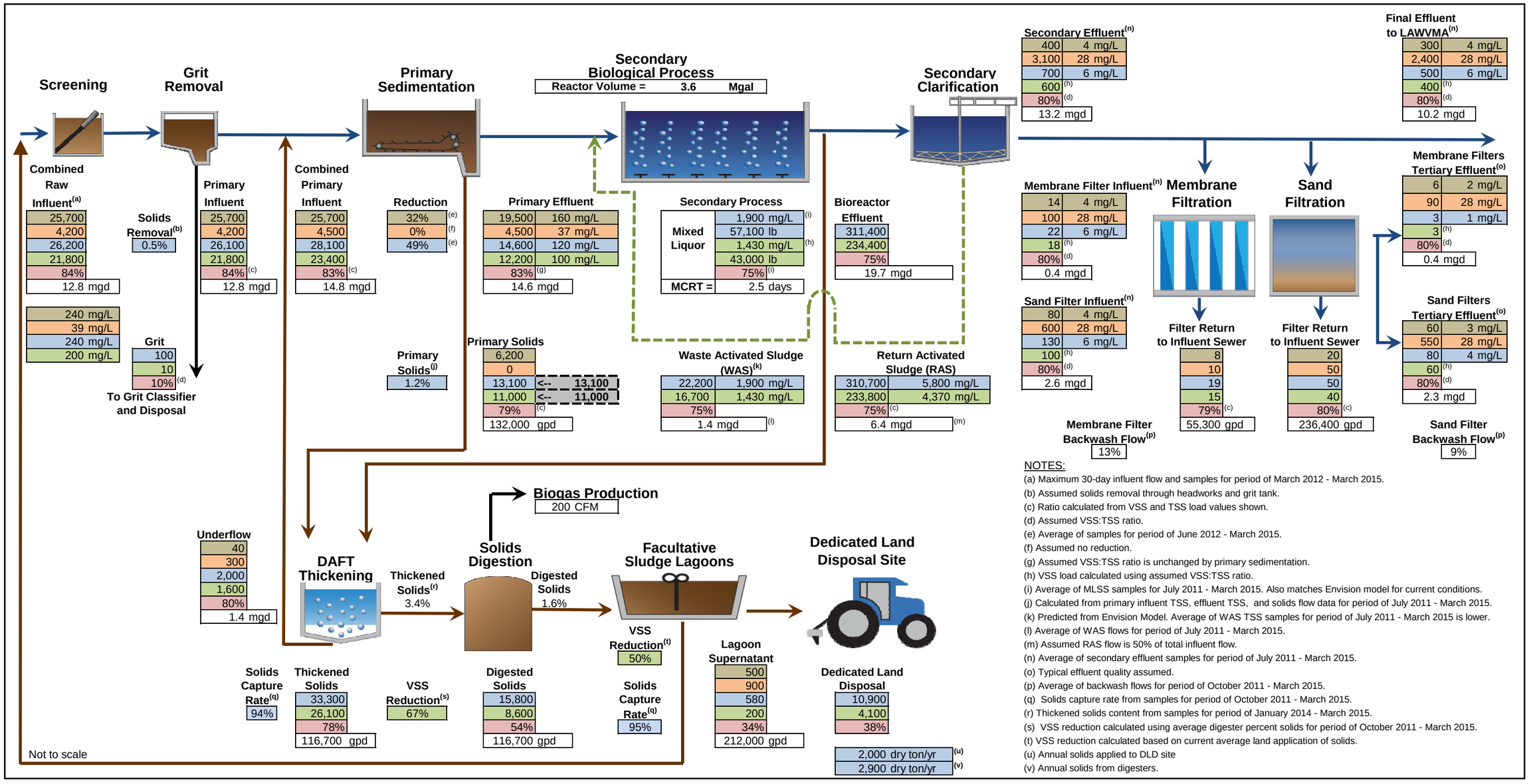
Solids flow through liquid treatment
Solids flow through solids treatment
Solids flow recycle streams
Solids flow external to mass balance (grit disposal)

WEST YOST ASSOCIATES
Consulting Engineers

Figure 3-16

Current Average Solids Balance

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



NOTES:

(a) Maximum 30-day influent flow and samples for period of March 2012 - March 2015.
(b) Assumed solids removal through headworks and grit tank.
(c) Ratio calculated from VSS and TSS load values shown.
(d) Assumed VSS:TSS ratio.
(e) Average of samples for period of June 2012 - March 2015.
(f) Assumed no reduction.
(g) Assumed VSS:TSS ratio is unchanged by primary sedimentation.
(h) VSS load calculated using assumed VSS:TSS ratio.
(i) Average of MLSS samples for July 2011 - March 2015. Also matches Envision model for current conditions.
(j) Calculated from primary influent TSS, effluent TSS, and solids flow data for period of July 2011 - March 2015.
(k) Predicted from Envision Model. Average of WAS TSS samples for period of July 2011 - March 2015 is lower.
(l) Average of WAS flows for period of July 2011 - March 2015.
(m) Assumed RAS flow is 50% of total influent flow.
(n) Average of secondary effluent samples for period of July 2011 - March 2015.
(o) Typical effluent quality assumed.
(p) Average of backwash flows for period of October 2011 - March 2015.
(q) Solids capture rate from samples for period of October 2011 - March 2015.
(r) Thickened solids content from samples for period of January 2014 - March 2015.
(s) VSS reduction calculated using average digester percent solids for period of October 2011 - March 2015.
(t) VSS reduction calculated based on current average land application of solids.
(u) Annual solids applied to DLD site
(v) Annual solids from digesters.

LEGEND:

1,000	BOD Value (lb/day, unless noted otherwise)
1,000	NH_4-N Value (lb/day, unless noted otherwise)
1,000	TSS Value (lb/day, unless noted otherwise)
1,000	VSS Value (lb/day, unless noted otherwise)
1,000	Assumed VSS percentage of TSS
1,000	Flow (units as indicated)
1,000	Values used for iterative process

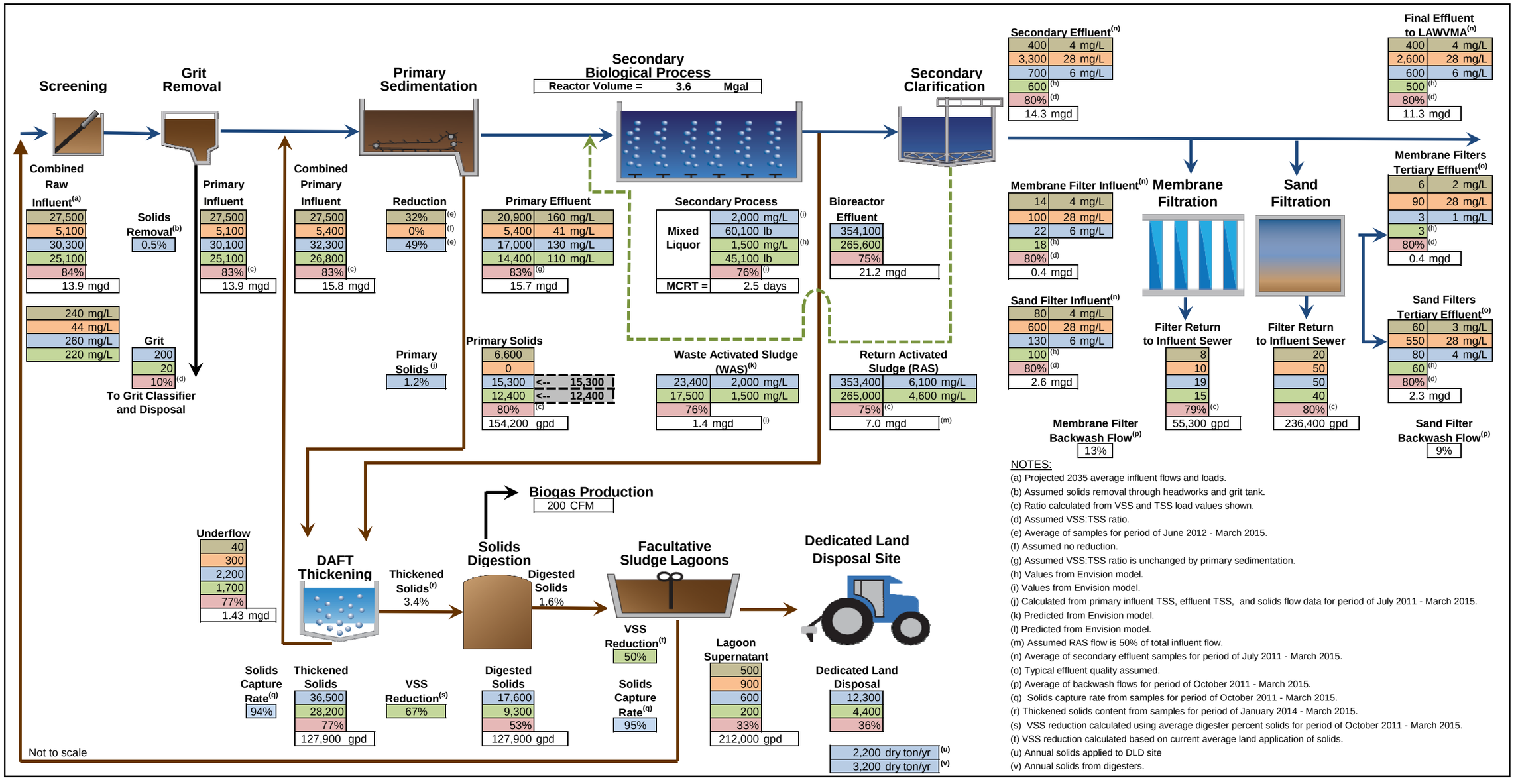
Solids flow through liquid treatment
Solids flow through solids treatment
Solids flow recycle streams
Solids flow external to mass balance (grit disposal)

WEST YOST ASSOCIATES
Consulting Engineers

Figure 3-17

Current Maximum 30-Day Solids Balance

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



NOTES:

(a) Projected 2035 average influent flows and loads.
(b) Assumed solids removal through headworks and grit tank.
(c) Ratio calculated from VSS and TSS load values shown.
(d) Assumed VSS:TSS ratio.
(e) Average of samples for period of June 2012 - March 2015.
(f) Assumed no reduction.
(g) Assumed VSS:TSS ratio is unchanged by primary sedimentation.
(h) Values from Envision model.
(i) Values from Envision model.
(j) Calculated from primary influent TSS, effluent TSS, and solids flow data for period of July 2011 - March 2015.
(k) Predicted from Envision model.
(l) Predicted from Envision model.
(m) Assumed RAS flow is 50% of total influent flow.
(n) Average of secondary effluent samples for period of July 2011 - March 2015.
(o) Typical effluent quality assumed.
(p) Average of backwash flows for period of October 2011 - March 2015.
(q) Solids capture rate from samples for period of October 2011 - March 2015.
(r) Thickened solids content from samples for period of January 2014 - March 2015.
(s) VSS reduction calculated using average digester percent solids for period of October 2011 - March 2015.
(t) VSS reduction calculated based on current average land application of solids.
(u) Annual solids applied to DLD site
(v) Annual solids from digesters.

LEGEND:

1,000	BOD Value (lb/day, unless noted otherwise)
1,000	NH_4-N Value (lb/day, unless noted otherwise)
1,000	TSS Value (lb/day, unless noted otherwise)
1,000	VSS Value (lb/day, unless noted otherwise)
1,000	Assumed VSS percentage of TSS
1,000	Flow (units as indicated)
1,000	Values used for iterative process

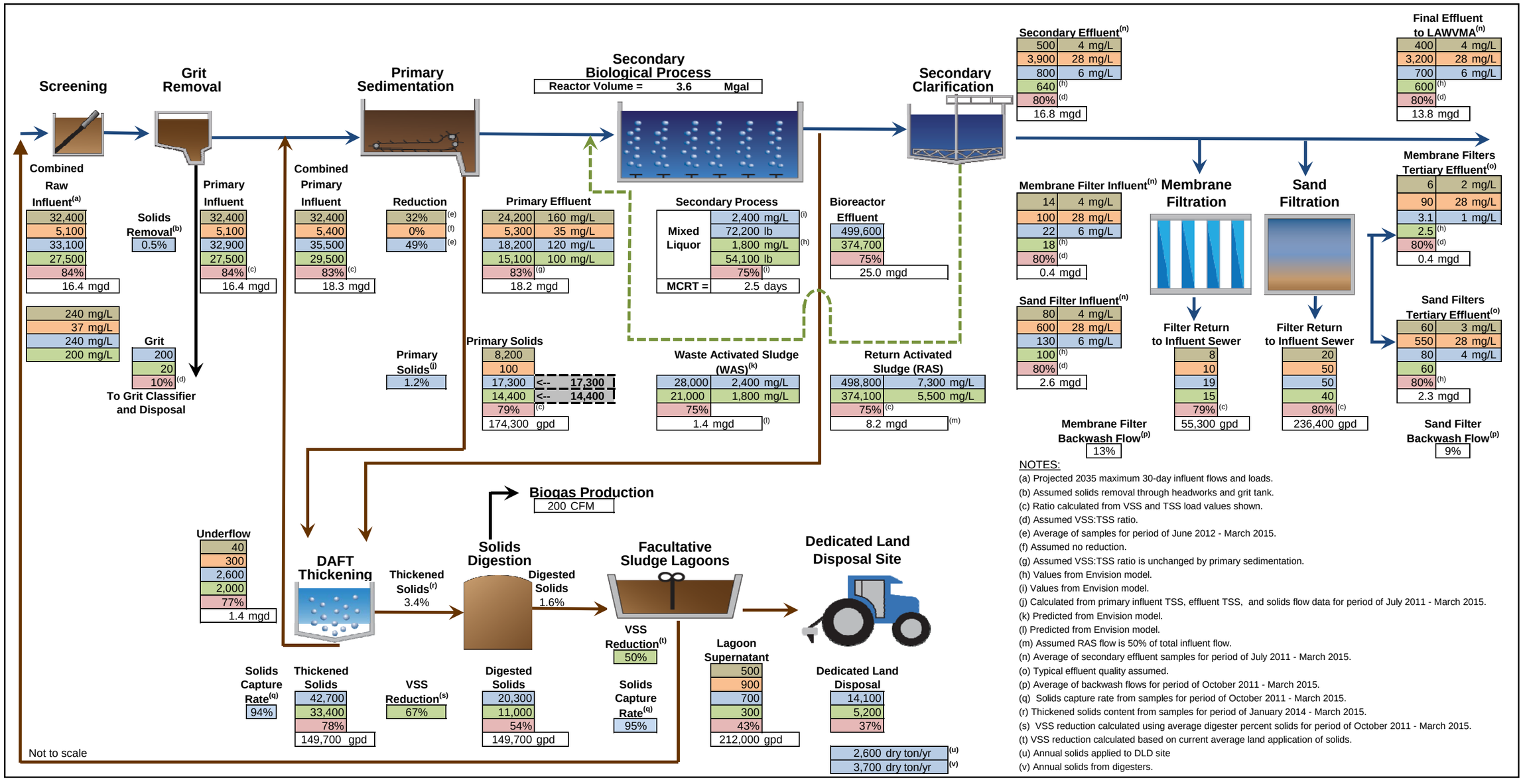
→ Solids flow through liquid treatment
→ Solids flow through solids treatment
⇢ Solids flow recycle streams
→ Solids flow external to mass balance (grit disposal)

WEST YOST ASSOCIATES
Consulting Engineers

Figure 3-18

2035 Average Solids Balance

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



NOTES:

(a) Projected 2035 maximum 30-day influent flows and loads.
(b) Assumed solids removal through headworks and grit tank.
(c) Ratio calculated from VSS and TSS load values shown.
(d) Assumed VSS:TSS ratio.
(e) Average of samples for period of June 2012 - March 2015.
(f) Assumed no reduction.
(g) Assumed VSS:TSS ratio is unchanged by primary sedimentation.
(h) Values from Envision model.
(i) Values from Envision model.
(j) Calculated from primary influent TSS, effluent TSS, and solids flow data for period of July 2011 - March 2015.
(k) Predicted from Envision model.
(l) Predicted from Envision model.
(m) Assumed RAS flow is 50% of total influent flow.
(n) Average of secondary effluent samples for period of July 2011 - March 2015.
(o) Typical effluent quality assumed.
(p) Average of backwash flows for period of October 2011 - March 2015.
(q) Solids capture rate from samples for period of October 2011 - March 2015.
(r) Thickened solids content from samples for period of January 2014 - March 2015.
(s) VSS reduction calculated using average digester percent solids for period of October 2011 - March 2015.
(t) VSS reduction calculated based on current average land application of solids.
(u) Annual solids applied to DLD site
(v) Annual solids from digesters.

LEGEND:

1,000	BOD Value (lb/day, unless noted otherwise)
1,000	NH_4-N Value (lb/day, unless noted otherwise)
1,000	TSS Value (lb/day, unless noted otherwise)
1,000	VSS Value (lb/day, unless noted otherwise)
1,000	Assumed VSS percentage of TSS
1,000	Flow (units as indicated)
1,000	Values used for iterative process

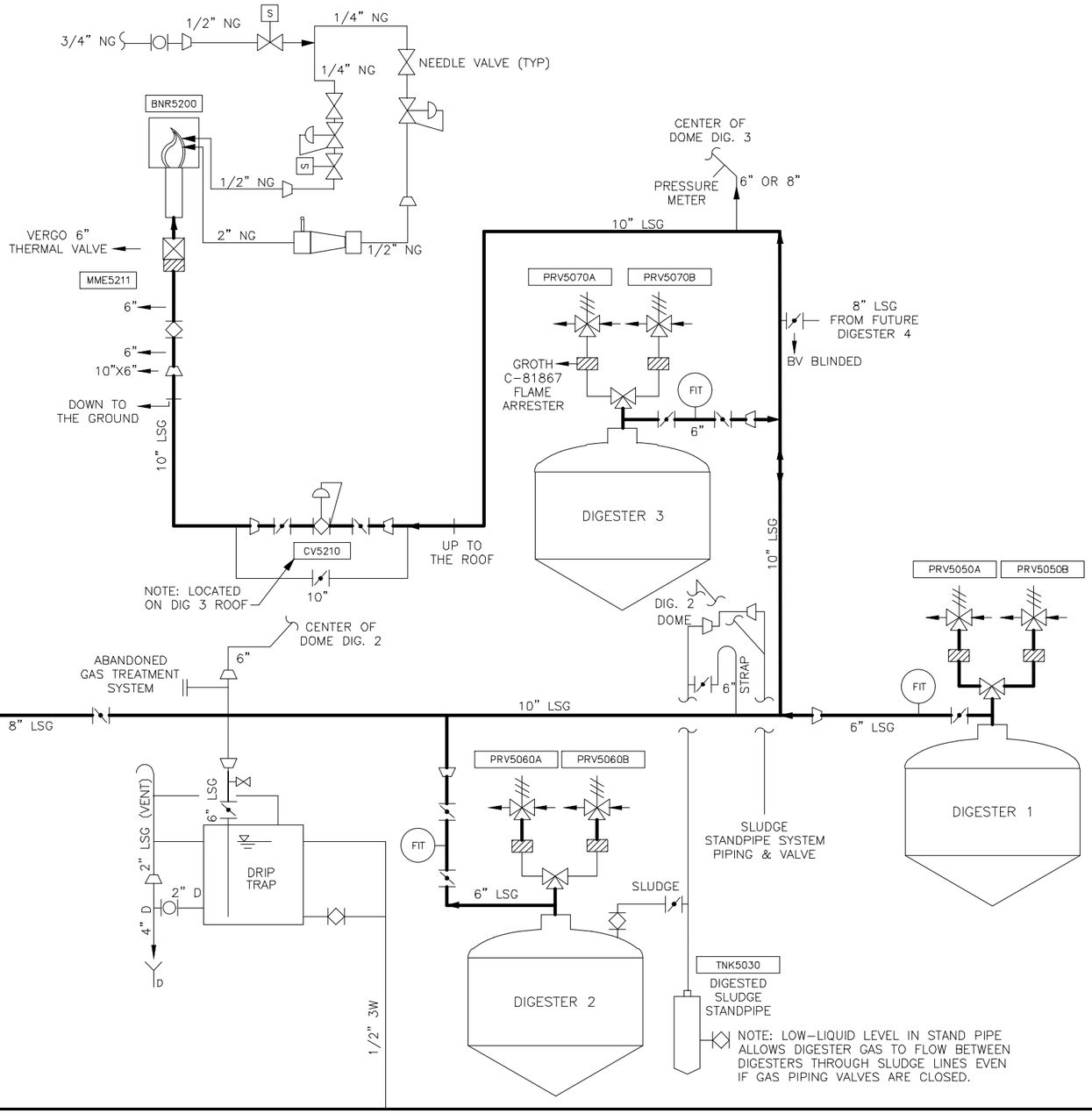
Solids flow through liquid treatment
Solids flow through solids treatment
Solids flow recycle streams
Solids flow external to mass balance (grit disposal)

WEST YOST ASSOCIATES
Consulting Engineers

Figure 3-19

2035 Maximum 30-Day Solids Balance

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



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LEGEND

- ACTUATED VALVE, A=AIR M=MOTOR S=SOLENOID
- ACTUATOR
- ASSET ID
- BALL VALVE
- BLIND FLANGE
- BUTTERFLY VALVE
- EQUIPMENT DRAIN
- FLAME ARRESTER
- FLAME TRAP
- FLARE
- GATE VALVE
- INCREASER / REDUCER
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PLUG VALVE
- PRESSURE REGULATOR
- THREE WAY VALVE
- SAFETY SHUT OFF

ABBREVIATIONS:

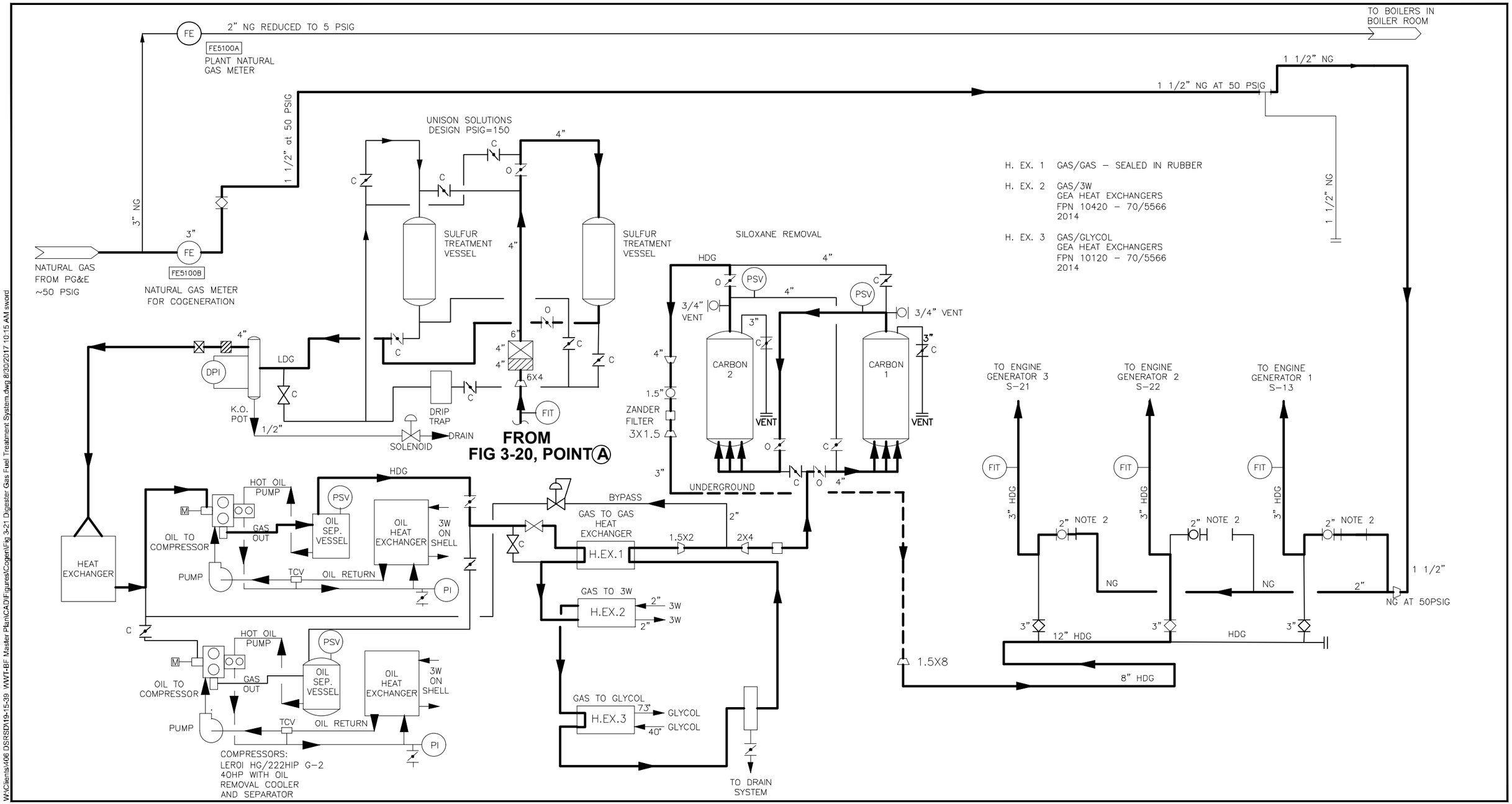
D	DRAIN
DIG	DIGESTER
FIT	FLOW INDICATING TRANSMITTER
LDG	LOW PRESSURE DIGESTER GAS
LSG	LOW PRESSURE SLUDGE (DIGESTER GAS)
NG	NATURAL GAS
RV	WEIGHTED PRESSURE/VACUUM RELIEF VALVE
3W	PLANT EFFLUENT FOR COOLING

WEST YOST ASSOCIATES

Figure 3-20

Low Pressure Digester Gas System

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



WA\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-21 Digester Gas Fuel Treatment System.dwg 8/30/2017 10:15 AM sword

LEGEND

- ACTUATED VALVE, A=AIR M=MOTOR S=SOLENOID
- ASSET ID
- BALL VALVE
- BLIND FLANGE
- BUTTERFLY VALVE
- FLAME ARRESTER
- FLAME TRAP
- GATE VALVE
- INCREASER / REDUCER
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PRESSURE REGULATOR
- PLUG VALVE
- PUMP
- ROTARY LOBE COMPRESSOR
- ROTARY LOBE PUMP
- VESSEL/ TANK

ABBREVIATIONS:

C/NC	CLOSED / NORMALLY CLOSED	NG	NATURAL GAS
DG		O/NO	OPEN / NORMALLY OPEN
DPI	DIFFERENTIAL PRESSURE INDICATOR	PI	PRESSURE INDICATOR (PRESSURE GAUGE)
FE	FLOW ELEMENT	PIT	PRESSURE INDICATOR TRANSMITTER
FIT	FLOW INDICATOR TRANSMITTER	PSV	PRESSURE SAFETY VALVE
HDG	HIGH PRESSURE DIGESTER GAS	TCV	TEMPERATURE CONTROL VALVE
H.EX	HEAT EXCHANGER	3W	PLANT EFFLUENT FOR COOLING
K.O. POT	KNOCK OUT POT		
LDG	LOW PRESSURE DIGESTER GAS		

NOTES:

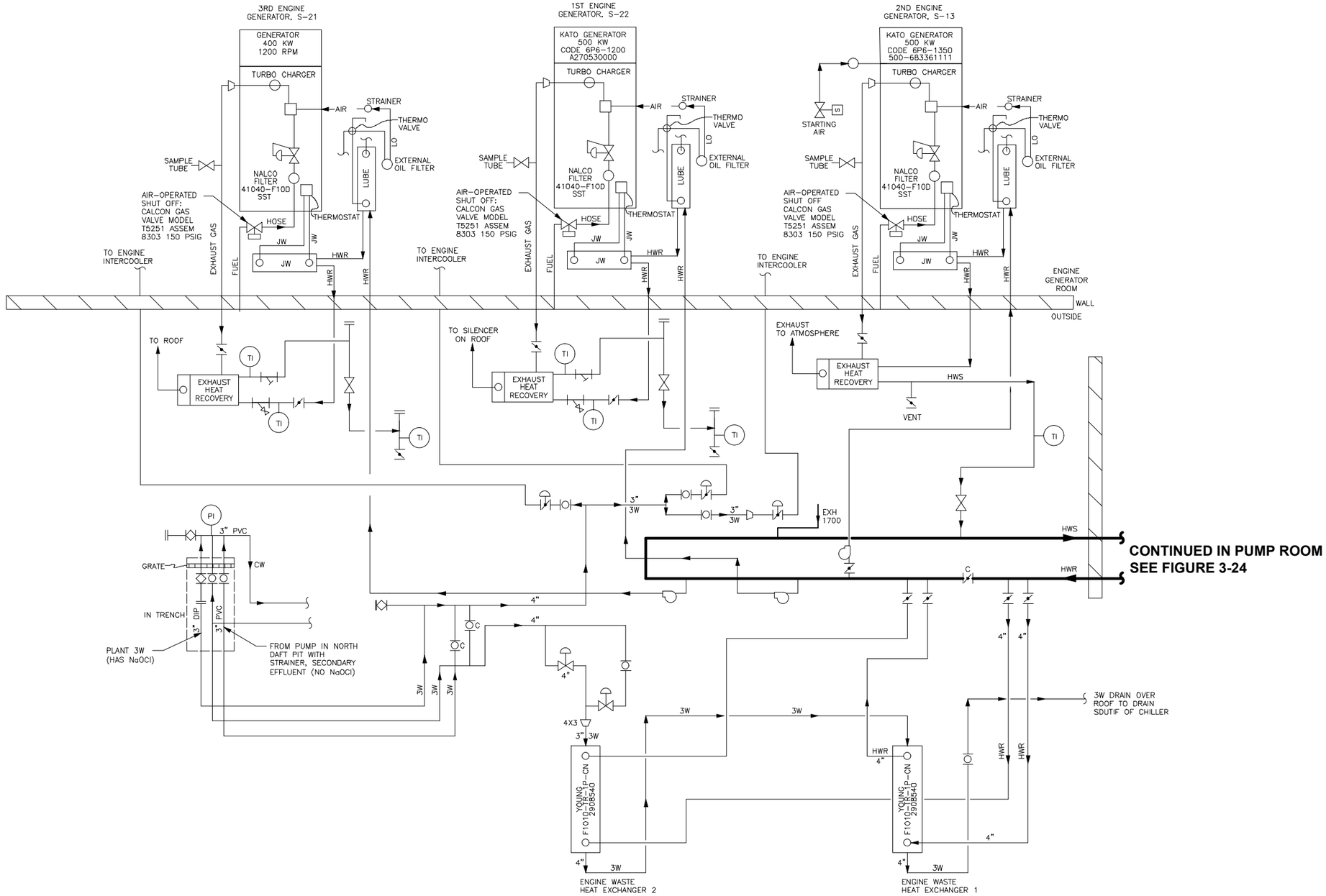
1. PIPING AT SULFUR TREATMENT MAY NEED FURTHER FIELD CHECKING.
2. SPOOL PIECE FOR NATURAL GAS CONNECTION CAN BI REMOVED IF ENGINE IS U· ONLY DIGESTER GAS.

WEST YOST ASSOCIATES

Figure 3-21

Digester Gas Fuel Treatment System

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



LEGEND

- ACTUATED VALVE, A=AIR M=MOTOR S=SOLENOID
- ACTUATOR
- BALL VALVE
- BLIND FLANGE
- BUTTERFLY VALVE
- FLANGE
- GATE VALVE
- INCREASER / REDUCER
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PLUG VALVE
- PRESSURE REGULATOR
- PUMP
- STRAINER
- STRAINER WITH VALVE

ABBREVIATIONS:

C/NC	CLOSED / NORMALLY CLOSED	JW	JACKET WATER
CW	COLD WATER	LO	LUBE OIL
DIP	DUCTILE IRON PIPE	PI	PRESSURE INDICATOR (PRESSURE GAUGE)
EXH	EXHAUST	PVC	POLYVINYL CHLORIDE
HWS	HOT WATER SUPPLY	TI	TEMPERATURE INDICATOR
HWR	HOT WATER RETURN	3W	PLANT EFFLUENT FOR COOLING

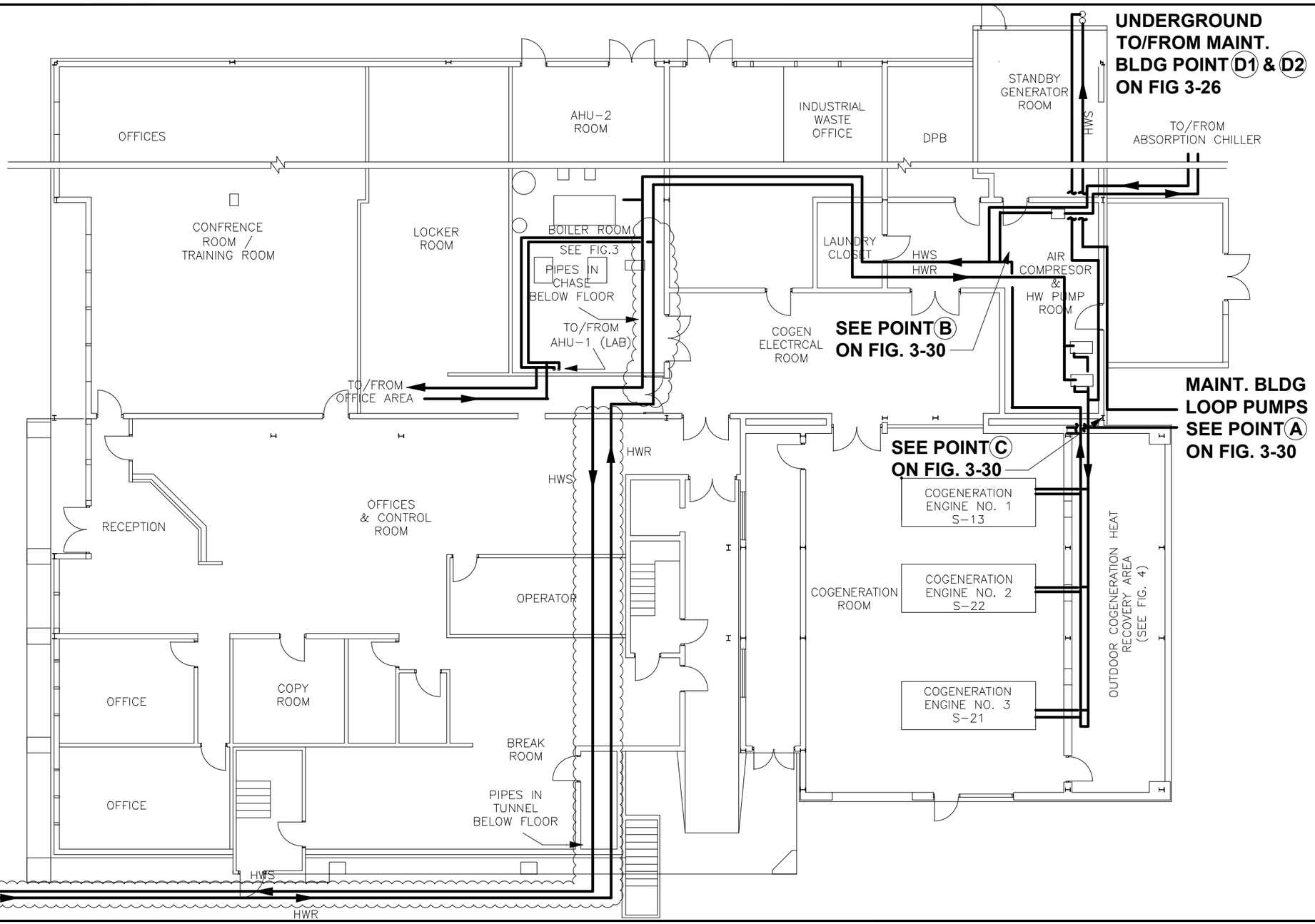
WEST YOST ASSOCIATES

Figure 3-22

Heat Recovery System At Cogeneration Engines

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-22 Heat Recovery at Cogeneration.dwg 8/30/2017 10:20 AM sward



ABBREVIATIONS:

AHU	AIR HANDLING UNIT
DPB	DISTRIBUTION PANEL B
HWS	HOT WATER SUPPLY
HWR	HOT WATER RETURN

NOTE:

ONLY MAIN HW LOOP SHOWN. SEE SUBLOOPS AND DETAILS ON OTHER FIGURES

N

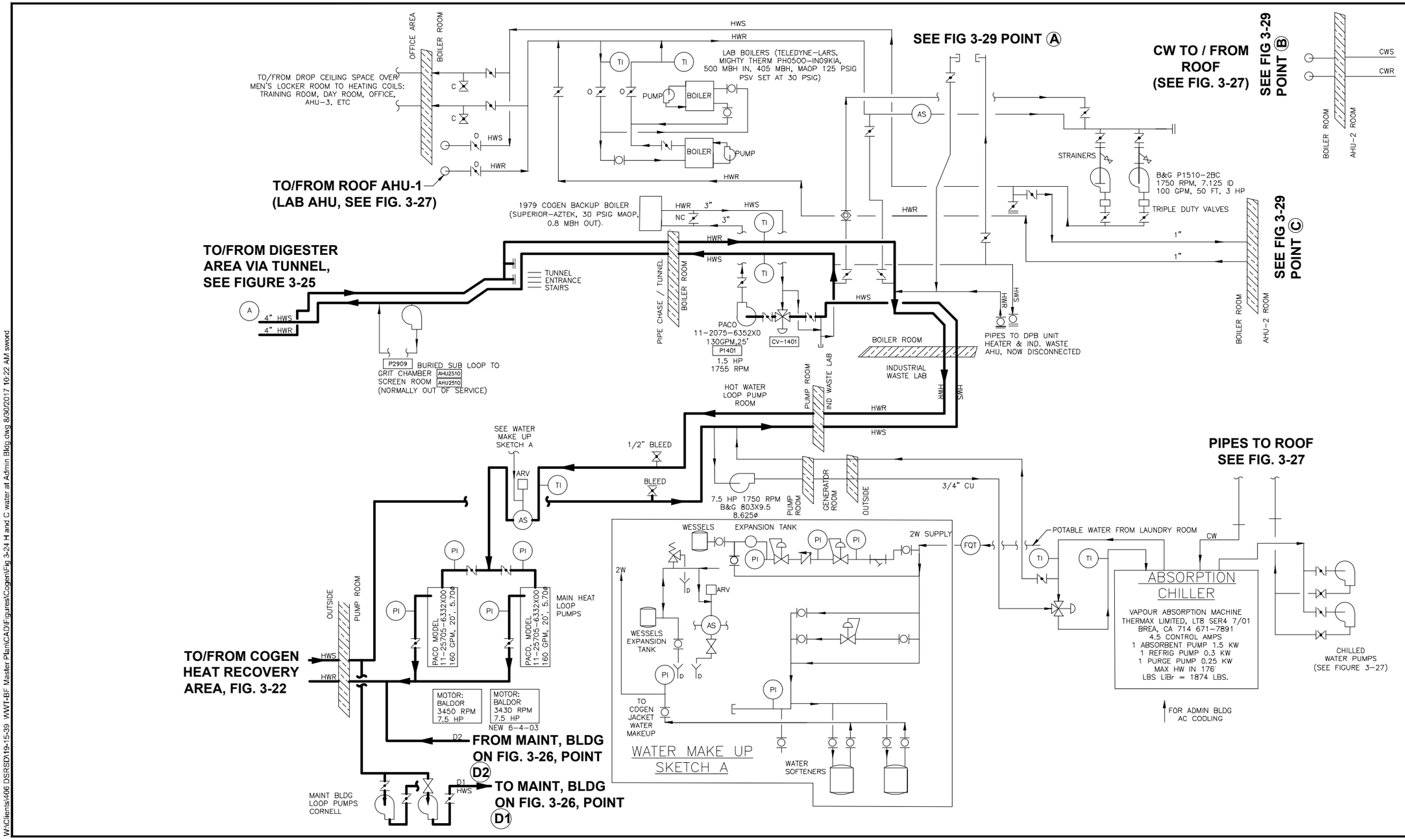
WEST YOST ASSOCIATES

Figure 3-23

Hot Water System Overview In Administration Building

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-23 Hot Water System Overview in Admin BLDG.dwg 10/14/2016 2:02 PM bgong



LEGEND

- ACTUATOR
- ASSET ID
- BALL VALVE
- BLIND FLANGE
- BUTTERFLY VALVE
- CHECK VALVE
- CAP (SCREWED ON)
- EQUIPMENT DRAIN
- GATE VALVE
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PRESSURE REGULATOR
- PRESSURE SAFETY VALVE
- PUMP
- STRAINER
- STRAINER WITH VALVE
- THREE WAY VALVE
- VESSEL/ TANK

ABBREVIATIONS:

AS	AIR SEPARATOR	HWS	HOT WATER SUPPLY
ARV	AIR RELEASE VALVE	HWR	HOT WATER RETURN
C/NC	CLOSED / NORMALLY CLOSED	O/NO	OPEN / NORMALLY OPEN
CU	COPPER	PI	PRESSURE INDICATOR (PRESSURE GAUGE)
CW	COLD WATER	TI	TEMPERATURE INDICATOR
CWS	COLD WATER SUPPLY	1W	DRINKING WATER
CWR	COLD WATER RETURN	2W	POTABLE WATER PASSED THROUGH AIR
FQ	FLOW TOTALIZER		GAP TANK (NOT POTABLE)
FQT	FLOW TOTALIZER TRANSMITTER		

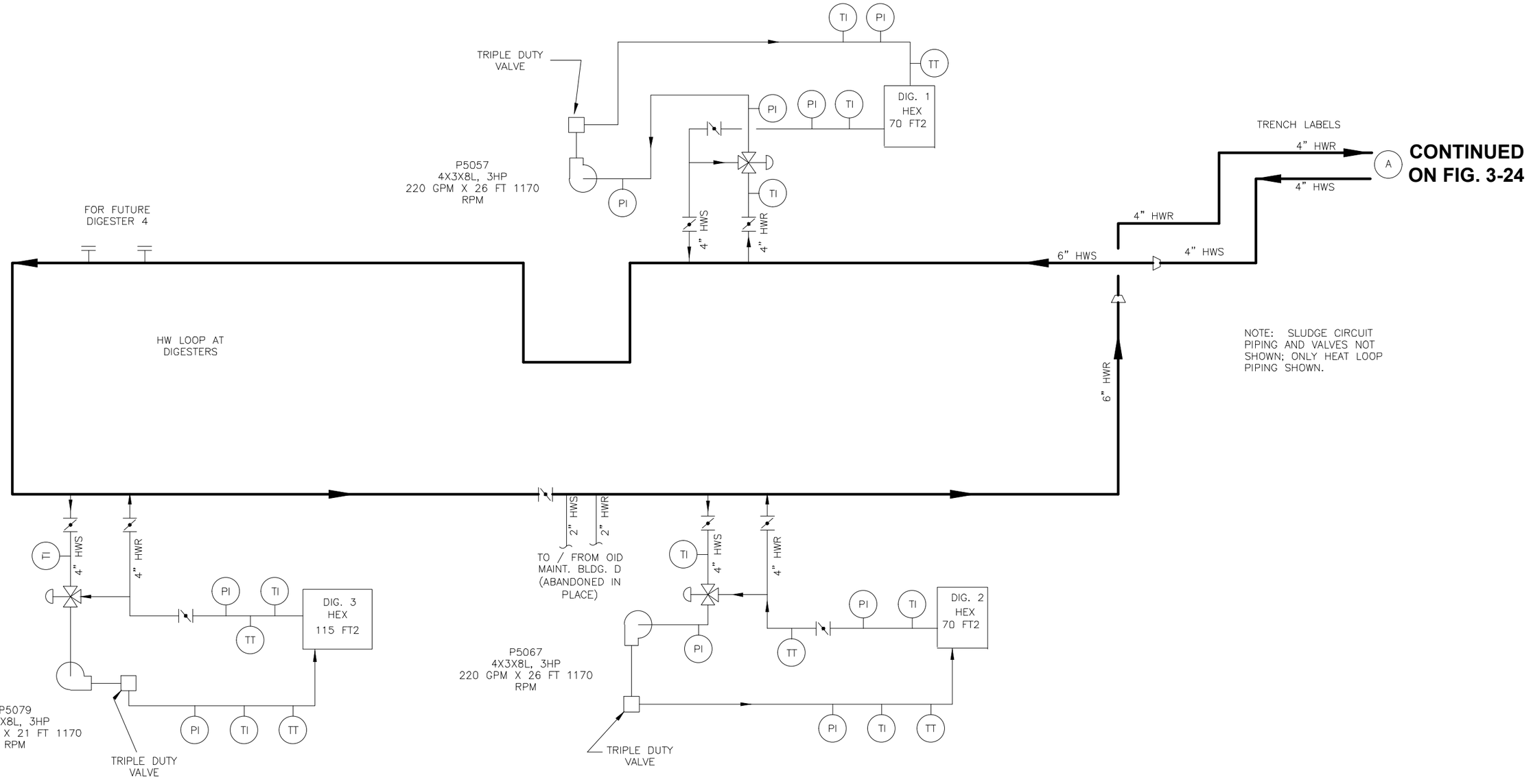
WEST YOST ASSOCIATES

Figure 3-24

Hot & Chilled Water Systems At Administration Building

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-24 H and C water at Admin Bldg.dwg 8/30/2017 10:22 AM sword



LEGEND

- ACTUATOR
- BLIND FLANGE
- BUTTERFLY VALVE
- EQUIPMENT DRAIN
- GATE VALVE
- INCREASER / REDUCER
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PLUG VALVE
- PUMP
- THREE WAY VALVE

ABBREVIATIONS:

HWR	HOT WATER RETURN
HWS	HOT WATER SUPPLY
PI	PRESSURE INDICATOR (PRESSURE GAUGE)
TI	TEMPERATURE INDICATOR
TT	TEMPERATURE TRANSMITTER

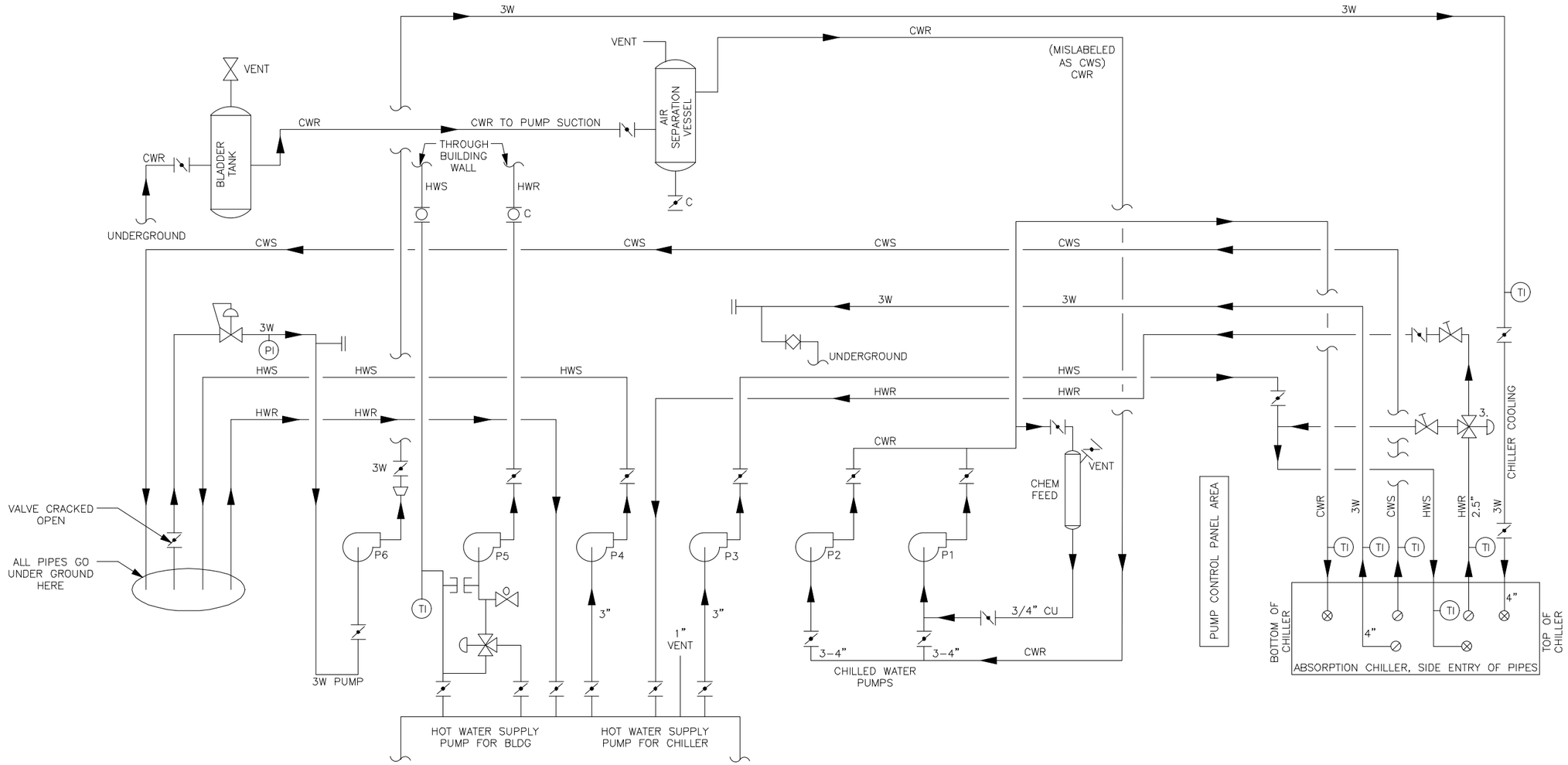
WEST YOST ASSOCIATES

Figure 3-25

Digester Heat Loop At Digesters

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-25 Digester Heat Loop at Digesters.dwg 9/18/2017 12:16 PM bgong



UNDERGROUND POINT (D2), HWR (SEE FIG. 3-24)

UNDERGROUND POINT (D1), HWS (SEE FIG. 3-24)

PUMP NAME PLATE DATA:

P6	P5	P4	P3	P2 & P1
MFGR: PATTERSON	MFGR: PATTERSON	MFGR: PATTERSON	MFGR: PATTERSON	MFGR: PATTERSON
MODEL E3P7A	MODEL E125BA-1	MODEL E2F7A-1	MODEL E25J7A-1	MODEL E2F7A-1
GPM=300, H=30FT	GPM=38, H=25FT	GPM=115, H=40FT	GPM=170, H=25FT	GPM=100, H=30FT
MOTOR: BALDOR	MOTOR: BALDOR	MOTOR: BALDOR	MOTOR: BALDOR	MOTOR: BALDOR
HP=5	HP=1	HP=3	HP=2	HP=1.5
RPM=1750	RPM=1750	RPM=1755	RPM=1720	RPM=1760
SF=1.15	SF=1.15	SF=1.15	SF=1.15	SF=1.15

CHILLER DATA:

MFGR: VAPOR ABSORPTION MACHINE BY THERMAX LTD, INDIA
MODEL LT8, SN 37-2009
PUMP KW:
ABSORBANT=1.1 KW
REFRIGERANT=0.3 KW
PURGE PUMP=0.75KW
WATER CHARGE=T1LB
LIBR CHARGE=1190LB

LEGEND

- ACTUATOR
- BALL VALVE
- BLIND FLANGE
- BUTTERFLY VALVE
- CAP (SCREWED ON)
- GATE VALVE
- INCREASER / REDUCER
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PLUG VALVE
- PIPE OUT OF BUILDING WALL OR FROM UNDERGROUND
- PIPE INTO BUILDING WALL OR TO UNDERGROUND
- PUMP
- PRESSURE REGULATOR
- THREE WAY VALVE
- VESSEL/ TANK

ABBREVIATIONS:

C/NC	NORMALLY CLOSED	SF	SERVICE FACTOR
CU	COPPER	TI	TEMPERATURE INDICATOR
CWR	COLD WATER RETURN	3W	PLANT EFFLUENT FOR COOLING
CWS	COLD WATER SUPPLY		
HWR	HOT WATER RETURN		
HWS	HOT WATER SUPPLY		
O/NO	NORMALLY OPEN		
PI	PRESSURE INDICATOR (PRESSURE GAUGE)		

NOTES:

1. SCHEMATIC BASED ON FIELD OBSERVATION OF VISIBLE/ACCESSIBLE ITEMS IN SEPT 2015
2. P3 WAS VIBRATING EXCESSIVELY
3. NEED TO FIELD CHECK PIPES TO THIS VALVE

WEST YOST ASSOCIATES

Figure 3-26

Hot Water And Chilled Water At Maintenance Building

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\15-15-39 WWTP Master Plan\CAD Figures\Cogen\Fig 3-26 Hot Water and Chilled Water at Main. Bldg. dwg 8/30/2017 10:26 AM svarot





LEGEND

- BUTTERFLY VALVE
- INCREASER / REDUCER
- PUMP

ABBREVIATIONS:

AHU	AIR HANDLING UNIT
CW	COLD WATER
CWR	COLD WATER RETURN
CWS	COLD WATER SUPPLY
HW	HOT WATER
HWR	HOT WATER RETURN
HWS	HOT WATER SUPPLY

NOTES:

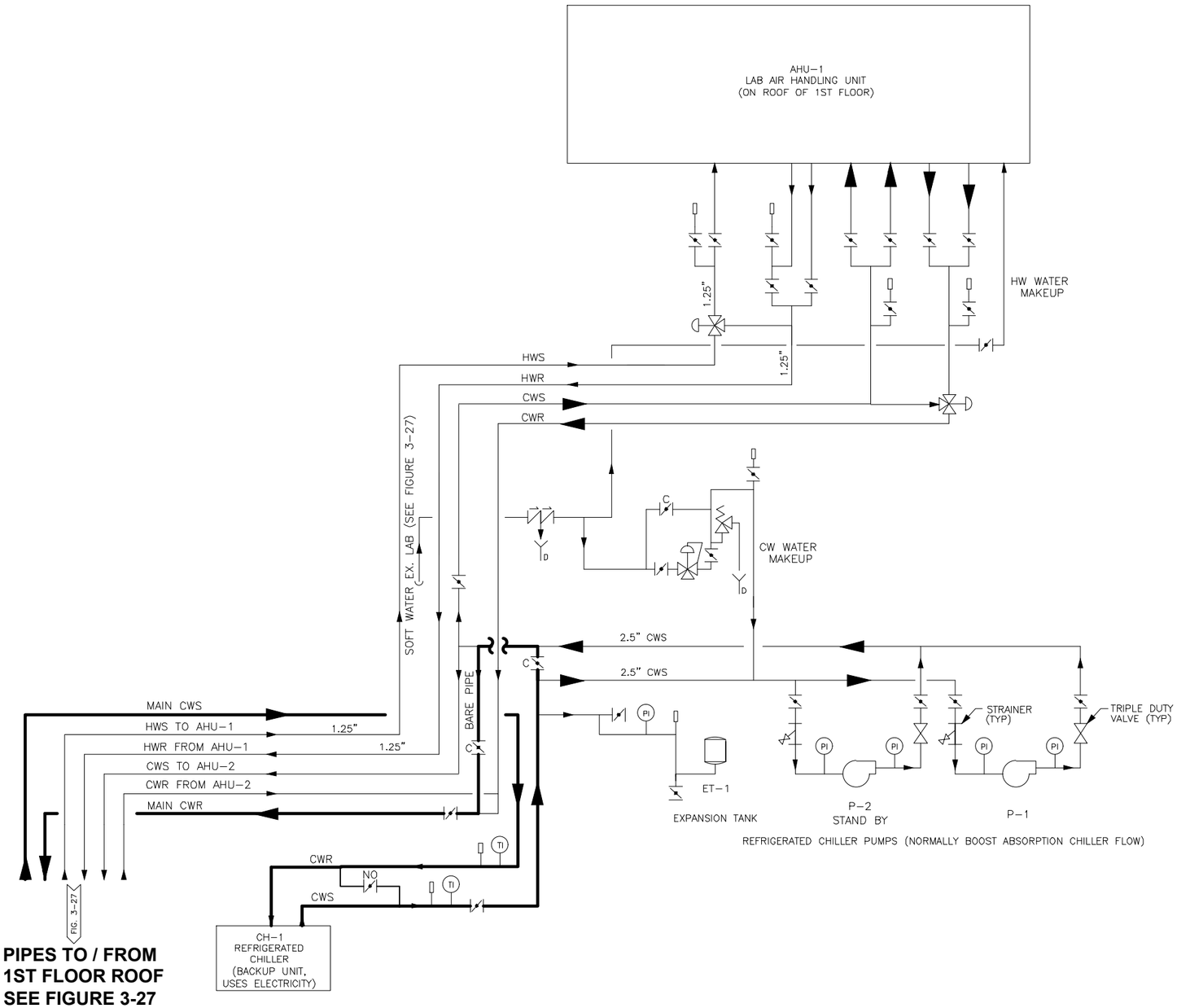
1. CHILLED WATER SYSTEM SHOWN AFTER PUMP RELOCATION
2. ONLY PARTIAL DETAILS OF HEATING WATER PIPING SHOWN ON ROOFS.
3. EXISTING BELOW GRADE HW & CW PIPES ARE ABANDONED IN PLACE
4. SEE FIG. 3 FOR HOT WATER PIPING AT CHILLER
5. CHILLER 3W COOLING PIPES NOT SHOWN

WEST YOST ASSOCIATES

Figure 3-27
Chilled Water Piping 1st Floor Roof Of Administration Building

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-27 Chilled Water.dwg 8/30/2017 10:27 AM sword



LEGEND

- AIR RELIEF
- ACTUATOR
- BUTTERFLY VALVE
- EQUIPMENT DRAIN
- CHECK VALVE
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PRESSURE REGULATOR
- PRESSURE SAFETY VALVE
- PUMP
- STRAINER WITH VALVE
- THREE WAY VALVE
- VESSEL/ TANK

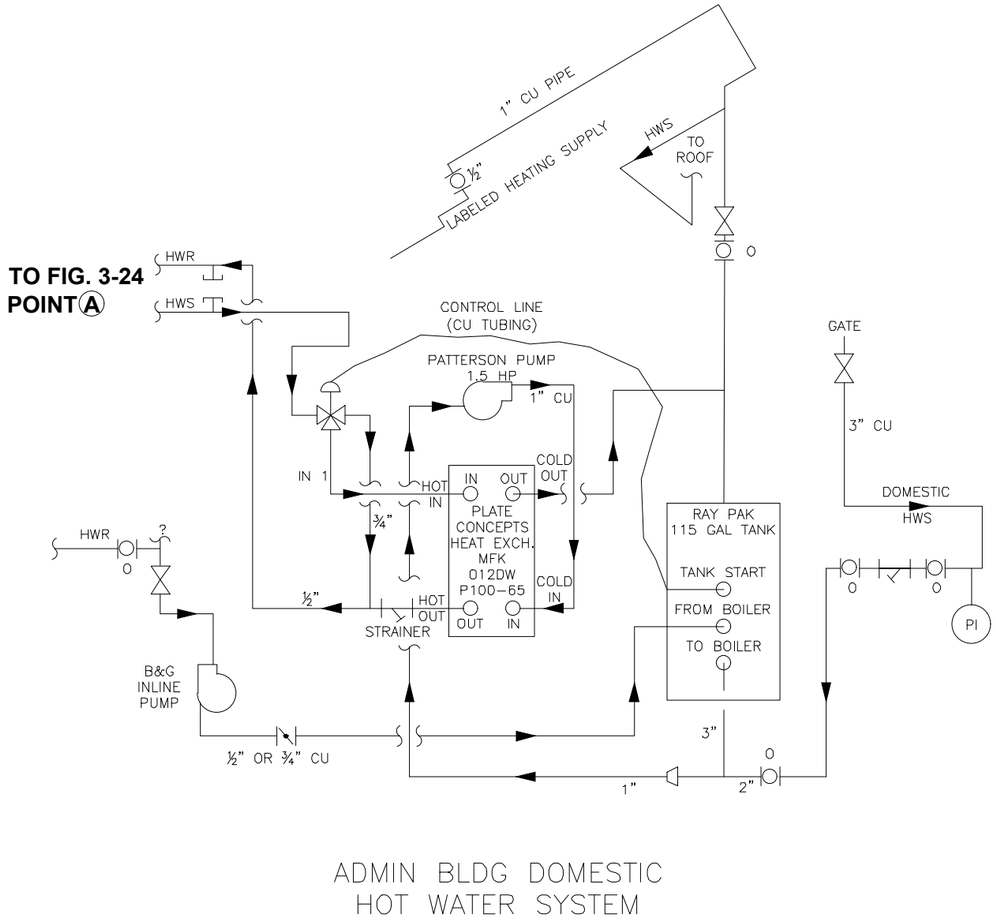
ABBREVIATIONS:

AHU	AIR HANDLING UNIT	O/NO	OPEN / NORMALLY OPEN
C/NC	CLOSED / NORMALLY CLOSED	PI	PRESSURE INDICATOR (PRESSURE GAUGE)
CW	COLD WATER	TI	TEMPERATURE INDICATOR
CWR	COLD WATER RETURN		
CWS	COLD WATER SUPPLY		
HWR	HOT WATER RETURN		
HWS	HOT WATER SUPPLY		

WEST YOST ASSOCIATES

Figure 3-28
Chilled Water Piping
2nd Floor Roof Of
Administration Building
Dublin San Ramon Services District
Wastewater Treatment
Master Plan

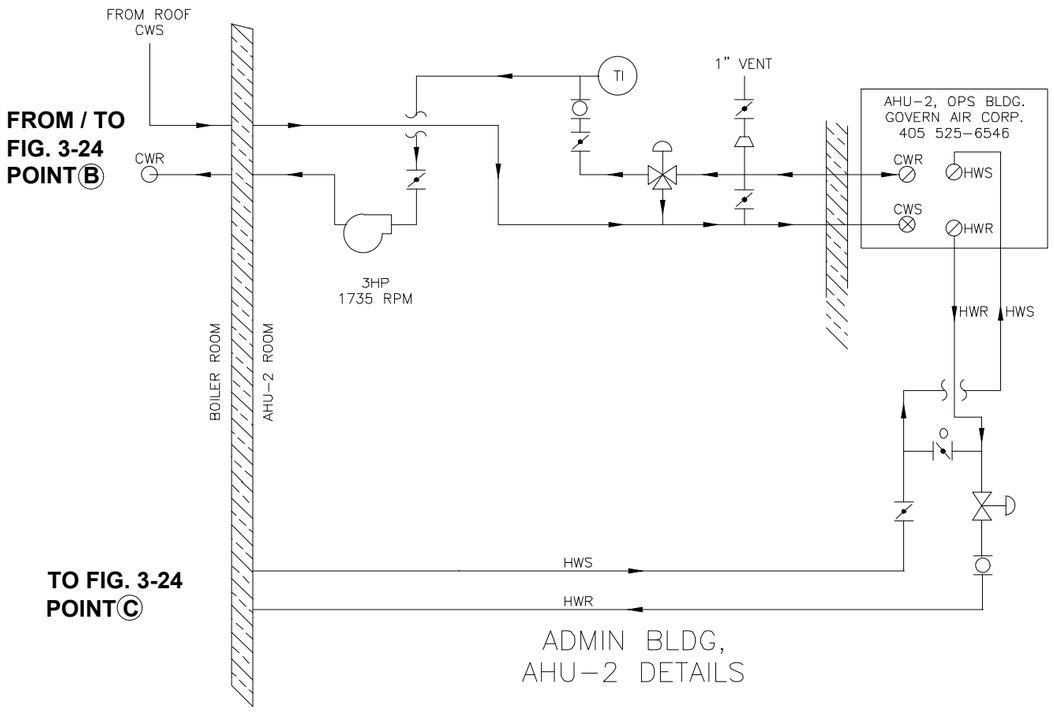
W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-28 Chilled Water.dwg 8/30/2017 10:28 AM sword



ADMIN BLDG DOMESTIC HOT WATER SYSTEM

TO FIG. 3-24 POINTⒶ

FROM / TO FIG. 3-24 POINTⒷ



ADMIN BLDG, AHU-2 DETAILS

TO FIG. 3-24 POINTⒸ

LEGEND

- ACTUATOR
- BALL VALVE
- BUTTERFLY VALVE
- CAP (SCREWED ON)
- GATE VALVE
- INCREASER / REDUCER
- INSTRUMENT (USE IS GIVEN BY LETTERS INSIDE)
- PIPE OUT OF BUILDING WALL OR FROM UNDERGROUND
- PIPE INTO BUILDING WALL OR TO UNDERGROUND
- PUMP
- STRAINER
- THREE WAY VALVE

ABBREVIATIONS:

AHU	AIR HANDLING UNIT
CU	COPPER
CWR	COLD WATER RETURN
CWS	COLD WATER SUPPLY
HWR	HOT WATER RETURN
HWS	HOT WATER SUPPLY
O/NO	OPEN / NORMALLY OPEN
PI	PRESSURE INDICATOR (PRESSURE GAUGE)
TI	TEMPERATURE INDICATOR

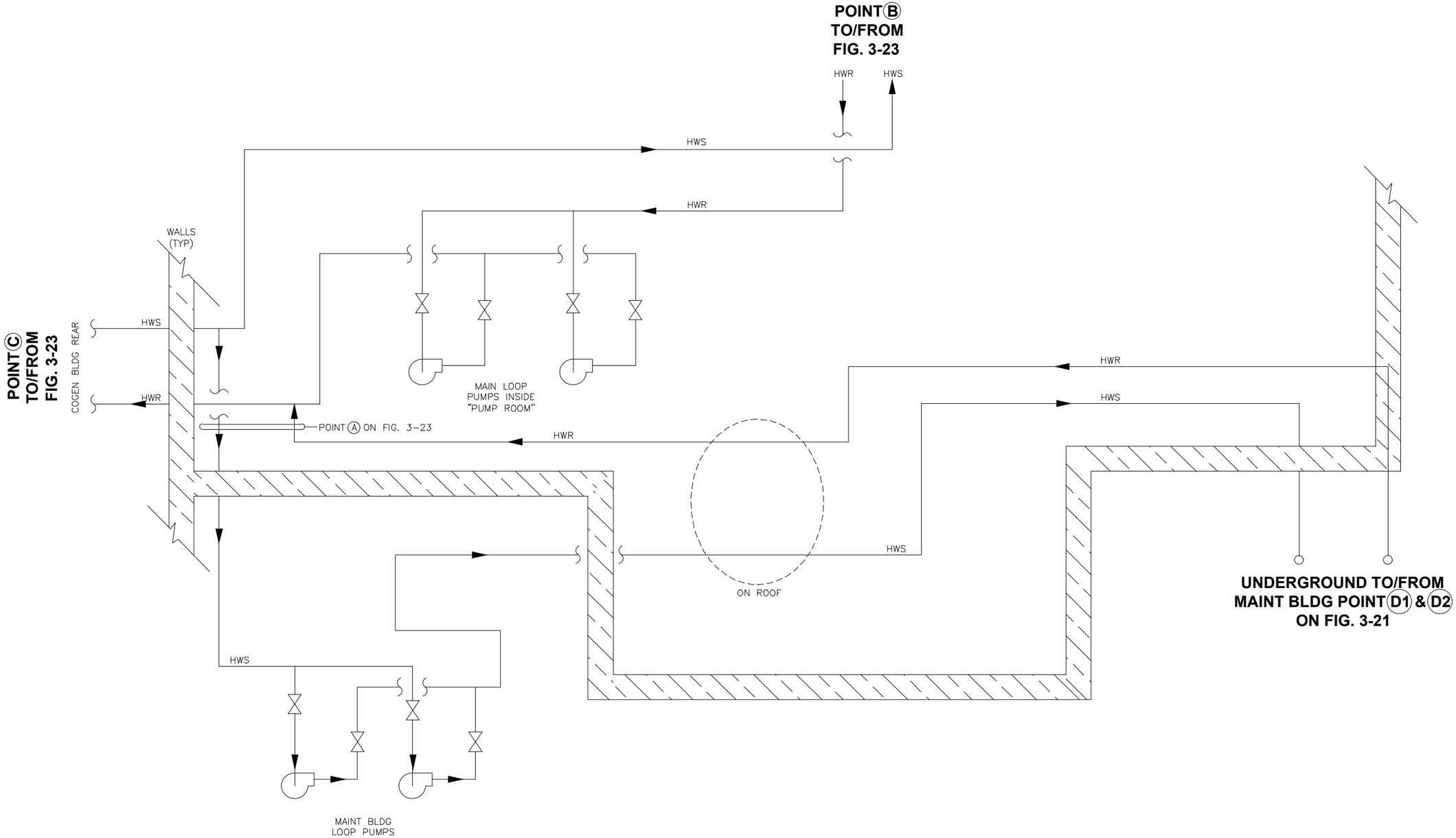
WEST YOST ASSOCIATES

Figure 3-29

Hot Water System, Miscellaneous Details

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-29 Hot Water System, Misc Details.dwg 8/30/2017 10:29 AM sword



LEGEND

GATE VALVE

PUMP

ABBREVIATIONS:

HWR	HOT WATER RETURN
HWS	HOT WATER SUPPLY

Figure 3-30

Administration Building, Details at Pump Room

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

WEST YOST ASSOCIATES

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Cogen\Fig 3-30 Main Loop & Maint. Bldg. Loop Pumps.dwg 8/30/2017 10:30 AM sword

CHAPTER 4
Regulatory Requirements

This chapter presents a review of the current and potential future regulatory requirements applicable to the District's WWTP. The following specific subjects are discussed:

- Overview of Current WWTP Permits
- Flow Requirements for Surface Water Discharge
- Water Quality Requirements for Surface Water Discharge
- Recycled Water Requirements
- Biosolids Surface Disposal Requirements
- Air Quality Requirements

A list of references is also included at the end of the chapter.

4.1 OVERVIEW OF CURRENT WWTP PERMITS

The following WWTP activities are regulated through permits:

- Discharges of secondary treated effluent to the Lower San Francisco Bay (Lower SF Bay)
- Irrigation reuse of disinfected tertiary recycled water
- Surface disposal of biosolids at the District's DLD site
- Air emissions associated with the WWTP co-generation system operations

There are a total of six water quality-related permits that regulate discharges of wastewater, recycled water, and biosolids, and there is one air quality permit that regulates air emissions associated with the WWTP co-generation system. These permits are summarized below.

4.1.1 Water Quality Permits

Pursuant to the federal Clean Water Act, the United States Environmental Protection Agency (USEPA) sets national water quality standards to protect public health and the environment. In California, the State Water Resources Control Board (State Water Board) is the agency responsible for coordinating and overseeing state and local water pollution control programs and implementing the Clean Water Act. Finally, the San Francisco Bay Regional Water Quality Control Board (Regional Board) is the agency responsible for developing and enforcing requirements related to protection of water quality for the WWTP discharges to surface water and groundwater, which includes recycled water operations and surface disposal of biosolids.

The current water quality permits applicable to the WWTP are as follows:

- Routine discharges of secondary treated effluent to the Lower SF Bay are subject to specific regulatory requirements detailed in a site-specific National Pollutant Discharge Elimination System (NPDES) permit (the "NPDES Permit"), which also permits discharge of reverse osmosis reject (i.e., brine) flow from the Alameda County Flood Control and Water Conservation District Zone 7's (Zone 7's) treated groundwater.[1]
- Routine discharges of secondary treated effluent to the Lower SF Bay are also subject to two, watershed-wide NPDES permits for specific pollutants: the "Mercury Watershed Permit" specific to mercury and polychlorinated biphenyls (PCBs) and the "Nutrients Watershed Permit" specific to nutrients (i.e., nitrogen and phosphorus compounds).
- Discharges of secondary treated effluent to surface water related to extreme wet weather events are subject to separate regulatory requirements detailed in a separate NPDES permit (the "Wet Weather Permit").
- Reuse of tertiary treated effluent (as recycled water) from the WWTP is regulated pursuant to a separate General Water Reuse Requirements Permit (the "General Order").
- Surface disposal of biosolids from the WWTP are regulated pursuant to a Waste Discharge Requirements permit specific to the DLD site (the "Biosolids Permit").

Additional details regarding these six permits are provided in Table 4-1. As indicated, the four permits related to surface water discharge have expiration dates within the next few years, and are anticipated to be renewed between 2017 and 2022. The Biosolids Permit, on the other hand, does not have a specific expiration date.

The General Order itself also does not contain an expiration date. However, the State Water Board adopted a statewide general permit for use of recycled water from Publicly Owned Treatment Works (POTWs) in June 2016 (State Water Board, 2016) that will supersede similar, regional general permits, such as the General Order. The statewide permit specifically requires that the Regional Board to terminate its regional permit within three years, or by June 2019. Requirements of both of these permits are addressed in this Chapter.

[1] The Zone 7 discharge to the WWTP is permitted as an industrial discharge under DSRSD's Pretreatment Program.

Table 4-1. Water Quality Permits Applicable to the WWTP

Permit Type	Permittees	Regional Board Permit Number	Adoption Date	Expiration Date	Permitted Discharge	DSRSD Discharge Location
Surface Water Discharge Permit (NPDES Permit)	DSRSD, LAVWMA, and EBDA	*Order No. R2-2017-0017, NPDES No. CA0037613* (Regional Board, 2017a)	5/10/17	6/30/22	Secondary treated municipal wastewater (effluent) and Zone 7 reverse osmosis reject water	Lower SF Bay
Mercury Watershed Permit	Municipal and industrial wastewater dischargers to the San Francisco Bay (including DSRSD)	*Order No. R2-2012-0096, NPDES No. CA0038849* (Regional Board, 2012)	12/12/12	12/31/17	Mercury and PCBs from DSRSD secondary effluent[a]	Lower SF Bay
Nutrients Watershed Permit	Municipal wastewater dischargers to the San Francisco Bay (including DSRSD)	*Order No. R2-2014-0014, NPDES No. CA0038873* (Regional Board, 2014)	4/9/14	6/30/19	(*Does not establish any new permitted discharges beyond individual NPDES permits*)[b]	Lower SF Bay
Wet Weather Permit	LAVWMA, DSRSD, City of Pleasanton, and City of Livermore	*Order No. R2-2016-0015, NPDES No. CA0038679* (Regional Board, 2016)	4/13/16	5/31/21	Intermittent peak wet weather extreme storm event discharge of secondary treated, disinfected, and dechlorinated effluent	San Lorenzo Creek or Alamo Canal
Recycled Water General Order (General Order)	[c]	*Order 96-011*[d] (Regional Board, 1996)	1/17/96	June 2019[e]	Municipal wastewater reuse	*Varies* (irrigated landscaping)
Biosolids Permit	DSRSD	*Order No. R2-2007-0053* (Regional Board, 2007)	8/8/07	N/A[f]	Surface disposal of biosolids to a Class II Land Treatment Unit	DSRSD's DLD site

[a] The Mercury Watershed Permit includes requirements specific to discharge of only mercury and PCBs (in wastewater effluent). This permit implements EPA-approved Total Maximum Daily Loads (TMDLs) for mercury and PCBs.

[b] The Nutrients Watershed Permit implements nutrient-related requirements (e.g. studies, monitoring, and reporting) that supersede those in individual NPDES permits for the permittees.

[c] The Recycled Water Permit is a region-wide "general permit" under which numerous publicly-owned wastewater and water agencies that recycle treated municipal wastewater can apply for coverage.

[d] The permit title is "Order 96-011." This permit is alternatively identified as "Order No. R2-1996-011 in other sources.

[e] A statewide recycled water general permit was adopted in June 2016 (State Water Board, 2016) that requires phasing out of regional general permits for recycled water from POTWs within three years (i.e. by June 2019).

[f] The Biosolids Permit does not have an expiration date.

4.1.2 Air Quality Permit

Pursuant to the federal Clean Air Act (CAA), the USEPA sets national air quality standards to protect public health and the environment. In California, the California Air Resources Board (CARB) is the agency responsible for coordinating and overseeing state and local air pollution control programs and implementing the CAA. In addition, the Bay Area Air Quality Management District (BAAQMD) is responsible for developing and enforcing regulation of air emission sources and odor control in the seven counties in the San Francisco Bay Area, including Alameda County. BAAQMD currently regulates air discharges from the WWTP through permit A1371 (Air Permit), which expires on July 1, 2018 (BAAQMD, 2017).

4.2 FLOW REQUIREMENTS FOR SURFACE WATER DISCHARGE

This section includes a summary of the District's flow limitations with respect to surface water discharge of treated wastewater. The following specific topics are addressed:

- Overview of Effluent Storage and Conveyance to Surface Water
- LAVWMA Flow Limits and Operational Requirements
- EBDA Flow Limits and Operational Requirements
- Current NPDES Effluent Flow Limitations
- Proposed NPDES Effluent Flow Limitations
- Summary of Current Flow Limitations
- Potential Future Flow Requirements

4.2.1 Overview of Effluent Storage and Conveyance to Surface Water

Disinfected secondary effluent from the WWTP is conveyed to a LAVWMA junction box, where it is combined with disinfected secondary effluent from the City of Livermore's (Livermore's) Water Reclamation Plant. In addition, Zone 7 has reverse osmosis (RO) facilities that can treat approximately 15 mgd of brackish groundwater, and a maximum of 3.2 mgd of reject water (brine) can be conveyed to the WWTP for discharge to the LAVWMA junction box. The Zone 7 brine enters the WWTP outfall pipeline upstream of the final effluent monitoring station at LAVWMA, but downstream of the WWTP treatment facilities and the diversion point for the District's recycled water treatment facilities.

The flows from the WWTP, Livermore's Water Reclamation Plant, and Zone 7 are pumped via the LAVWMA Export Pumping Station over the Dublin Grade (Interstate 580) through the LAVWMA Export Pipeline to conveyance and outfall facilities owned and operated by EBDA. The LAVWMA flows, along with treated effluent from the EBDA member agencies, are conveyed via the EBDA facilities to a dechlorination station located near the San Leandro Marina, and then to the deepwater EBDA Common Outfall to Lower SF Bay.

Under wet weather conditions, WWTP effluent flows may be dechlorinated in LAVWMA facilities and discharged to Alamo Canal or San Lorenzo Creek via separate outfalls. A manually operated gate at the LAVWMA junction structure controls overflows to the Alamo Canal. These flows are dechlorinated at the LAVWMA junction structure prior to discharge. Discharges to the San Lorenzo Creek, which are intended to be used in case of an interruption in EBDA's service, would occur downstream of the LAVWMA storage reservoirs and Export Pumping Station. Prior to discharge, these flows are dechlorinated at a dechlorination station located near the point of discharge.

The District, Livermore, and LAVWMA have flow equalization/storage basins that can be used to manage peak flows. The total storage capacity of the flow equalization storage basins is 54.7 MGal, comprising the following:

- 20.7 MGal in the District's four holding basins[2]
- 18.0 MGal in LAVWMA's three storage reservoirs
- 16.0 MGal in Livermore's emergency holding basin and peaking pond

During summer dry periods, the LAVWMA storage reservoirs are typically used for diurnal storage, to take advantage of off-peak electricity rates for the LAVWMA Export Pumping Station. During winter months, the basins continue to be used for diurnal storage and off-peak pumping until the LAVWMA Operator asserts that a wet weather event has begun. This occurrence is declared once at least a half inch of precipitation is measured within a 24-hour period in the District's and Livermore's service areas, and the combined flows from the District and Livermore exceed normal diurnal variations (LAVWMA, 2014).

4.2.2 LAVWMA Flow Limits and Operational Requirements

The LAVWMA member agencies (DSRSD and the Cities of Livermore and Pleasanton) currently operate under a joint exercise of powers agreement (LAVWMA Agreement) (LAVWMA, 1997) that was approved by each agency in September 1997. Much of the LAVWMA Agreement is focused on the LAVWMA structure and cost sharing and procedural conditions that are not directly applicable to the Master Plan. Discussed below are the flow limits and operational requirements from the LAVWMA Agreement that are relevant to the current master planning process.

[2] This value is taken from the 2014 LAVWMA Annual Technical Report (LAVWMA, 2014), which is consistent with the information presented in Chapter 3 of this Master Plan. However, the NPDES Permit indicates that these holding basins have a capacity of 22 Mgal with 2 feet of freeboard. As discussed in Chapter 3, the holding capacity of the District's four equalization basins is approximately 18 Mgal when operating with a two-foot freeboard.

Section 13.1 of the LAVWMA Agreement establishes a total influent ADWF limit of 31.8 mgd allocated to the member agencies as follows:

- 10.4 for DSRSD
- 10.3 mgd for Pleasanton
- 11.1 mgd for Livermore

The above flow limitations apply to "raw wastewater influent to all existing and future wastewater treatment facilities providing wastewater treatment for the Service Area…" "ADWF" is defined in the LAVWMA Agreement as "the average daily influent flow over the three consecutive months of lowest flows," and "influent" is defined as "the amount of wastewater flowing into all existing and future facilities providing wastewater treatment for the LAVWMA Service Area, exclusive of [internal] recycled flows associated with a treatment plant."

The LAVWMA Agreement does not allow LAVWMA member agencies to issue new connections once the agency's influent limit has been reached. However, member agencies are allowed to transfer a portion of their influent allocation to another member agency.

The Service Area referenced above is defined in terms of service to incorporated areas or to unincorporated areas. "Service to incorporated areas" is defined in Section 14.1 as follows:

> *Livermore, Pleasanton and DSRSD may provide wastewater and disposal service, including collection, conveyance and treatment of raw wastewater, from only those areas which now or in the future are incorporated within the city limits of Livermore, Pleasanton, and Dublin respectively. DSRSD may provide such service to the area within the city limits of San Ramon that DSRSD serves as of the date of this Agreement and any area within the specific plan for Westside San Ramon dated November 1989 that is subsequently incorporated into the City of San Ramon, and Livermore may continue to provide such service to the Ruby Hill sub-division in the City of Pleasanton.*

"Service to unincorporated areas" is defined in Section 14.2 of the LAVWMA Agreement. With regards to the District, this definition limits wastewater treatment and disposal service to unincorporated areas that were served at the time of adopting the LAVWMA Agreement, as well as the federal Camp Parks facility in Dublin. Unanimous approval of the LAVWMA Board is required to serve any other unincorporated area. Such approval is also required for a proposed service that would exceed ten unincorporated residential connections (or its equivalent) in one contiguous area.

At the time of the LAVWMA Agreement's adoption, the Peak Wet Weather Flow (PWWF) capacity of LAVWMA's export and storage facilities was defined as 21 mgd[3]. However, the LAVWMA Agreement also describes an expansion of the LAVWMA pumping and pipeline facilities (the "Project") to increase the total PWWF design capacity of the LAVWMA system to 41.2 mgd. This expansion project has since been completed, and thus the current PWWF capacity is 41.2 mgd.

Section 15.2 of the LAVWMA Agreement specifies that this PWWF capacity would be allocated to the member agencies as follows:

- 14.4 mgd for DSRSD
- 14.4 mgd for Pleasanton
- 12.4 mgd for Livermore

These PWWF capacity and allocation limits do not affect the influent ADWF limits defined in the LAVWMA Agreement.

The LAVWMA Agreement includes two additional requirements relevant for master planning purposes. These requirements, which both require coordination and cooperation among the LAVWMA member agencies are as follows:

- Paragraph 17 requires the LAVWMA member agencies to "operate all treatment and disposal facilities in the LAVWMA Service Area, including storage ponds and reverse osmosis facilities, in a coordinated manner to minimize the risk of a LAVWMA overflow."
- Paragraph 18 requires the LAVWMA member agencies to coordinate the implementation of any additional storage and/or discharge facilities needed to provide additional PWWF capacity when any member agency's facility reaches 75 percent of its allocated PWWF capacity.

[3] PWWF defined in the LAVWMA agreements is assumed to the same as the PHWWF defined in this Master Plan.

Specific operational requirements used by LAVWMA to meet EBDA flow limits are described in an annual Wet Weather Facilities Operations Plan (WWOP) prepared by LAVWMA. LAVWMA annually evaluates its system facilities operation and prepares an updated WWOP to describe practices for managing influent and effluent flows and storage during wet weather events. The WWOP provides a strategy for coordinating the various wet weather equalization/storage facilities for the LAVWMA agencies and a training guide for operators of these facilities. The last major revision to the WWOP was completed in May 2009 (LAVWMA, 2014). The most recent WWOP available is for 2012 (LAVWMA, 2012), which describes the LAVWMA operational strategy once a wet weather event has begun[4] as follows:

- Storage levels in the LAVWMA holding basins are brought down to minimal levels.
- The Export Pumping Station is placed in "storm mode," allowing the LAVWMA Operator to adjust the effluent flow rate to match the influent flow rate, up to the 41.2 mgd capacity of the pump station.
- EBDA is notified that LAVWMA has switched to the storm mode of control.
- Zone 7 is notified that a stoppage of brine discharge may be necessary and would be preceded by a 24-hour notice.

4.2.3 EBDA Flow Limits and Operational Requirements

The current master agreement between EBDA and LAVWMA governing LAVWMA's discharge to the EBDA system was signed April 26, 2007. This master agreement requires EBDA to provide LAVWMA with firm capacity discharge rights of 19.72 mgd and an additional 21.48 mgd in interruptible discharge rights. Thus, LAVWMA flows discharged to the EBDA Common Outfall are currently limited to a total of 41.2 mgd. This EBDA flow limit is consistent with the current PWWF capacity of the LAVWMA facilities, as described above.

Firm capacity rights are always available for use by LAVWMA, and are allocated to the LAVWMA member agencies per the LAVWMA Agreement as follows:

- 4.385 mgd for DSRSD
- 7.106 mgd for Pleasanton
- 8.229 mgd for Livermore

[4] As noted previously, a wet weather event is declared once at least a half inch of precipitation is measured within a 24-hour period in the District's and Livermore's service areas, and the combined flows from the District and Livermore exceed normal diurnal variations

Interruptible capacity is clarified in the LAVWMA Agreement as being "interrupted when the combined EBDA member agency instantaneous flow reaches 169.4 mgd." This interruptible flow is allocated to the LAVWMA member agencies as follows:

- 10.014 mgd for DSRSD
- 7.294 mgd for Pleasanton
- 4.172 mgd for Livermore

Thus, the LAVWMA Agreement provides the District with firm capacity discharge rights of approximately 11.5 mgd and interruptible discharge rights of 17.3 mgd, for a total potential PWWF discharge capacity to the EBDA Common Outfall of 28.8 mgd. The flows for the District can include Zone 7 brine flow, if capacity is available. However, the Zone 7 brine flow is considered by LAVWMA and the District to be "interruptible," allowing for flexibility in meeting the EBDA flow limits.

4.2.4 Current NPDES Effluent Flow Limitations

The NPDES Permit includes the following specific flow limitations, which are equal to design flows listed in the Stage 4 Drawings for the WWTP's facilities:

- 17.0 mgd for ADWF,[5] plus 3.2 mgd for Zone 7 brine flow, for a total ADWF of 20.2 mgd
- 60.7 mgd for wet weather flow

As noted above, LAVWMA's agreement with EBDA provides LAVWMA with firm capacity discharge rights of 19.72 mgd and an additional 21.48 mgd in interruptible discharge rights. To account for the fact that discharges to EBDA can be restricted, the Wet Weather Permit allows LAVWMA to discharge up to 21.5 mgd of its peak wet weather flow to San Lorenzo Creek. Discharge Prohibition III.B of the Wet Weather Permit specifically prohibits discharge to San Lorenzo Creek "except during peak wet weather flows [and] after [LAVWMA] fully utilizes the maximum allowed hydraulic capacity available in the EBDA pipeline …" An exception is also made under certain conditions when the discharge flap gates located in the San Lorenzo Creek outfall are exercised. Exercise events are limited to two a year and a maximum discharge of 140,000 gallons per event, must be during the wet season (November to April), and must be when there is a significant flow in San Lorenzo Creek (i.e. during a storm event).

[5] "ADWF" is defined in the NPDES Permit as being the average flow over three consecutive dry weather months each year (May 1 to October 31).

Similarly, the LAVWMA Agreement provides the District with discharge rights up to 28.8 mgd, which is significantly lower than the PHWWF capacity of the WWTP. To account for the difference between the allowed discharge capacity and the potential PHWWF from the WWTP, the Wet Weather Permit allows for LAVWMA to discharge up to 55 mgd to Alamo Canal. Specifically, Discharge Prohibition III.C of the Wet Weather Permit prohibits discharge to Alamo Canal, except during peak wet weather conditions and after LAVWMA has fully utilized its maximum export pumping capacity of 41.2 mgd to the EBDA pipeline (or to the EBDA pipeline and San Lorenzo Creek outfall). These discharges to Alamo Canal must also be conducted in accordance with Provision VI.C.5.b of the permit, which requires operation in accordance with a Wet Weather Facilities Management Plan that describes strategies to maximize storage and export capacity and minimize discharges during wet weather events.

4.2.5 Proposed NPDES Effluent Flow Limitations

In preparing for the adoption of the previous (2012) version of the NPDES Permit, the District had submitted to the Regional Board an Antidegradation Study to allow for an increase in ADWF to 20.7 mgd. As described in the NPDES Permit, the Antidegradation Study documents that an increase in effluent flow to 20.7 mgd, plus the 3.2 mgd Zone 7 brine flow, would conform to federal and State antidegradation requirements. The current (2017) NPDES Permit conditionally allows the effluent flow limitations to be increased to the following flow limitations for the "proposed" conditions:

- 20.7 mgd for the proposed ADWF, plus 3.2 mgd for Zone 7 brine flow, for a total ADWF of 23.9 mgd
- 74.0 mgd for the proposed wet weather flow

Prior to discharging at these higher flows, the District must complete proposed upgrades and the following tasks (i.e., conditions) identified in Provision VI.C.5.c of the NPDES Permit, as well as receive the Regional Board's Executive Officer approval of successful completion of these tasks:

i. Certification by a licensed professional affirming that the upgrades have been constructed as designed and that the reliability, capability, and performance of the wastewater facility upgrades will maintain compliance with waste discharge requirements. Hydraulic and organic loading capacities of the treatment facilities shall be evaluated by appropriate combinations of desk-top analyses and treatment process stress testing to simulate design peak loading conditions. The evaluation shall include treatment process operations under both dry weather and wet weather design flow conditions and effluent disposal capacity including storage and discharge to land through reclamation.

ii. Compliance with all applicable provisions of the California Environmental Quality Act (California Public Resources Code Division 13, Chapter 3, Section 21100 et seq.).

iii. *Updates to the Operations and Maintenance Manual and Contingency Plan to include the facility upgrades, including financial provisions; and*

iv. *The specific date the* [District] *proposes to commence its increased average dry weather flow.*

4.2.6 Summary of Current Flow Limitations

The sections above describe various flow limits that apply to discharge from the WWTP. These permitted effluent flow limits are summarized in Table 4-2 for the total WWTP discharge, as well as for the District and Pleasanton's individual allocations of these discharges.

Table 4-2. Summary of Permitted Flow Limits for the WWTP

Flow Limit Description	Effluent Flow Limit by Service Area, mgd		
	DSRSD	Pleasanton	Combined WWTP Effluent
LAVWMA Agreement with EBDA			
Effluent PWWF Allocation	14.4	14.4	28.8
Allocation of EBDA Firm Flow Capacity	4.385	7.106	11.491
Allocation of EBDA Interruptible Flow Capacity	10.014	7.294	17.308
LAVWMA Agreement with DSRSD			
Average Influent Flow Limit	10.4	10.3	20.7
PWWF Flow Limit	14.4	14.4	28.8
NPDES Permit			
Existing ADWF Limit	(a)	(a)	17.0(b)
Existing Wet Weather Flow Limit	(a)	(a)	60.7
Proposed ADWF Limit	(a)	(a)	20.7(b)
Proposed Wet Weather Flow Limit	(a)	(a)	74.0
Wet Weather Permit			
Flow Limit for Discharge to San Lorenzo Creek(c)	(a)	(a)	21.5
Flow Limit for Discharge to Alamo Canal(d)	(a)	(a)	55

(a) Flow limits in the NPDES Permit are established only for total effluent flow.
(b) Plus up to 3.2 mgd of Zone 7 brine flow.
(c) Discharge to San Lorenzo Creek is also contingent upon peak wet weather conditions and full use of the maximum allowed hydraulic capacity available in the EBDA system.
(d) Discharge to Alamo Canal is also contingent upon peak wet weather conditions and use of the LAVWMA maximum export pumping capacity to the EBDA system.

4.2.7 Potential Future Flow Restrictions for Surface Water Discharge

Future restrictions on WWTP effluent flows to surface water in addition to those detailed above are not anticipated at this time.

4.3 WATER QUALITY REQUIREMENTS FOR SURFACE WATER DISCHARGE

This section includes discussion of the current and potential future water quality-related regulatory requirements for discharges of treated wastewater to the Lower SF Bay from the WWTP.

4.3.1 Current Surface Water Discharge Requirements

The District's NPDES and Wet Weather Permits both include surface water discharge requirements relevant to this Master Plan, such as effluent and receiving water limitations. These limits are based on the beneficial uses and Water Quality Objectives (WQOs) defined in the Basin Plan, as well as dilution credits granted for the District's receiving waters. This section thus details the following:

- Receiving Water Beneficial Uses
- Applicable WQOs
- Dilution Credits
- Effluent Water Quality Limitations
- Receiving Water Limitations

4.3.1.1 Receiving Water Beneficial Uses

Existing and potential beneficial uses of the receiving waters are defined in the Regional Board's *San Francisco Bay Basin (Region 2) Water Quality Control Plan* (Basin Plan) (Regional Board, 2017b). These beneficial uses affect what WQOs are applicable to discharges to the receiving waters within the Regional Board's jurisdiction. In other words, WQOs are the driver for effluent and receiving water limits in Regional Board permits. Reviewing the beneficial uses of the District's receiving waters is, therefore, helpful for understanding WQOs and existing and potential future surface water discharge requirements. Beneficial uses for the District's receiving waters are presented in Table 4-3.

The beneficial uses presented in Table 4-3 are based on the current Basin Plan and agree with the beneficial uses listed in the NPDES Permit and Wet Weather Permit, except as noted. The Alamo Canal each has two existing beneficial uses listed in the Basin Plan that are not listed in the Wet Weather Permit: Fish Migration and Fish Spawning. The listing of the Alamo Canal with these beneficial uses is not new with recent updates to the Basin Plan, and the previous (2011) version of the Wet Weather Permit included these beneficial uses. Therefore, the reason for these not being listed in the current Wet Weather Permit is unclear. In any case, these additional beneficial uses are not correlated with specific WQOs in the Basin Plan and would thus not necessarily lead to the introduction of additional WQOs in the Wet Weather Permit.

Table 4-3. Beneficial Uses for DSRSD's Receiving Waters

Basin Plan Beneficial Use	Basin Plan Abbreviation	Receiving Water		
		Lower SF Bay	Alamo Canal	San Lorenzo Creek
Industrial Service Supply	IND	Yes	No	No
Commercial and Sport Fishing	COMM	Yes	No	No
Shellfish Harvesting	SHELL	Yes	No	No
Estuarine Habitat	EST	Yes	No	No
Fish Migration	MIGR	Yes	Yes[a]	Yes
Preservation of Rare and Endangered Species	RARE	Yes	No	No
Fish Spawning	SPWN	Yes	Yes[a]	Yes
Wildlife Habitat	WILD	Yes	Yes	Yes
Water Contact Recreation	REC-1	Yes	Yes	Yes
Non-Contact Water Recreation	REC-2	Yes	Yes	Yes
Navigation	NAV	Yes	No	No
Groundwater Recharge	GWR	No	Yes	No
Cold Freshwater Habitat	COLD	No	Potential[b]	Yes
Cold Freshwater Habitat	WARM	No	Yes	Yes
Municipal and Domestic Supply	MUN	No	No	Yes
Freshwater Replenishment	FRSH	No	No	Yes

[a] The Wet Weather Permit does not list Alamo Canal as having a MIGR or SPWN beneficial use, but the Basin Plan does.

[b] Beneficial uses are listed in the Basin Plan as either "Existing" or "Potential." For NPDES permitting purposes, the Regional Board applies WQOs for both existing and potential beneficial uses.

4.3.1.2 Applicable WQOs

The Basin Plan establishes WQOs for the San Francisco Bay Area receiving waters, and these WQOs include the following:

- Narrative WQOs for chronic toxicity and bioaccumulation
- Numeric WQOs for acute toxicity and 10 priority pollutants, including arsenic, cadmium, chromium VI, copper, cyanide, lead, mercury, nickel, silver, and zinc
- Numeric WQOs for un-ionized ammonia, including an annual median value (0.025 mg/L) and a maximum value for the Lower SF Bay (0.4 mg/L)

In addition, WQOs for the 126 priority pollutants listed in the California Toxics Rule (USEPA, 2000) are applicable for all inland surface water bodies and enclosed bays and estuaries in California.

The Basin Plan and California Toxics Rule include WQOs for both freshwater and saltwater bodies. Freshwater WQOs apply to discharges to water bodies with salinities equal to or less than 1 part per trillion (ppt) at least 95 percent of the time. Saltwater WQOs apply to discharges to water bodies with salinities equal to or greater than 10 ppt at least 95 percent of the time in a normal water year.

The NPDES Permit identifies the Lower SF Bay as having salinities in the range to be classified as saltwater and thus relies on saltwater WQOs for discharges to the Lower SF Bay. The Wet Weather Permit indicates that Alamo Canal and San Lorenzo Creek are freshwater under wet weather conditions and thus relies on freshwater WQOs. The reasoning presented is that San Lorenzo Creek receives a "large amount of stormwater flow upstream" and "Alamo Canal receives upstream freshwater flows."

4.3.1.3 Dilution Credits

The Policy for Implementation of Toxics Standards for Inland Surface Water, Enclosed Bays, and Estuaries of California (State Implementation Plan or SIP) allows dilution credits for completely-mixed discharges and, under certain circumstances, for incompletely-mixed discharges. These dilution credits define an amount of dilution granted in calculating water-quality based effluent limits from the WQOs discussed above. Discussed below are specific dilution credits that the Regional Board has documented in the District's permits for discharges to the Lower SF Bay and the wet-weather discharge locations (Alamo Canal and San Lorenzo Creek).

4.3.1.3.1 Dilution Credits for Lower SF Bay

Effluent flows to the Lower SF Bay receive considerable dilution, as noted in the NPDES Permit. Tracer simulation results summarized an EBDA's recent Antidegradation Analysis (EBDA, 2006) indicate a current minimum hourly average dilution of 95-to-1 (effluent to receiving water ratio).

Although a relatively high dilution has been documented, for non-bioaccumulative pollutants (except ammonia), the Regional Board assigned a conservative dilution allowance of 10:1 in the NPDES Permit. The 10:1 dilution allowance is based, in part, on Basin Plan Prohibition 1, which prohibits discharges with less than 10:1 dilution. Section 1.4.2 of the SIP allows for limiting dilution credits, and the Regional Board has previously determined that limiting the dilution allowance to 10:1 is justified. For ammonia, which is considered a non-persistent, non-bioaccumulative pollutant, a dilution allowance of 79:1 is currently allowed by the NPDES Permit.

4.3.1.3.2 Dilution Credits for Alamo Canal/San Lorenzo Creek

Dilution credits granted in the Wet Weather Permit for Alamo Canal and San Lorenzo Creek are based on a mixing zone analysis (LAVWMA, 2010) provided in the 2010 permit renewal application for the Wet Weather Permit. The mixing zone analysis justifies a dilution credit for San Lorenzo Creek of 11:1 and for Alamo Canal of 15:1.

4.3.1.4 Effluent Water Quality Limitations

The NPDES Permit and Wet Weather Permit include effluent water quality limitations (effluent limits) for discharge of secondary treated effluent and Zone 7 brine to the District's three permitted surface water discharge locations: the Lower SF Bay, Alamo Canal, and San Lorenzo Creek. The Mercury Watershed Permit also includes effluent limits for mercury and PCBs for the EBDA discharge to the Lower SF Bay. These effluent limits are detailed below.

As indicated in Table 4-1, the current Mercury Watershed Permit expires at the end of 2017. The Regional Board is in the process of renewing this permit, and a draft version of the renewed permit was recently issued for adoption by the Regional Board in early November 2017. If the permit is adopted as drafted, the mercury and PCBs effluent limits for the EBDA discharge, which are detailed below, would be unchanged.

4.3.1.4.1 Effluent Limits for Discharge to Lower SF Bay

The NPDES Permit includes effluent limitations for several parameters. These include both "conventional pollutants," such as 5-day Carbonaceous Biochemical Oxygen Demand (CBOD) and TSS, as well as "priority pollutants," as established through the California Toxics Rule (USEPA, 2000); the State Water Board's *Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California* (State Water Board, 2005) – referred to as the "SIP"; and the Basin Plan.

Some of the NPDES Permit's effluent limits apply to the District's discharge to the LAVWMA system (at either monitoring location "M-002F1" or "M-002F2"), and some apply to the discharge from the EBDA Common Outfall to the Lower SF Bay (monitoring location "EFF-001"). Descriptions of these monitoring locations from the NPDES Permit are provided in Table 4-4.

Table 4-4. Summary of DSRSD Effluent Monitoring Locations

Name	Description
EFF-001	Point in the EBDA Common Outfall between the point of discharge and the point at which all waste tributary to that outfall is present.
M-002F1	Point in the WWTP at which adequate disinfection has taken place and prior to the addition of Zone 7 brine.
M-002F2	Point in the WWTP at which adequate disinfection has taken place just prior to where DSRSD transfers control of its effluent to the LAVWMA facilities.

Effluent limits for standard averaging periods (e.g., average monthly, average weekly, maximum daily) are summarized in Table 4-5. Additional effluent limits prescribed for non-standard averaging periods are summarized in Table 4-6. Both tables indicate the applicable discharge location for the effluent limits. Effluent limits for mercury and PCBs from the Mercury Watershed Permit are also included, as applicable, and apply to discharge from the EBDA Common Outfall. Compliance with mercury and PCBs effluent limits is verified through routine monitoring conducted by EBDA staff.

Table 4-5. Effluent Limits for Discharge to Lower SF Bay

Parameter	Units	Effluent Limit[a]				
		Average Monthly	Average Weekly	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum
Applicable at Monitoring Location M-002F1						
CBOD	mg/L	25	40	--	--	--
TSS	mg/L	30	45	--	--	--
Applicable at Monitoring Location M-002F2						
pH	s.u.	--	--	--	6.0	9.0
Applicable at EBDA Common Outfall (Monitoring Location EFF-001)						
Oil and Grease	mg/L	10	--	20	--	--
Chlorine, Total Residual	mg/L	--	--	--	--	0.0
Copper, Total	µg/L[d]	53	--	69	--	--
Cyanide, Total	µg/L[d]	21	--	40	--	--
Dioxin-TEQ[b]	µg/L[d]	1.4×10^{-8}	--	2.8×10^{-8}	--	--
Total Ammonia, as N	mg/L	91	--	120	--	--
Mercury[c]	µg/L[d]	0.066	0.072	--	--	--
PCBs[c]	µg/L[d]	0.012	--	0.017	--	--

[a] Cells listed with "--" indicate no effluent limit for the parameter at the specified averaging period.
[b] Dioxin (2,3,7,8-tetrachlorinated dibenzo-p-dioxin) and dioxin-like compounds are regulated through the use of toxicity equivalencies (TEQs). "Dioxin-TEQ" refers to a sum of concentrations for dioxin and dioxin-like compounds.
[c] Mercury and PCBs effluent limits are from the Mercury Watershed Permit. All other limits are from the NPDES Permit.
[d] Micrograms per liter.

Table 4-6. Additional Effluent Limits for Discharge to Lower SF Bay

Parameter	Limit Description	Limit Value	Units
Applicable at Monitoring Location M-002F1			
CBOD	Minimum Average Monthly Removal	85	percent
TSS	Minimum Average Monthly Removal	85	percent
Applicable at EBDA Common Outfall (Monitoring Location EFF-001)			
Fecal Coliform Bacteria	Maximum geometric mean of all fecal coliform results in a calendar month	500	MPN/100 mL[a]
	Maximum 90th percentile value for any eleven consecutive samples	1,100	MPN/100 mL
Enterococci Bacteria	Maximum geometric mean of all enterococci results in a calendar month	240	MPN/100 mL
Acute Whole Effluent Toxicity	Minimum 90th percentile value for any eleven consecutive bioassays	70	percent survival
	Minimum median for any eleven consecutive bioassays	90	percent survival
Mercury[b]	Average Annual Load	2.2	kilograms/year

[a] MPN/100 mL = Most Probable Number per 100 milliliters.
[b] The mercury effluent limit is from the Mercury Watershed Permit. (All other effluent limits are from the NPDES Permit.)

4.3.1.4.2 Effluent Limits for Wet Weather Discharge to Alamo Canal/San Lorenzo Creek

The Wet Weather Permit includes effluent limits for several parameters for discharge to either Alamo Canal or San Lorenzo Creek. These effluent limits are listed in Table 4-7.

Table 4-7. Effluent Limits for Discharge to Alamo Canal or San Lorenzo Creek

Parameter	Units	Effluent Limit[a]				
		Average Monthly	Average Weekly[b]	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum
CBOD	mg/L	--	40	--	--	--
TSS	mg/L	--	45	--	--	--
Oil and Grease	mg/L	--	--	20	--	--
pH	s.u.	--	--	--	6.5	8.5
Total Residual Chlorine	mg/L	--	--	--	--	0.0
Fecal Coliform	MPN/100 mL	--	--	400	--	--
Lead, Total	µg/L	5.5	--	11	--	--
Acute Whole Effluent Toxicity	percent survival	A 96-hour bioassay of the effluent cannot have less than 70 percent survival of test organisms				

[a] Cells listed with "--" indicate no effluent limit for the parameter at the specified averaging period.
[b] Average Weekly Effluent Limits apply only if a discharge lasts more than 24 hours in a calendar week.

Compliance with the wet weather effluent limits is verified through monitoring conducted by LAVWMA staff pursuant to the Wet Weather Permit's Monitoring and Reporting Program (MRP) (Attachment E of the Wet Weather Permit). If a discharge occurs to either Alamo Canal or San Lorenzo Creek, then the combined LAVWMA effluent must be sampled at the discharge location once per discharge event. Required monitoring parameters include duration of discharge, flow volume, and temperature. In addition, concentrations of the parameters with effluent limits (as shown in Table 4-7), Total Dissolved Solids, and chlorides must be monitored once per discharge event. Total Residual Chlorine must also be monitored once every two hours.

A subset of this monitoring must be also conducted for discharge events related to exercising the discharge flap gates on the San Lorenzo Creek outfall. Discharge duration and volume, fecal coliform, and Total Residual Chlorine must be monitored at least once per discharge event. Total Dissolved Solids and chlorides must be monitored at least once per permit term.

4.3.1.5 Receiving Water Limitations

The NPDES Permit and Wet Weather Permit include receiving water limitations for discharge to the District's three permitted surface water discharge locations: the Lower SF Bay, Alamo Canal, and San Lorenzo Creek. These receiving water limits are detailed below.

4.3.1.5.1 Receiving Water Limits for Discharge to Lower SF Bay

The NPDES Permit includes a Receiving Water Limitations section with various requirements related to the water quality and/or conditions of the Lower SF Bay. These Receiving Water Limitations are based on WQOs in the Basin Plan and require that effluent from the WWTP not cause exceedances of the WQOs.

The WQOs include narrative requirements, such as limiting biostimulatory substances (identified as "nutrients" in the NPDES Permit) that promote aquatic growth, limiting floating material and coloration to levels that do not cause nuisance conditions, and limiting dissolved sulfide to "natural background levels." Although the NPDES Permit does not require receiving water inspections to assess compliance with these narrative objectives, a well-operated wastewater treatment facility meeting the effluent limitations discussed above should be capable of achieving these objectives. Potential future requirements related to nutrients are addressed later in this chapter. For these reasons, additional discussion of these objectives is not warranted in the current section.

The WQOs also include numeric objectives for several parameters, such as dissolved oxygen, pH, and un-ionized ammonia (as noted above). The NPDES Permit includes numeric receiving water limitations only for dissolved oxygen, as detailed in Table 4-8.

Table 4-8. Numeric Receiving Water Limitations for Discharge to Lower SF Bay

Parameter	Limit Description
Dissolved Oxygen	5.0 mg/L, minimum
	The median dissolved oxygen concentration for any three consecutive months shall not be less than 80 percent of the dissolved oxygen content at saturation. When natural factors cause concentrations less than those specified above, the discharge shall not cause further reduction in ambient dissolved oxygen concentrations.

Compliance with the receiving water limits listed in Table 4-8 is verified through monitoring conducted as part of the San Francisco Estuary Institute's Regional Monitoring Program (RMP). Pursuant to Section VI of the NPDES Permit's MRP (Attachment E of the NPDES Permit), the District participates in and provides funding to the RMP to collect receiving water quality data for the Lower SF Bay.

As noted previously, considerable dilution of the combined EBDA discharge occurs. Thus, the potential for the discharge to cause dissolved oxygen excursions below the Receiving Water Limitations is very low or non-existent.

Similar to receiving water limits, the NPDES Permit also specifies "action levels" for copper and cyanide, based on Basin Plan site-specific objectives for these constituents. These action levels are as follows:

- Three-year rolling mean dissolved copper concentrations of the Lower SF Bay exceed 2.2 µg/L
- Cyanide concentrations of 1.0 µg/L or higher in the main body of the San Francisco Bay

If the Regional Board notifies the District that these action levels are exceeded, then the District (along with other dischargers to the receiving waters) is required to implement specific steps, such as evaluating the trend of effluent concentrations, identifying actions needed to control these constituents, and reporting on the progress of existing control measures and plans for additional control measures that may be needed. To date, the Regional Board has not notified the District that either of these action levels has been exceeded.

4.3.1.5.2 Receiving Water Limits for Wet Weather Discharge to Alamo Canal/San Lorenzo Creek

The Wet Weather Permit includes Receiving Water Limitations for Alamo Canal and San Lorenzo Creek that are identical to the Receiving Water Limitations for the Lower SF Bay that are included in the NPDES Permit. Compliance with the receiving water limits for Alamo Canal and San Lorenzo Creek is generally verified through monitoring conducted by LAVWMA staff. Monitoring for some parameters with narrative receiving water limits, such as dissolved sulfide, is not required.

The required receiving water monitoring includes collecting grab samples, at a minimum, twice during the permit term and analyzing for certain conventional pollutants (pH, temperature, salinity, hardness, ammonia, and dissolved oxygen). A grab sample for Total Dissolved Solids and chlorides must also be collected once per permit term. In addition, "standard observations" of various nuisance factors (e.g., floating materials, odors) are required once during each discharge event to confirm compliance with related receiving water limits.

Considerable dilution of the discharge occurs, as discussed above. Thus, the potential for the discharge to cause an excursion in violation of the Receiving Water Limitations is very low or non-existent.

4.3.2 Potential Future Surface Water Discharge Requirements

The District's surface water discharge permits are on five-year renewal cycles, so several renewals will occur during the planning horizon of this Master Plan. As these permits are renewed, additional surface water discharge requirements could be included. This section includes discussion of potential requirements that may be applied in renewed versions of these permits. State and Federal policy changes would be the main driver for changes in the District's discharge permits.

Future policy changes related to the recently adopted Nutrients Watershed Permit is the only regulatory factor that is expected to have a significant impact on the WWTP master planning process. The Nutrients Watershed Permit, and the potential for future policy changes associated with this permit, are discussed below. This discussion is followed by a brief overview of other potential policy actions, that are currently unforeseen, but that could result in permit changes in the future.

4.3.2.1 Nutrients Watershed Permit

Nutrients in the San Francisco Bay are a growing concern for the San Francisco Bay Area water quality community. Historically, the San Francisco Bay has not been adversely impacted by nutrient loading. However, there are now indications that the San Francisco Bay's historic resilience to the effects of nutrient enrichment may be weakening. While the definition of "impairment" has not been reached, there is concern that the San Francisco Bay has reached a tipping point that might lead to impairment. Numerous scientific studies are being conducted, such as the development of Nutrient Numeric Endpoints, to understand the impact of nutrients on the San Francisco Bay. As a result, it may be necessary to limit the availability of essential nutrients in SF Bay by implementing some form of nutrient removal at POTWs that discharge to the SF Bay. The following address three potential challenges and strategies are being discussed:

- **Ammonia toxicity and/or inhibition of phytoplankton growth**: Full or partial nitrification to remove ammonia may be required.
- **Eutrophication**: Full or partial denitrification may be required, where total inorganic nitrogen is the limiting nutrient.
- **Undesirable phytoplankton assemblage changes due to the nitrogen to phosphorus ratio**: Phosphorus reduction may be required.

On April 9, 2014, the Regional Board issued the Nutrients Watershed Permit, which sets forth a regional framework to facilitate collaboration on studies that will inform future management decisions and regulatory strategies. The permit includes four key elements for evaluating nutrient load reduction opportunities at POTWs (if supported by sound science):

- POTW optimization
- Sidestream treatment
- POTW upgrades
- Nutrient reduction by other means (including source control, natural treatment systems, diversion of effluent to water recycling, and others)

In response to the Nutrients Watershed Permit, the POTWs are working collectively under the joint powers agency, Bay Area Clean Water Agencies (BACWA), to submit one coordinated study. This study is evaluating the improvements for each POTW that would be needed to reach three effluent nutrient targets (not effluent limits), which are summarized in Table 4-9. As shown, the first level would involve a permit element of minor improvements to optimize nutrient removal, while levels 2 and 3 would involve upgrades to the WWTP.

Table 4-9. Nutrient Targets from the BACWA Nutrients Watershed Permit Study

Level[a]	Permit Element	Ammonia Target, mg/L as N	Total Nitrogen, mg/L	Total Phosphorus, mg/L
1	POTW Optimization	-	-	-
2	POTW Upgrades	2	15	1.0
3	POTW Upgrades	2	6	0.3

[a] Seasonal impacts will also be considered for each level for dry seasons (May 1 to September 30) and wet seasons (October 1 to April 30).

The Nutrients Watershed Permit does not currently include effluent limits that apply to POTWs separate from those already included in the individual NPDES permits for each POTW facility. The only nutrient limits included in the WWTP NPDES Permit is the current effluent limits for ammonia that includes an assumption of considerable dilution (see Table 4-5). Therefore, the WWTP is not operated to specifically provide nitrogen or phosphorus removal. However, when the secondary reactors are operated with an anaerobic zone during the summer months to improve settleability, this practice provides for biological phosphorus removal.

The current Nutrients Watershed Permit will be renewed around June 2019, and then again approximately every five years. During the planning horizon of this Master Plan, therefore, there could be three revisions of the Nutrients Watershed Permit (2019, 2024, and 2029). The Fact Sheet of the current Nutrients Watershed Permit states that the Regional Board anticipates the 2019 reissuance will consider "establishment of performance-based effluent limits for nutrients and may require implementation of treatment optimization or other means to reduce loads or increase assimilative capacity *if scientific studies show results that warrant such activities*" (emphasis added). However, recent discussions between the Regional Board and BACWA indicate that the renewed 2019 Nutrients Watershed Permit would not include any nutrient limits or load caps if the POTW community agrees to contribute additional funds to support ongoing studies to evaluate nutrient impacts on the Bay. This agreement appears to be moving forward.

The soonest nutrient limits would be applied to the WWTP is in the 2024 Nutrients Watershed Permit. It is unclear whether these limits would be load based, or concentration based. Similarly, it is unclear whether limits would apply to both nitrogen and phosphorus, or whether limits would be applied seasonally or year-round. Seasonal load-based limits would be beneficial (relative to year-round concentration-based limits) for the District because of the plans to increase recycled water production.

If the 2024 Nutrients Watershed Permit includes new effluent limits, a five to ten-year compliance timeline for implementation of necessary upgrades would be expected. There is also likely to be a multi-year evaluation period between when POTWs must comply with the 2024 requirements (i.e. between 2029 and 2034) and when new, more stringent requirements would be applied. Thus, it is not expected that more stringent nutrients requirements than those adopted in 2024 will be applied within the 20-year planning horizon of this Master Plan.

4.3.2.2 Other Potential Future Issues

There are a number of regulatory policy issues that are anticipated to be addressed by the Regional Board within the planning horizon being evaluated under this Master Plan. However, these issues are not expected to have a significant impact on the District's treatment facilities. These issues are as follows:

- **Contaminants of Emerging Concerns (CECs):** CECs include a number of constituents, such as endocrine-disrupting compounds, pharmaceuticals, and "personal care products" that have uncertain environmental and human health impacts. These CECs do not currently have WQOs for the District's receiving waters. Without specific WQOs, the Regional Board does not have a mechanism to regulate CECs. Nevertheless, the trend of increasingly stringent requirements for wastewater treatment may result in the adoption of WQOs for CECs. Given the high level of dilution of the WWTP discharge, it is not expected at this time that advanced treatment for CECs would be required even if WQOs are adopted.
- **State Water Board's Toxicity Plan**: The State Water Board is in the process of developing a statewide toxicity plan, whose requirements will supersede toxicity requirements of the Basin Plan, which are currently used as the basis of toxicity requirements in the NPDES Permit. The main elements of this plan, as it is currently drafted, are numeric toxicity objectives, a standardized method of toxicity data analysis, corresponding monitoring and reporting requirements, and provisions for compliance determination. Anticipated impacts of this plan to toxicity requirements in the District's discharge permits are not expected to be a concern because the District's discharge receives a significant dilution credit for toxicity results. The requirements of the plan, once adopted, are expected to be implemented by the Regional Board on a permit-by-permit basis. The Wet Weather Permit expires before the NPDES Permit but does not include effluent toxicity requirements. Therefore, any changes related to this statewide toxicity plan would be implemented with renewal of the NPDES Permit, which is expected to occur in 2022.
- **Bacterial Objectives for the Water Contact Recreation Beneficial Use:** The State Water Board is in the process of adopting updated water quality objectives for bacteria that could slightly lower the objective for enterococcus bacteria that currently applies in marine waters (e.g., the Lower SF Bay) (State Water Board, 2017). However, the proposed changes are not expected to be a concern because the District receives a dilution credit of 10:1 for bacteria concentrations, and the average enterococcus concentration reported in the NPDES Permit is well below the revised objective after accounting for dilution.

- **Total maximum daily load (TMDL) determinations for chlordane, dichlorodiphenyltrichloroethane (DDT), or dioxin:** Legacy pesticides (chlordane, DDT, and dioxin) have been identified in the USEPA's 303(d) list as impairing the Lower SF Bay, based on information provided by the Regional Board to the USEPA. However, the Regional Board is currently considering removing (delisting) these legacy pesticides (San Francisco Estuary Institute, 2014) from the 303(d) list. Given this uncertainty, it would not be prudent to address removal requirements for these pollutants at this time.
- **TMDL determinations for invasive species and trash:** The Regional Board also has plans to establish TMDLs in the Lower SF Bay for invasive species by the year 2019 and trash by the year 2020. Specific sources of these impairments have not been identified, although trash has been regulated by the Regional Board from stormwater discharges to the SF Bay. In any case, impairment by invasive species is not related to POTW discharges, and trash is readily controlled through POTWs. Therefore, neither of these impairments are relevant to the WWTP.

4.4 RECYCLED WATER REQUIREMENTS

This section details current and potential future discharge requirements related to reuse of WWTP effluent (i.e., recycled water). The following specific topics are addressed:

- Overview of Current Recycled Water Uses
- Current Recycled Water Permitting
- Water Quality Requirements
- Operational Requirements
- Requirements of the New Statewide General Permit
- Potential Future Recycled Water Use Requirements

4.4.1 Overview of Current Recycled Water Uses

As detailed in Chapter 3, the District currently produces "Disinfected Tertiary" recycled water for irrigation purposes in two separate treatment trains: a GMF/UV system and a MF/UV system.

The recycled water produced is distributed to irrigation customers within the water supply service areas of DSRSD and EBMUD under the San Ramon Valley Recycled Water Program (SRVRWP). The SRVRWP is owned and operated by DERWA, a joint powers authority formed by the two districts. The City of Pleasanton is also in the process of implementing a recycled water program that will receive recycled water from the WWTP. The initial phases of this project will be online by the end of 2015. Offsite use of recycled water is also facilitated via a commercial fill station that has been operational at the WWTP since 2007. From 2014 through 2016, in response to severe drought, the District operated a residential recycled water fill station, allowing the public to fill personal containers with limited volumes of recycled water.

4.4.2 Current Recycled Water Permitting

Recycled water uses within California are subject to the public health protection-related regulatory requirements detailed in the State Water Board's Division of Drinking Water reclamation criteria, found in California Code of Regulations Title 22, Division 4, Chapter 3 (Title 22). The Regional Board has adopted a region-wide permit – the General Order – that enforces the Title 22 requirements, and can serve as a discharge permit for POTWs and water agencies that recycle treated municipal wastewater, such as the District.

The General Order was adopted to replace individual recycled water use permits within the San Francisco Bay Area. As indicated in Table 4-1, the District is a current permittee under the General Order for the recycled water uses described above. The Regional Board had previously adopted a site-specific permit (Order 93-159) in 1993 for the District's recycled water use. In 2014 the Regional Board formally rescinded this site-specific permit – which was issued jointly to DSRSD, the Zone 7 Water Agency, and Livermore – because the permittees had by then enrolled under the later General Order, and a site-specific permit was no longer needed.

4.4.3 Water Quality Requirements

Consistent with Title 22, Prohibition B of the General Order details minimum recycled water quality allowed for permitted uses. "Disinfected Tertiary" recycled water needs to be adequately oxidized, filtered, and disinfected to meet specific concentration limits for dissolved oxygen, dissolved sulfide, total coliform organisms, and turbidity. In addition, "Disinfected Tertiary" recycled water that is filtered through a granular media filter needs to be coagulated prior to filtration except when the turbidity of the influent to the granular media filtration system does not exceed 5 Nephelometric Turbidity Units (NTUs) more than five percent of the time. Coagulation is not required per Title 22 for production of "Disinfected Tertiary" recycled water using a membrane filtration system.

In addition, Prohibition B.6 of the General Order requires that tertiary recycled water that is disinfected using any non-chlorine process (like the UV systems at the WWTP) must have been treated through a disinfection process that, when combined with the filtration process, has been demonstrated to reduce the concentration of specific microorganisms in the filter effluent to "one hundred thousandths…of the initial concentration in the filter influent throughout the range of qualities of wastewater that will occur during the recycled water treatment process." Demonstration of this alternative disinfection process must also be approved by the State Water Board staff overseeing recycled water use as well as the Regional Board's Executive Officer. The District has completed the required validation testing for the two onsite UV systems.

The specific concentration limits in the General Order for "Disinfected Tertiary" water are summarized in Table 4-10. Compliance with these recycled water limits is verified through routine monitoring of the District's recycled water pursuant to the General Order's Self-Monitoring Program (Attachment C of the General Order).

Table 4-10. Recycled Water Limits in the General Order	
Parameter	Limit Description
Dissolved Oxygen	Minimum of 1.0 mg/L
Dissolved Sulfide	Maximum of 0.1 mg/L
Total Coliform Organisms	Maximum 7-day median of 2.2 MPN/100 mL
	Maximum in any 30-Day period of 23 MPN/100 mL
	Maximum (at any time) of 240 MPN/100 mL
Turbidity	Maximum daily average of 2 Nephelometric Turbidity Units (NTUs)
	Maximum of 5 NTUs no more than 5 percent of the time
	Maximum (at any time) of 10 NTUs

4.4.4 Operational Requirements

The General Order includes operational requirements related to the storage basins and the irrigation reuse areas.

4.4.4.1 Storage Basin Operational Requirements

The recycled water storage operational requirements listed under Prohibition B.7 of the General Order are as follows:

> a. *Any storage facility containing recycled water for reuse applications shall be managed in a manner to control odor or nuisance conditions. Should such problems develop, a management plan shall be devised and implemented to monitor, correct, and control future occurrences.*
>
> b. *All wastewater storage ponds shall be adequately protected from erosion, washout and flooding from a 24-hour rainfall event having a predicted frequency of once in 100 years.*

The first storage requirement is related to odor control, which is discussed later in this chapter. The second storage requirement requires protection of recycled water storage facilities from erosion and washout during a 100-year flood. The main treatment and storage areas of the WWTP are not within a mapped 100-year floodplain. Therefore, compliance with this requirement is not an issue.

4.4.4.2 Reuse Area Operational Requirements

The applicable reuse area operational requirements listed in the Prohibition A of the General Order prohibit the following:

- Creating nuisance during the treatment, storage, distribution, or reuse of recycled water;
- Applying recycled water during periods when soils are saturated;
- Allowing recycled water to runoff from reuse areas or pond onsite;
- Allowing spray or runoff to enter a dwelling or food-handling facility or contact a drinking water fountain;
- Applying recycled water in groundwater recharge or wellhead protection areas;
- Causing rising groundwater discharging to surface water to impair WQOs or beneficial uses of the surface water (as defined in the Basin Plan);
- Discharging to waters of the State in a manner that unreasonably affects present or anticipated beneficial uses or that results in water quality less than that prescribed in water quality control plans (i.e., the Basin Plan) or policies;
- Discharging recycled water in a manner not permitted by the General Order;
- Using recycled water for domestic or animal water supply; and
- Cross-connecting between potable water supply and piping containing recycled water.

The General Order's Self-Monitoring Program also includes two inspection requirements for the recycled water producer (i.e., the District) to ensure compliance with these management prohibitions, specifically, and with Title 22 criteria, generally. First, the District is required to inspect, or require the recycled water users to inspect, the reuse areas at least annually. These routine inspections are required to include standard observations for items related to the land management prohibitions. (For the users to conduct these inspections, such requirements would need to be specified in Water Reuse Permits issued by the District to the user.) Second, the District is required to conduct periodic *random* inspections of recycled water users to ensure compliance with Title 22 criteria and with specific requirements in the user's Water Reuse Permit.

4.4.5 Requirements of the New Statewide General Permit

The State Water Board statewide general permit for use of recycled water from POTWs (State Water Board, 2016) will supersede the General Order by June 2019 at the latest. The statewide permit includes similar requirements as the General Order, with the exception that it does not include recycled water limits for dissolved oxygen or dissolved sulfide and does not include the storage basin (pond) operating requirements found in the General Order.

Even though the statewide permit does not include limits for dissolved oxygen or sulfide, it does prohibit the recycled water use from creating nuisance conditions, and the reuse area must be monitored quarterly for "nuisance odors/vectors." This is similar to what the General Order requires, except that the General Order also requires the source and direction of the odor to be identified if an odor of wastewater origin is observed.

The District's current recycled water permit and this statewide general permit are effectively similar for purposes of this Master Plan. Therefore, implementation of the new general permit for the District's recycled water use is not expected to impact the findings of this Master Plan.

4.4.6 Potential Future Recycled Water Use Requirements

Significant changes to Title 22 or the General Order related to irrigation reuse of "Disinfected Tertiary" recycled water are not anticipated at this time. In addition, while the General Order is expected to be superseded by the statewide permit for municipal recycled water uses, that permit does not have an expiration date and so is not subject to changes as part of a periodic renewal process. However, the State Water Board could initiate a review and revision process if warranted by changes in the governing water quality policies. Such a revision is not currently anticipated. Therefore, potential future recycled water use requirements are expected to be similar to the existing requirements detailed above.

4.5 BIOSOLIDS SURFACE DISPOSAL REQUIREMENTS

The biosolids generated by the WWTP are stored (and receive some additional treatment) in the FSLs and are then injected into and incorporated into the soil at the 46-acre DLD site. This surface disposal of biosolids is permitted primarily under the following regulations (by jurisdiction level):

- *Federal:* Title 40 of the Code of Federal Regulations, Part 503 (40 CFR 503)
- *State:* California Code of Regulations Title 27 (Title 27)
- *Local:* Biosolids Permit

This section presents an overview of the current and potential future discharge requirements that are most relevant to the disposal of biosolids at the DLD site. The following specific topics are addressed:

- Title 27 Compliance Requirements
- Metals Loading Limits
- Soil Treatment Requirements
- Groundwater Concentration Limits
- Pathogen and Vector Attraction Reduction Standards
- Monitoring and Reporting Requirements
- Potential Future Biosolids Disposal Requirements

Note that odor control requirements in the Biosolids Permit are discussed later in this chapter with other air quality requirements for the WWTP activities.

4.5.1 Title 27 Compliance Requirements

Title 27 contains regulatory requirements for the treatment, storage, processing, and disposal of hazardous wastes. As noted in the Biosolids Permit, the Regional Board classifies the District's biosolids as "designated waste" (non-municipal, non-hazardous waste) and the DLD site as a Class II Land Treatment Unit subject to Title 27.

Specification 16 of the Biosolids Permit is a blanket requirement for the District to comply with all applicable provisions of Title 27 that are not specifically referred to in the permit. The Regional Board has, however, incorporated the Title 27 requirements most critical to master planning into the Biosolids Permit. The remainder of this section thus focuses on requirements detailed in the Biosolids Permit, including those related to Title 27.

4.5.2 Metals Loading Limits

The Biosolids Permit does not include specific limits on metals loading to the DLD site. However, the 40 CFR 503 includes a number of requirements for metals loading. For an active sewage sludge unit that does not have a liner or leachate collection system, concentrations of three metals (arsenic, chromium, and nickel) in the applied biosolids must be below specific dry-weight concentrations (loadings). The value of these metals loading limits depends on the distance of the site from the property boundary, with the most stringent limits applying to sites within 25 meters (about 80 feet) of the property line. The active biosolids disposal units on the DLD site are located within 25 meters of the property line, so these most stringent metals loadings limits apply. These metals loading limits are summarized in Table 4-11.

Table 4-11. Metals Loading Limits for Biosolids Disposed at the DLD Site

Parameter	Concentration,[(a)] mg/kg[(b)] (dry weight)
Arsenic	30
Chromium	200
Nickel	210

(a) "Pollutant concentrations" are from Table 2 of Section 503.23 of the 40 CFR 503, for a biosolids disposal site within 25 meters of the property line.
(b) Milligrams per kilogram.

4.5.3 Soil Treatment Requirements

Specification B.10 of the Biosolids Permit expresses the Regional Board's view that the unsaturated native soils extending to a depth of 5 feet beneath the initial surface of the DLD site are a "treatment zone" as defined in Section 20250 of Title 27. Specification B.11 of the permit requires the District to maintain soil pH within this treatment zone at or above 5.0. However, the Biosolids Permit does not include any soil monitoring requirements.

The only requirements to monitor pH are in groundwater or in seepage water, when seepage is found to be occurring. Specifically, the permit requires semi-annual monitoring of pH as one of the "general water quality parameters" in the DLD site's monitoring wells, as well as daily monitoring of pH in any seepage (as long as seepage is occurring).

4.5.4 Groundwater Concentration Limits

Per requirements of Title 27, the Biosolids Permit includes groundwater concentration limits for "constituents of concern," which are waste constituents reasonably expected to be in the discharge. The constituents of concern and groundwater concentration limits are listed in Table 4-12. For most of the constituents, the limits are defined as being the higher of the routine Practical Quantitation Limit (PQL) for the constituent or the "background value." To date, the District has not defined these background concentrations because a limited number of data points are available, making it difficult to conduct a meaningful statistical evaluation (DSRSD, 2012) and establish background values. For pesticides, PCBs, and chlorophenoxy herbicides, the concentration limits are just the routine PQLs, which essentially requires that these constituents be below levels that can be reliably measured (i.e., Reporting Levels).

Table 4-12. Groundwater Concentration Limits for Biosolids Application at DLD Site

Constituent of Concern	PQL, µg/L[a]	Concentration Limits, µg/L[b]
Arsenic	7	Higher of PQL or background value[c]
Barium	20	
Cadmium	5	
Chromium	10	
Copper	10	
Lead	5	
Mercury	1	
Nickel	40	
Vanadium	10	
Zinc	20	
Antimony	5	
Beryllium	5	
Cobalt	10	
Selenium	10	
Silver	20	
Thallium	5	
Tin	50	
Pesticides and PCBs	Varies	PQLs[d]
Chlorophenoxy Herbicides	Varies	

[a] The values of the PQLs are as listed in Table 2 of the Biosolids Permit, which does not specify the units for the PQLs. Units of µg/L, which are listed for the concentration limits, are assumed.

[b] Table 2 of the Biosolids Permit lists the units of the concentration limits as parts per billion (ppb), which is equivalent to the more commonly used µg/L that is presented instead here.

[c] The concentration limit is the higher of either the routine PQL or the background value. Background values are not defined in the Biosolids Permit.

[d] Table 1 of the Biosolids Permit clarifies that the required test methods (which impacts PQLs) are USEPA Method 8080 for pesticides and PCBs, and USEPA Method 8151 for chlorophenoxy herbicides.

The District's groundwater concentration limits must be met at the "point of compliance" for the DLD site, which is determined based on results of groundwater monitoring conducted pursuant to the Biosolids Permit. Specification B.13 of the permit specifically requires that the District "maintain the DLD site so as to prevent a statistically significant increase in water quality parameters at points of compliance…"

When the Biosolids Permit was adopted, the District had six existing "point of compliance" groundwater wells located around the FSLs. Provision C.4 of the permit required that additional wells be installed around the DLD site and an installation report provided to the Regional Board by December 1, 2007. The additional monitoring wells were required to be installed to monitor both an upper aquiclude and a deeper upper aquifer. Accordingly, eight new monitoring wells were installed in pairs in four locations around the DLD site, as documented in a November 2007 installation report (Luhdorff & Scalmanini, 2007).

As discussed above, the District has not collected sufficient data from the new wells to evaluate whether a statistically significant increase in water quality concentrations has occurred.

4.5.5 Pathogen and Vector Attraction Reduction Standards

The 40 CFR 503 and Biosolids Permit require that Class B pathogen reduction standards must be met prior to biosolids disposal. These Class B pathogen reduction standards can be met by using one of the following three options:

1. Monitoring of indicator organisms, specifically fecal coliform density at the time of biosolids use or disposal, to achieve a seven-sample geometric mean of less than 2 million MPN (of organisms) per gram of biosolids (dry weight).
2. Use of a Process to Significantly Reduce Pathogens (PSRP), which includes the following:
    a. Aerobic digestion (to achieve a mean cell residence time of the biosolids between 40 days at 20°C and 60 days at 15°C)
    b. Air drying (for a minimum of 3 months and, during two of the three months, ambient average daily temperature must be above 0°C)
    c. Anaerobic digestion (to achieve a mean cell residence time of the biosolids between 15 days at 35-55°C and 60 days at 20°C)
    d. Composting (to maintain a biosolids temperature of at least 40°C for five days and at least 55°C for at least four hours during the 5-day period)
    e. Lime stabilization (to raise the biosolids pH to 12 after two hours of contact)
3. Use of a Process Equivalent to PSRP. (Such a process would need to be determined by a permitting authority and generally requires pilot testing unless conditions are exactly like another facility.)

The 40 CFR 503 and Biosolids Permit also require that one of ten vector attraction standards detailed in the 40 CFR 503 be met for surface disposal of biosolids. These vector attraction reduction standards, as detailed in the 40 CFR 503, include the following:

1. Reduce the mass of volatile solids by a minimum of 38 percent.
2. Demonstrate vector attraction reduction with additional anaerobic digestion in a bench scale unit.
3. Demonstrate vector attraction reduction with additional aerobic digestion in a bench scale unit.
4. Meet a specific oxygen uptake rate for aerobically treated biosolids.
5. Use aerobic processes at greater than 40°C (with average temperatures above 45°C) for at least 14 days.
6. Add alkaline materials to raise the biosolids pH to at least 12 for two hours and then at least 11.5 for an additional 22 hours.
7. Reduce moisture content of biosolids that do not contain unstabilized (i.e., not aerobically or anaerobically digested) solids generated in a primary wastewater treatment process shall be equal to or greater than 75 percent based on the moisture content and total solids prior to mixing with other materials.
8. Reduce moisture content of biosolids that contains unstabilized solids to at least 90 percent.
9. Inject biosolids below the surface of the land (within one hour for Class B biosolids)
10. Incorporate biosolids applied to or placed on the land surface into the soil within six hours after application or placement.

The District currently employs anaerobic digestion to meet both the pathogen reduction (Option 2.c) and vector attraction reduction (Option 1) standards to comply with the requirements of the 40 CFR 503 and Biosolids Permit.

4.5.6 Monitoring and Reporting Requirements

The Biosolids Permit includes limited monitoring requirements to ensure compliance with the various requirements of the permit detailed above. For instance, the District's groundwater monitoring wells must be sampled semi-annually for dissolved metals, and monitoring once every five years is required for the additional metals and organic compounds that have groundwater concentration limits (as listed in Table 4-12). Semi-annual monitoring is also required various "general water quality parameters", which includes temperature, specific conductance, pH, turbidity, Total Dissolved Solids, chloride, nitrate, and ammonia. The groundwater monitoring results are reported semi-annually.

Monitoring of biosolids is not specifically detailed in the Biosolids Permit, although the 40 CFR 503 has specific monitoring requirements. In addition, the NPDES Permit's MRP (Attachment E of the NPDES Permit) and Regional Standard Provisions (Attachment G of the NPDES Permit) both require biosolids monitoring. The NPDES Permit's MRP specifically requires samples to be collected annually from biosolids that are to be disposed. These samples are required to be tested for various organic compounds, cyanide, and various metals (arsenic, cadmium, copper, hexavalent chromium, lead, nickel, selenium, silver, and zinc). The NPDES Permit's Regional Standard Provisions also require, for surface disposal sites, that biosolids be monitored for arsenic, (total) chromium, and nickel (i.e., the constituents with metals loading limits in Table 4-11). The frequency of this separate metals monitoring depends on the volume of biosolids disposed. In recent years, the District has disposed of about 2,500 dry metric tons of biosolids per year (DSRSD, 2013), which requires monitoring six times per year.

An additional monitoring requirement for the DLD site is included in a 2012 letter from the USEPA. In early 2012, the District had requested permission from the USEPA to grow and harvest fodder crops at the DLD site, and this request was approved in an USEPA letter dated December 20, 2012 (USEPA, 2012). The USEPA's approval was based results of 2011-2012 pilot testing of the District's cropping plan and related tissue sampling of crops grown onsite during the pilot testing. The USEPA specifically required that plant tissue samples continue to be collected at harvest time to assess pollutant uptake. The USEPA also required that the results of these assessments be included in the District's annual biosolids reports submitted to the USEPA. The USEPA letter does not list specific limits to plant tissue results but requires the District to notify the USEPA if any evidence is found of phytotoxicity in the crops.

4.5.7 Potential Future Biosolids Disposal Requirements

Significant changes to the District's Biosolids Permit or the 40 CFR 503 are not anticipated at this time. In addition, the Biosolids Permit does not have an expiration date and so is not subject to changes simply as part of a renewal process. However, the District or the Regional Board could initiate a review and revision process if warranted by changes in the governing water quality policies or changed characteristics of the permitted biosolids disposal practices. Changes to the Biosolids Permit would likely only occur under one of two conditions.

One condition would be if, for any reason, the District needed to begin disposing of biosolids at a different site or use a different treatment process onsite for biosolids disposal. This condition would require alternatives to the District's current biosolids practices. The District would need to submit a new Report of Waste Discharge (i.e., permit application) to the Regional Board to receive a separate discharge permit or revised Biosolids Permit prior to the change in operations. Further evaluation of such alternatives and any related regulatory requirements that would then apply are presented in later chapters of this Master Plan.

A second condition under which changes to the Biosolids Permit could occur is if the Regional Board had concerns regarding the District's compliance with Title 27 requirements at the DLD site. Compliance concerns could arise if groundwater quality results exceeded the groundwater concentration limits for the DLD site-specific constituents, as detailed above. Such exceedances may indicate a release from the DLD site and potential offsite migration.

If a release is reported to the Regional Board, the Regional Board would be expected to order the District to cease operation of the land treatment units at the DLD site and not resume operations until one of two actions are taken pursuant to Title 27. The District would need to either (1) commit to changing practices at the DLD site or (2) retrofit the treatment units with liners; and construct a leachate collection and removal system. For both actions, Title 27 also requires that the District complete appropriate removal or remedial actions to the satisfaction of the Regional Board and submit an amended Report of Waste Discharge (i.e., permit application).

As discussed above, background values have not yet been established for the constituents of concern at the DLD site. Therefore, it has yet to be determined whether there has been a measurably significant increase in one or more constituents of concern relative to background values.

4.6 AIR QUALITY REQUIREMENTS

The WWTP includes a number of facilities that are currently subject, or potentially subject, to air quality requirements. This section presents a discussion of current and potential future air quality requirements that may be applicable to the WWTP. The following specific topics are addressed:

- Air Emission Requirements
- Greenhouse Gas (GHG) Emission Requirements
- Odor Control Requirements
- Potential Future Air Quality Requirements

4.6.1 Air Emission Requirements

Air emissions are currently regulated through the District's Air Permit. This section addresses the air emission standards that are applicable to the WWTP followed by a discussion of the current Air Permit requirements.

4.6.1.1 Applicable Air Emission Regulations

This section addresses the two primary air emission standards that are the basis of the District's BAAQMD Air Permit requirements: the BAAQMD Combustion Engine Requirements and the CARB Hazardous Pollutant Standards.

4.6.1.1.1 BAAQMD Combustion Emission Requirements

There are three key regulations adopted by BAAQMD that are applicable to the WWTP operations: Regulation 6, Regulation 9, Rule 1 and Regulation 9, Rule 8.

Regulation 6 (Particulate and Visible Emissions) includes a limit of Ringelmann 1 that applies to any source of particulate emissions (except open fires). The Ringelmann scale is measure of the apparent density of smoke, with a value of 1 corresponding to an opacity of 20 percent. Regulation 6) does not define particulate emission limits in terms of grams per brake horsepower per hour.

Regulation 6 also includes a narrative limitation for visible particles, as follows:

> *A person shall not emit particles from any operation in sufficient number to cause annoyance to any other person, which particles are large enough to be visible as individual particles at the emission point or of such size and nature as to be visible individually as incandescent particles. This…shall only apply if such particles fall on real property other than that of the person responsible for the emission.*

Regulation 9, Rule 1 (Sulfur Dioxide) establishes emission limits for sulfur dioxide (SO_2) from all sources including ships, and limits ground level concentrations of sulfur dioxide. Regulation 9, Rule 1 specifically states that "A person shall not burn any liquid fuel having a sulfur content in excess of 0.5 percent by weight, or solid fuel of such sulfur content as would result in the emission of a gas stream containing more than 300 ppm (dry) of sulfur dioxide."

Regulation 9, Rule 8 (NOx and CO from Stationary Internal Combustion Engines) limits nitrogen oxide (NOx) and carbon monoxide (CO) emissions from stationary Internal Combustion (IC) engines[6] having a rated brake horsepower (bhp) of 50 bhp or more. Certain exemptions apply for emergency standby engines, as well as any IC engine operated less than 100 hours in any 12-month period (exclusive of emergency use).

The BAAQMD emission limits for NOx and CO are defined separately for spark-ignited engines powered by fossil-derived fuels, spark-ignited engines powered by waste-derived fuels, and compression-ignited (CI) engines. Only the latter two are relevant to the District, since the cogen engines are spark-ignited engines powered by waste-derived fuels, and the emergency generators have CI engines. The BAAQMD emission limits on NOx and CO are summarized in Table 4-13 for these two engine types, in units of ppm by volume dry basis (ppmvd).

Table 4-13. BAAQMD Combustion Emission Limits Applicable to the WWTP

Engine Type	Emissions Limit, ppmvd	
	NOx	CO
Cogen Engines		
Spark-ignited engine powered by waste-derived fuel, >50 bhp	70	2,000
Emergency generator engines[(a)]		
CI (diesel) engine, >175 bhp	110	310

[(a)] These combustion emission limits do not apply if the engine is used for emergency purposes or non-emergency purposes for less than 100 hours in a 12-month period.

[6] A stationary IC engine is defined in the rule as "[a]ny spark or compression ignited internal combustion engine operated, or intended to be operated, at a specific site for more than one year or attached to a foundation at a site."

For existing engines, Regulation 9, Rule 8 allows for delayed compliance with the emission limits detailed in Table 4-13 until January 1, 2016. The delayed compliance is contingent upon specific alternative requirements being met, including notifying the BAAQMD of election to delay compliance by January 1, 2012. Delayed compliance for operating an existing spark-ignited engine is also contingent upon both of the following:

- The engine is rated between 51 to 250 bhp (i.e., has a maximum rating of 250 bhp) or has a model year of 1996 or later; and
- The engines comply with Best Available Control Technology (BACT) requirements for a stationary IC, spark-ignited engine no later than January 1, 2016.

Delayed compliance for operating an existing CI engine is contingent upon either of the following, in addition to the BAAQMD notification requirement:

- NOx and CO emissions shall not exceed BACT limits for a stationary IC, compression-ignited engine; or
- The NOx emissions shall not exceed 22 ppmvd, and CO emissions shall not exceed 310 ppmvd.

4.6.1.1.2 CARB Hazardous Air Pollutant Standards

USEPA issues requirements for limiting hazardous air pollutants (HAPs) and sets national emissions standards for HAPs for major point sources. Existing facilities are subject to the federal CAA requirements to implement maximum achievable control technology only if a process is modified to emit toxic air contaminants above the "major" threshold (10 tons/year of a single HAP or 25 tons/year of a combination of HAPs). California has its own comprehensive air quality control program that continues to be more restrictive than national requirements. Therefore, CARB requirements tend to govern emissions controls in most cases.

CARB adopts Airborne Toxic Control Measures (ATCMs) to regulate emissions of HAPs, which are referred to by CARB as "toxic air contaminants" (TACs). Since 1998, CARB has considered diesel particulate matter a TAC. CARB adopted an ATCM for stationary CI engines in 2004 to reduce emissions of diesel particulate matter from these engines. The ATCM also limits emissions of nitrogen oxides and carbon monoxide from diesel engines.

Following adoption of the ATCM for particulate matter, the EPA promulgated new federal performance standards for CI engines. In 2010 CARB approved amendments to the 2004 ATCM for diesel engines to closely align CARB's requirements with the federal requirements (CARB, 2010). The amended ATCM became effective May 19, 2011.

The current ATCM for diesel engines includes the following requirements that would apply to any new or replacement CI engine with a model year of 2008 or later:

- Comply with a particulate matter emission limit of 0.15 grams per brake horsepower per hour (g/bhp-hr) for CI engines sized greater than or equal to 50 horsepower (HP).
- Limit annual maintenance and testing hours to 50 hours per calendar year.

4.6.1.2 Current Air Permit Requirements

This section addresses the following topics related to the District's current Air Permit requirements:

- WWTP Facilities Subject to the Air Permit
- Emissions Limits
- Operational Requirements

The Air Permit also includes various monitoring and reporting requirements that do not warrant discussion in this Master Plan and are thus not covered.

4.6.1.2.1 Facilities Subject to the Air Permit

A number of specific facilities at the WWTP are listed in the Air Permit as subject to specific air quality requirements, including the following:

- Three IC cogeneration (cogen) engines fueled by either digester gas or natural gas;
- A digester waste gas burner/flare (for digester gas that is produced in amounts greater than demand), with a maximum capacity of 2,700,000 British Thermal Units (BTUs) per hour;
- Three CI (diesel) engines each used to drive an emergency generator; and
- Three diesel-fueled portable pump engines.

In addition, the Air Permit regulates air quality from the various treatment components of the WWTP, which are referenced generally by treatment type (e.g., primary treatment, secondary treatment, anaerobic digestion).

The various WWTP engines subject to the Air Permit are further detailed in Table 4-14, which includes the BAAQMD source number for each engine and its power rating, as listed in the Air Permit. In addition, any notes in the Air Permit regarding an approved abatement measure or exemption of the source are provided. The additional facilities subject to the Air Permit are listed in Table 4-15 with their source numbers and notes regarding approved abatement measures.

Table 4-14. WWTP Engines Subject to the Air Permit

Engine Description	Source Number	Power, HP	Abatement/Exemption
Cogen Engines			
Cogen Engine #1	S-13	740	Abated by: A-5 Non-Selective Catalytic Reduction (NSCR)
Cogen Engine #2	S-22	706	
Cogen Engine #3	S-21	592	Abated by: A-4 NSCR
Emergency Generator Engines			
Portable Standby Diesel Engine	S-23	232	(a)
Standby Diesel Engine	S-25	2,220	Abated by: A-25 Diesel Oxidation Catalyst
Standby Diesel Engine	S-26	1,125	
Pump Engines			
Portable Water Pump Standby Diesel Engine	S-24	71	
Portable Diesel-Fired Pump Engine	S-27	74	
Portable Standby Pump Engine	S-28	54	

(a) Although the S-23 is listed with the other sources and not specifically noted as "exempt," the Air Permit does not include any conditions specific to this source.

Table 4-15. Additional WWTP Facilities Subject to the Air Permit

Facility Description	Source Number	Abatement/Exemption
Natural Gas-Fueled Standby Boiler	S-14	(a)
Industrial Flare (Waste Gas Burner)	S-15	Exempt(b)
Municipal Wastewater Treatment Plant	S-100	
Preliminary Treatment	S-110	Abated by: A-110 Packed Bed Scrubber and A-2 Packed Bed Scrubber
Primary Treatment	S-120	
Flow Equalization	S-130	
Secondary Treatment	S-140	
Secondary Clarifiers	S-150	
Disinfection	S-160	
Sludge Handling	S-170	Abated by: A-6 Packed Bed Scrubber/ A-170 Soil Media Biofilter(c)
Anaerobic Digesters	S-180	Abated by: A-15 Waste Gas Flare(d)

(a) Although the boiler (S-14) is listed with the other sources and not specifically noted as "exempt," the Air Permit does not have any conditions specific to this source.

(b) Although noted as "exempt" as a source in the Air Permit, the District's flare also appears to be referenced as abatement measure A-15 and subject to a number of requirements in the permit per that reference.

(c) The Air Permit appears to refer to A-6 and A-170 interchangeably.

(d) Digester gas from the anaerobic digesters are required to be used to fuel the cogen engines S-13 or S-21 or be abated by A-15 (i.e., flared) as a result of (1) gas production exceeding available energy recovery capacity, (2) equipment testing, or (3) emergency conditions.

As indicated in Tables 4-14 and 4-15, the Air Permit lists two facilities as sources but does not include any specific emissions or operational requirements for them: a 232-HP portable standby diesel engine (S-23) and a natural gas-fueled boiler that is used as a backup for heating the digesters (S-14). These sources do not require further discussion since they have no specific air quality requirements. In addition, the WWTP includes two other boilers that are relatively small and used as a backup to the cogen facilities for heating WWTP buildings. Due to their size and use, these boilers are unaddressed in the Air Permit and so are also not discussed herein.

4.6.1.2.2 Emission Limits

Requirements that limit emissions of specific air pollutants are generally limited to the cogen engines and the following compounds:

- NOx, including nitrogen dioxide (NO_2)
- CO
- Non-methane hydrocarbons
- SO_2

The emission limits for the cogen engines are summarized in Table 4-16. The emission limits for engines S-13 and S-21, which are the two engines that have abatement measures in place, are expressed in the same units as the as Regulation 9, Rule 8 emission limits in Table 4-13 (parts per million by volume corrected to 15 percent oxygen, dry basis). Comparing the Regulation 9, Rule 8 emission limits in Table 4-13 to the current Air Permit limits for these two engines provided in Table 4-15, demonstrates that the current Air Permit is more restrictive for engines S-13 and S-21. The emission limits for engine S-22, which does not have abatement, are listed in units of grams per brake horsepower per hour and cannot be compared to the Regulation 9, Rule 8 emission limits without stack flow information.

Table 4-16. Air Permit Emission Limits for Cogen Engines

Facility Name	BAAQMD Source Number	Emissions Limit				
		NOx (as NO_2)	CO	NHMC	SO_2	Units[a]
Cogen Engine #1	S-13	25	200	150[b]	-	ppmvd
Cogen Engine #2	S-21	9	56	-	-	ppmvd
Cogen Engine #3	S-22[c]	1.0	2.65	0.6	0.3	g/bhp-hr

(a) Units are defined as follows:
ppmvd = parts per million by volume corrected to 15 percent oxygen, dry basis
g/bhp-hr = grams per brake horsepower per hour
(b) NHMC emissions limit for S-13 is noted as "ppmvd as methane."
(c) S-22 also has a limit on visible particulate emissions, which cannot exceed a Ringelmann 1.

In addition, as noted in Table 4-16, the Air Permit includes a particulate emission limit of a Ringelmann 1 that applies only to S-22, which is a 71-hp engine. The Air Permit does not include particulate emission limits for the other cogen engines (S-13 and S-21).

The Air Permit also notes that S-24 engine is subject to the requirements of Regulation 9, Rule 1 and Regulation 6. However, specific emission limits called out by these regulations are not listed. The Air Permit also notes that Engine S-24 "may be subject to other [BAAQMD] regulations, including Regulation 9, Rule 8 in the future." The Air Permit does not include emission limits for the other diesel engines listed in the permit.

Finally, the permit prohibits the waste gas flare (A-15) from emitting any visible particulate emissions for three consecutive minutes in any hour or that result in fallout on adjacent property in such quantities as to cause a public nuisance.

4.6.1.2.3 Operational Requirements

The Air Permit includes operational requirements related to the engines, the treatment facilities, the flare (A-15), and emergency and portable engines.

4.6.1.2.3.1 Engine Operational Requirements

In addition to specific emission limits, the cogen engines must be operated per specific operational requirements in the Air Permit. The operational requirements that are especially relevant to the District for current master planning purposes are as follows:

- Engine S-22 may be fired exclusively on either natural gas or digester gas.
- Engine S-13 must be fired principally on natural gas and secondarily on digester gas, with a combined thermal throughput not exceeding 51,200 MMBTU during any cumulative 12-month period.
- During operation, Engine S-13 must be abated by a properly operated and maintained A-5, three-way catalyst with air/fuel control.
- Engine S-21 must be fired principally on natural gas and secondarily on digester gas, in the event Engines S-13 or S-22 are unable to adequately process the digester gas load, with a combined thermal throughput not exceeding 45,550 MMBTU during any cumulative 12-month period.
- During operation, Engine S-21 must be abated by a properly operated and maintained A-4, three-way catalyst with air/fuel control.

The main focus of these operational controls are that the engines are operated on natural gas when possible. The Air Permit also includes a clear requirement (Condition 6) that collected gases from the digester (S-180) must be abated "at all times" by the digester flare (A-15) or be used by Engines S-13 or S-21. Condition 5 also prohibits any venting of raw digester gas to the atmosphere.

4.6.1.2.3.2 Treatment Facility Operational Requirements

Operational requirements for the treatment facilities include the specific abatement measures noted in Table 4-15 for S-170 (sludge handling) and S-180 (anaerobic digestion). Additional operational requirements for these facilities that are especially relevant for current master planning purposes are discussed below.

One condition limits the wastewater flow (called "throughput" in the permit) at the various treatment facilities listed in Table 4-15 (i.e., sources S-100, S-110, S-120, S-130, S-140, S-150, S-160, S-170, and S-180) to 17 mgd. A separate condition of the permit specific to S-100 (i.e. the treatment plant) indicates that the 17 mgd flow limit is for a calendar month average during dry weather periods. This separate condition also includes a wet weather monthly average flow limit of 74 mgd at S-100, with "wet weather" defined as the "months of October through May." These flows are consistent with the design flows associated with the Stage 4 project.

A second operational requirement for the treatment facilities (S-100, S-110, S-120, S-130, S-140, S-150, S-160, S-170, and S-180) and related abatement measures (A-2, A-15, A-100, and A-170) indicates that the permission to operate these facilities is contingent upon compliance with the BAAQMD nuisance and odor requirements. If three or more notices of odor violations have been issued within a 30-day period, then the District is required to submit within 60 days of receiving the third notice an application to construct additional abatement measures to control odor.

4.6.1.2.3.3 Flare Operational Requirements

The Air Permit also includes the following operational requirements for the flare as an abatement measure (A-15):

- A-15 must be operated at a gas flow rate that does not exceed 353,500 cu-ft per day; and
- A-15 must have a destruction efficiency of at least 90 percent by weight.

4.6.1.2.3.4 Emergency and Portable Engine Operational Requirements

BAAQMD's Regulation 2, Rule 1 (BAAQMD, 2012) indicates that "portable equipment" is defined under state regulations as equipment that does not remain at a single location for greater than 12 consecutive months. The operational requirements applicable to portable equipment at the WWTP are summarized in Table 4-17. Two of the engines designated as "portable" (S-24 and S-28) include an additional requirement beyond those listed in Table 4-17, which is related to their being defined as "portable equipment." The remaining portable pump with listed conditions – S-27 – does not include this condition. For S-24, the applicable hours of operation, as indicated in Table 4-17, depend on whether the engine has retained its portable status. If the engine remains at any fixed location in the San Francisco Bay Area Air Basin for more than 12 months, its status will automatically revert to a conventional permanent location, and its loss of portable status must be reported to the BAAQMD Director of the Compliance and Enforcement Division no later than 30 calendar days after its occurrence.

Table 4-17. Operational Requirements for Emergency and Pump Engines

Engine Description	Source Number	Allowed Uses	Maximum Allowed Hours of Operations
Emergency generator engines			
Standby diesel engine	S-25	Mitigate emergency conditions or emissions testing for compliance purposes	Unlimited
		Reliability-related activities[a]	50 hours per year
Standby diesel engine	S-26	Mitigate emergency conditions or emissions testing for compliance purposes	Unlimited
		Reliability-related activities	20 hours per year
Portable pump engines			
Portable water pump standby diesel engine[b]	S-24	Mitigate emergency conditions or emissions testing for compliance purposes	Unlimited
		Reliability-related activities	Currently, 24 hours per year (at portable offsite locations) If reverted to a permanent source (at the WWTP): • 100 hours per year total • 81 hours per year onsite • 24 hours per year offsite
Portable diesel-fired pump engine	S-27	*Not specified*[b]	• 16 hours per 24-hour period • 1,300 hours for any consecutive 12-month period
Portable standby pump engine[b]	S-28	Mitigate emergency conditions or emissions testing for compliance purposes	Unlimited
		Reliability-related activities and in conformance	50 hours per year

[a] "Reliability-related activities" are indicated in the permit to be maintenance activities and activities to test the engine's ability to perform for an emergency use.

[b] The Air Permit does not restrict the specific use of pump engine S-27 but requires that it be exclusively fueled by CARB diesel fuel. The permit also refers to it as a "portable" pump but does not include

4.6.2 GHG Emission Requirements

GHG emissions are regulated separately from HAP emissions and are not addressed in the District's Air Permit. The following topics related to GHG emissions are addressed below:

- State and Federal Mandatory Reporting
- State Cap-and-Trade Program
- USEPA GHG Tailoring Rule
- BAAQMD Climate Protection Program

4.6.2.1 State and Federal Mandatory Reporting

CARB is responsible for leading the development of regulations to monitor and reduce statewide GHG emissions to address requirements of California's Assembly Bill Number 32 (AB 32), the *California Global Warming Solutions Act of 2006* (California, 2006). AB 32 requires that statewide GHG emissions be reduced to 1990 levels by the year 2020. This legislation is consistent with the California Governor's Executive Order S-3-05 that initially established GHG emission reduction targets (State of California, 2005).

The GHGs regulated under AB 32 are carbon dioxide, methane, nitrous oxide, and fluorinated gases. AB 32 does not affect wastewater treatment process emissions, but it does cover onsite general stationary combustion sources. If a facility's onsite stationary combustion unit emissions (excluding flares) are greater than or equal to 10,000 metric tons of carbon dioxide equivalent (CDE) emissions each calendar year, then the facility is required to report its emissions annually to CARB.

In addition, the USEPA's Mandatory GHG Reporting Rule was adopted October 30, 2009 (USEPA, 2009). The Reporting Rule explicitly states that centralized domestic wastewater treatment facilities are not required to report emissions; however, any stationary combustion of fossil or non-fossil fuels taking place at a wastewater treatment facility may be considered a "large" source of GHGs if it emits a total of 25,000 metric tons or more of CDE per calendar year.

As discussed above, the District's onsite stationary combustion units at the WWTP consist of cogen engine generators, standby diesel generators, backup boilers, and a waste gas flare. The GHG emissions of this equipment are estimated to be about 2,000 metric tons of CDE per year, well below 10,000 metric tons of CDE per year, so all of these units are excluded from CARB's and USEPA's mandatory reporting regulations. In fact, the total rated full design capacity of all combustion equipment is less than the reporting threshold, so even full use of the equipment would not trigger reporting. Therefore, the District is not required to report its GHG emissions from the WWTP to CARB or USEPA at this time.

4.6.2.2 State Cap-and-Trade Program

In addition to requiring mandatory reporting of GHGs, CARB adopted a GHG cap-and-trade program that became effective in January 2012. As part of this program, agencies emitting 25,000 metric tons or more of fossil fuel-based (i.e., natural gas and diesel) CDE per calendar year beginning in 2011 or any subsequent year will be capped and required to reduce their emissions over time.

Because the emissions from the onsite stationary combustion units at the WWTP are approximately 2,000 metric tons of CDE per year, the WWTP is excluded from the regulation at this time, and the District is not at risk of becoming a capped entity. CARB is also requiring annual inventory reporting and verification for agencies emitting down to the 10,000 metric ton CDE level, so GHG programs may be expanded to the 10,000 to 25,000 metric ton CDE level in the future. Even at the lower level, these requirements would not apply to the WWTP.

4.6.2.3 USEPA GHG Tailoring Rule

In 2010, the USEPA adopted the final Tailoring Rule, which sets thresholds for GHG emissions that define when permits under the New Source Review Prevention of Significant Deterioration and Title V Operating Permit programs are required for industrial facilities that have existing Prevention of Significant Deterioration or Title V Operating permits. The rule "tailors" the thresholds from the usual 100 or 250 tons/year thresholds used in the CAA for criteria pollutants to higher thresholds that are more implementable for GHGs.

Defining GHG emission sources at wastewater treatment plants that are covered in this final rule is open to each air district's interpretation (e.g., whether combustion and/or process emissions are included). The BAAQMD is currently only considering stationary combustion unit emission sources but may, in the future, include GHG information from other sources as part of the annual inventory.

As of June 23, 2014, the United States Supreme Court ruled the following:

- USEPA exceeded its authority by interpreting the CAA to require Prevention of Significant Deterioration and Title V permitting for stationary sources based on GHGs, and USEPA plans to rescind the requirement.
- GHG emissions alone cannot bring a facility into Prevention of Significant Deterioration and Title V permitting programs.
- If a facility is already in a permitting program and exceeds GHG thresholds, USEPA can bring GHGs into the permitting program.

USEPA has not yet determined whether it will consider biogenic carbon dioxide as part of the threshold estimates.

Activities at the WWTP currently do not trigger reporting under the Tailoring Rule.

4.6.2.4 BAAQMD Climate Protection Program

As part of its Climate Protection Program, BAAQMD adopted a *10-Point Climate Action Work Program* (Work Program) in April 2014 (BAAQMD, 2014) that establishes direction for the BAAQMD in coordinating implementation of federal, state and local GHG emissions policies. The Work Program establishes a goal to reduce GHG emission in the San Francisco Bay Area to 80 percent below 1990 levels by 2050, to be consistent with the State's Executive Order S-3-05. In addition, BAAQMD commits in the Work Program to "[i]nitial rule development to advance GHG reduction in sources subject to Air District regulatory authority, and identify opportunities to require GHG emission reductions in existing rules and policies." Examples of potential rules include integrating GHG reduction emission into BAAQMD permits consistent with CARB's GHG cap-and-trade program.

The Work Program includes a number of other proposals for the BAAQMD to work with stakeholders to reduce GHG emissions in the San Francisco Bay Area. To date, specific GHG emissions reductions have not been implemented that would impact the WWTP.

4.6.3 Odor Control Requirements

The WWTP operations are subject to a number of requirements limiting offensive odors from leaving the WWTP premises. In general, odor control requirements are established and enforced not at a federal or state level but at a local level. One exception is that CARB has established an ambient air quality standard for hydrogen sulfide of 0.03 ppm for a one-hour average. This standard, however, was established in 1969 and has not been revised since, whereas more restrictive standards are required by the BAAQMD, as discussed below. Since the Regional Board also has odor control requirements in the District's water quality permits, these are also discussed in this section.

4.6.3.1 BAAQMD Odor Control Requirements

The BAAQMD has adopted a number of regulations relevant to odor control. Regulation 1 includes a generally applicable provision (Regulation 1-301) that limits a facility[7] from discharging a volume of air pollutants that may cause nuisance to "any considerable number of persons or the public…" (BAAQMD, 2011). In addition, the rule asserts that three or more violation notices issued to a facility within a 30-day period for public nuisance indicates that violations resulted from negligent conduct.

[7] The BAAQMD regulations discussed herein are written as applying to a person. To avoid confusion, "facility" is substituted in this section for references in these regulations to "person."

The BAAQMD's Regulation 9, which is a set of limits on inorganic pollutants, includes two relevant subparts (rules) focused on potentially odorous compounds: SO_2 and hydrogen sulfide. Regulation 9, Rule 1 (BAAQMD, 1995) limits a facility from emitting a volume of SO_2 that would result in ground level concentrations greater than the following:

- Three-minute average of 0.5 ppm
- One-hour average of 0.25 ppm
- 24-hour average of 0.05 ppm

Regulation 9, Rule 1 also limits SO_2 emissions specifically to 300 ppm (dry).

Regulation 9, Rule 2 (BAAQMD, 1999) limits a facility from emitting a volume of hydrogen sulfide that would result in ground level concentrations greater than the following:

- Three-minute average of 0.06 ppm
- One-hour average of 0.03 ppm (i.e., equal to the CARB standard for hydrogen sulfide)

The BAAQMD's most extensive set of odor control requirements is found in Regulation 7, focused on "odorous substances." The limitations set forth in Regulation 7 do not initially apply to a facility until the BAAQMD receives at least 10 odor complaints within a 90-day period alleging that objectionable odors have been perceived at or beyond the facility's property lines. However, for master planning purposes, it is worthwhile to consider the requirements of Regulation 7 to be in effect. In addition, the District has historically used Regulation 7's "Limit on Odorous Substances at or Beyond Property Line" to guide its odor control planning (DSRSD, 1999).

Regulation 7 specifically limits a facility from discharging "any odorous substance which causes the ambient air at or beyond the property line of such [facility] to be odorous and to remain odorous after dilution with four parts of odor-free air" (BAAQMD, 1982). The District's Odor Control Master Plan thus describes this limit as requiring four "dilutions to threshold" at the fence line (DSRSD, 1999).

Regulation 7 does not require zero odor levels downwind of a facility but rather that odor levels be sufficiently low that most people will not find them objectionable. In practice, the BAAQMD implements this limit by requiring that odor from a sample of ambient air at the public fence line or downwind of a facility, once diluted four times with odor-free air, cannot be detectable by more than half of a trained three-person odor panel.

The Odor Control Master Plan sets a range of thresholds for off-site odors to 0.08 to 0.47 parts per billion (ppb) of hydrogen sulfide equivalents in a three-minute average concentration. In addition, the Odor Control Master Plan includes a target of limiting the frequency of off-site odor exceedances to less than one percent of the year, equal to about 88 hours per year.

An Odor Control Master Plan Update completed in 2004 established that the District had accomplished significant improvements in odor control at the WWTP since 1999 and that the District had met the one-percent goal for hydrogen sulfide (DSRSD, 2004). Odors from ammonia had also been contained within the fence line of the WWTP.

The Air Permit, as noted above, allows operation of the treatment facilities (S-100, S-110, S-120, S-130, S-140, S-150, S-160, S-170, and S-180) and related abatement measures (A-2, A-15, A-100, and A-170) pursuant to these facilities complying with BAAQMD nuisance and odor requirements. Specifically, these facilities must be operated to comply with Regulation 1-301 and Regulation 7.

Air Permit requirements specific to sulfur (i.e., Regulation 9 requirements) are limited to the existing SO_2 limits on Engine S-22 per Table 4-16 and Engine S-24 per the discussion following the table.

4.6.3.2 Regional Board Odor Control Requirements

Odor control requirements in the various Regional Board permits to which the WWTP is subject are detailed below. In general, these require that the District manage wastewater and biosolids at the WWTP in such a way that offensive odors are not reported offsite (i.e., not causing a nuisance). Therefore, meeting the BAAQMD odor control requirements described above should be sufficient to meet the Regional Board's odor control requirements.

Neither the NPDES Permit nor the Wet Weather Permit includes specific requirements to limit odors in the effluent. The NPDES Permit includes only one direct odor requirement, a monitoring requirement found in the Land Retention or Disposal Area Observations in the Regional Standard Provisions (Attachment G of the NPDES Permit). Specifically, Standard Observation C.4.c. requires the District to monitor for the following with respect to odors from on-site surface impoundments:[8]

- Presence or Absence
- Characterization
- Source
- Distance of Travel
- Wind Direction

[8] The required frequency of such observations is not clearly indicated in the NPDES Permit or Attachment G of the NPDES Permit.

The Wet Weather Permit also includes the same Regional Standard Provisions (Attachment G of the Wet Weather Permit) and thus the same requirement to monitor on-site impoundments for odors. In addition, as noted above, the Wet Weather Permit's MRP (Attachment E of the Wet Weather Permit) requires that "standard observations" be conducted once per discharge event in the receiving waters (Alamo Canal and/or San Lorenzo Creek). These receiving water observations are further specified in Standard Observation C.1.c of the Regional Standard Provisions. The required receiving water observations for odor are for the same monitoring parameters as specified for on-site surface impoundments.

As discussed earlier in this chapter, the General Order also includes prohibitions against causing nuisance conditions from treatment, storage, distribution, or reuse of recycled water. Similar to the surface water discharge permits, the General Order also includes a standard observation requirement to monitor for "odors of wastewater origin" from irrigation sites and, if present, to note the apparent source, characterization, and direction of travel.

The Biosolids Permit includes two discharge prohibitions specifically focused on odor (Prohibitions 8 and 9). Prohibition 1 also prohibits creating nuisance conditions. As with the BAAQMD requirements, nuisance conditions include those that cause offensive odors. In fact, the California Water Code defines nuisance conditions as being "anything…offensive to the senses." Prohibition 1 thus limits odors at the DLD site, as well.

Applicable portions of these nuisance and odor control prohibitions from the Biosolids Permit read as follows:

1. *The relocation of wastes to or from any waste management unit shall not create a condition of pollution or nuisance as defined in Section 13050 (1) and (m) of the California Water Code...*

...

8. *Biosolids disposed in the DLD site shall be covered within 24 hours after application or if it poses an odor and/or vector nuisance. Injection of biosolids is considered covered unless improper application results in ponding or spillage during application, in which case the biosolids must be covered if it threatens to cause an odor and/or vector nuisance conditions.*
9. *Discing at the DLD site shall not result in odor and/or vector nuisance conditions.*

Special Provision VI.C.4.b.(2) of the NPDES Permit also requires that the District's treatment, storage and disposal or reuse of biosolids "not create a nuisance, such as objectionable odors or flies…"

The Standard Observation C.4.c mentioned above that requires observations of any odors for facilities with on-site surface impoundments also applies to disposal areas that are in use. The preliminary text for disposal area observation requirements clarifies that these requirements apply to both liquid and solid wastes. Therefore, this observation requirement applies not only to the onsite storage ponds but also to the District's biosolids storage facilities and disposal areas.

4.6.4 Potential Future Air Quality Requirements

Potential future air emission, GHG emission and odor requirements that could impact the WWTP are discussed below.

4.6.4.1 Air Emissions

Expansion and upgrade of equipment at the WWTP that are regulated under the BAAQMD Air Permits would require continued compliance with BAAQMD standards through a permit revision process. In addition, improvements and changes to wastewater processes may require a revised Air Permit.

The primary requirements of the air quality permitting process that could have an impact on strategic master planning decisions are related to the need for cleaning of digester gas. Specifically, digester gas is currently used to fuel Cogen Engine #2, which is a lean burn engine. It is possible that the District would be required to convert Cogen Engine #2 to a rich burn engine in the future, if BAAQMD determines that this approach meets the criteria of a BACT. To operate as a rich-burn engine, a catalytic converter would need to be installed on Cogen Engine #2. (Cogen Engine #1 and #3 are already equipped with catalytic converters.) The primary cost associated with this conversion is associated with digester gas treatment. However, the gas treatment that is likely needed for using digester gas in a rich burn engine is already provided for all of the digester gas in the District's cogen engines. Moreover, continued treatment of digester gas is recommended, regardless of BAAQMD requirements, because the treatment provides direct benefits to the lean burn engine. Therefore, the potential for new BAAQMD requirements to drive decisions under this strategic master planning process are not significant relative to other potential regulatory requirements, such as water quality requirements for continued surface water discharge.

In addition, BAAQMD is in the process of adopting a new rule Regulation 11, Rule 18 (Rule 11-18) for reducing risk from toxic air emissions at existing facilities. The implementation of this rule could impact the WWTP. Rule 11-18 would require a facility to install new or additional air pollution control equipment to lower emissions if the facility has emissions of TACs that are determined by BAAQMD to exceed certain thresholds.

The first step in implementing Rule 11-18 is for the BAAQMD to update POTW prioritization scores, which are based on the volume of TACs emitted, the toxicity of the pollutants, and the proximity of the POTW to potential receptors (people who could be exposed to air pollutants from the POTW source). For facilities with prioritization scores greater than a specific threshold, BAAQMD will then develop updated Health Risk Assessments.

The results of a facility's Health Risk Assessment will determine whether a facility is subject to further requirements under Rule 11-18. Facilities with risks determined to be above risk action levels specified in the rule will then need to prepare a Risk Reduction Plan for approval by BAAQMD. The Risk Reduction Plan must identify sources, quantities, and causes of emissions responsible for exceedance of any risk action levels, and detail measures that will be implemented to reduce risk below those thresholds.

BAAQMD plans to implement Rule 11-18 in four phases. BAAQMD recently issued a letter to BACWA clarifying that all POTWs would be placed in the second phase (BAAQMD, 2017). Health Risk Assessments for facilities in the second phase would be performed in the 2019 to 2021 timeframe. Risk Reduction Plans, if needed, would then need to be developed between 2021 and 2022 and implemented between 2022 and 2025.

4.6.4.2 GHG Emissions

Changes to GHG emission accounting are not necessarily anticipated but, if implemented, could impact GHG regulation at the WWTP. Potential changes to GHG emission accounting include the following:

- Inclusion of nitrous oxide in wastewater treatment operations (nitrous oxide has a relatively large CDE value);
- Inclusion of digester gas flare emissions in GHG accounting, as these are currently excluded; and
- Inclusions of biogenic carbon dioxide emissions, which are currently excluded.

As noted above, CARB may expand the State Cap-and-Trade Program for GHGs to the 10,000 to 25,000 metric ton CDE level in the future. Depending on the future accounting of GHG emissions and/or potential expansion of the cogen system, this potential change could impact the District. However, given that the current GHG emissions are less than 20 percent of the 10,000 metric tons/year level, the expansion of the State Cap-and-Trade Program is unlikely to impact the District unless the wastewater treatment operations come into question.

If the District were required to report GHG emissions, reporting of both direct (onsite) and indirect (offsite) emissions would be required. "Indirect" GHG emissions are those resulting from power used onsite but generated offsite. Reducing indirect GHG emissions could be done by generating more renewable energy onsite, including fats, oils and grease (FOG) or food waste to produce more digester gas. In addition, reducing emissions could be done by implementing energy efficiency projects, such as upgrading the aeration systems to use less air and energy.

4.6.4.3 Odor Requirements

If the District were to have difficulty meeting odor control requirements at the FSL and DLD sites, enhancing odor control at these sites could be prohibitively expensive, thus requiring an alternative site or biosolids treatment process for disposal of the District's biosolids. Evaluation of such alternatives and additional odor control requirements is addressed in later chapters of this master plan.

4.7 REFERENCES

BAAQMD, 1982, *Regulation 7: Odorous Substances*, Amended March 17, 1982.

BAAQMD, 1995, *Regulation 9: Inorganic Gaseous Pollutants, Rule 1: Sulfur Dioxide*, Amended March 15, 1995.

BAAQMD, 1999, *Regulation 9: Inorganic Gaseous Pollutants, Rule 2: Hydrogen Sulfide*, Amended October 6, 1999.

BAAQMD, 2007a, *Regulation 6, Particulate Matter, Rule 1: General Requirements*, Amended December 5, 2007.

BAAQMD, 2007b, *Regulation 9, Inorganic Gaseous Pollutants, Rule 8: Nitrogen Oxides and Carbon Monoxide from Stationary Internal Combustion Engines*, Amended July 25, 2007.

BAAQMD, 2011, *Regulation 1: General Provisions and Definitions*, Amended May 4, 2011.

BAAQMD, 2012, *Regulation 2: Permits, Rule 1: General Requirements*, Amended December 19, 2012.

BAAQMD, 2017, *Permit to Operate, Permit A1371, Plant #1371*, June 15, 2017.

BAAQMD, 2017, *Letter to David R. Williams, BACWA Executive Director, Re: Summary of the BACWA-BAAQMD Rule 11-18 Meeting*, May 19, 2017.

California (State), Assembly, 2006, *Assembly Bill No. 32, Chapter 488, California Global Warming Solutions Act of 2006*, AB 32, Approved by the California Governor on September 27, 2006.

CARB, 2010, *Final Regulation Order, Amendments to the Airborne Toxic Control Measure for Stationary Compression Ignition Engines*, Adopted October 21, 2010.

DSRSD, 1999, *Dublin San Ramon Services District Odor Control Master Plan*, Prepared by CH2MHill in association with Whitley, Burchett & Associates, Inc., Environmental Management Consulting, and CES Environmental Consultant, June 1999.

DSRSD, 2004, *Dublin San Ramon Services District Odor Control Master Plan Update*, Prepared by CH2MHill, July 2004.

DSRSD, 2011, *2010 Urban Water Management Plan*, June 2011.

DSRSD, 2013, *Dublin San Ramon Services District Dedicated Land Disposal Facility, Annual/Semi-Annual Monitoring and Maintenance Report, Year 2012*, March 2013.

LAVWMA, 1997, *Amended and Restated Joint Exercise of Powers Agreement for the Livermore-Amador Valley Water Management Agency (LAVWMA)*, Prepared on June 30, 1997, Approved September 1997.

LAVWMA, 2010, *LAVWMA Report of Waste Discharge for Renewal of NPDES Permit No. CA0038679 Mixing Zone Analysis for Wet Weather Discharges*, December 8, 2010.

LAVWMA, 2012, *Livermore Amador Valley Water Management Agency Wet Weather Facilities Operations Plan*, Updated November 2012.

LAVWMA, 2014, *LAVWMA Export and Storage Facilities Annual Technical Report*, Prepared by Dan Gallagher, June 2014.

Luhdorff & Scalmanini Consulting Engineers, 2007, *Letter to Mr. Terry Seward, San Francisco Bay Regional Water Quality Control Board, Subject: Report of Well Installation and Abandonment Dublin San Ramon Services District Dedicated Land Disposal Site, Pleasanton, Alameda County, CA, File No. 07-1-067*, November 28, 2007.

San Francisco Estuary Institute, 2014, *RMP Update 2014*.

San Francisco Regional Water Quality Control Board, 1996, *Order 96-011, General Water Reuse Requirements for: Municipal Wastewater and Water Agencies*, Adopted January 17, 1996.

San Francisco Regional Water Quality Control Board, 2007, *Order No. R2-2007-0053, Waste Discharge Requirements, Dublin San Ramon Services District, Dedicated Land Disposal Site, Class II Land Treatment Unit, Pleasanton, Alameda County*, Adopted August 8, 2007.

San Francisco Regional Water Quality Control Board, 2012, *Order No. R2-2012-0096, NPDES No. CA0038849, Waste Discharge Requirements for Mercury and PCBs from Municipal and Industrial Wastewater Discharges to San Francisco Bay*, Adopted December 12, 2012.

San Francisco Regional Water Quality Control Board, 2014, *Order No. R2-2014-0014, NPDES No. CA0038873, Waste Discharge Requirements for Nutrients from Municipal Wastewater Discharge to San Francisco Bay*, Adopted April 9, 2014.

San Francisco Regional Water Quality Control Board, 2016, *Order No. R2-2016-0015, NPDES No. CA0038679*, Adopted April 13, 2016.[9]

San Francisco Regional Water Quality Control Board, 2017a, *Order No. R2-2017-0017, NPDES No. CA0037613*; Waste discharge requirements for DSRSD, LAVWMA, and EBDA; Adopted May 10, 2017.[10]

San Francisco Regional Water Quality Control Board, 2017b, *San Francisco Bay Basin (Region 2) Water Quality Control Plan (Basin Plan)*, With amendments approved by the State of California's Office of Administrative Law as of May 4, 2017.

State of California, Executive Department, 2005, Executive Order S-3-05 by the Governor of the State of California, Filed on June 1, 2005.

State Water Board, 2005, *Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California*. Resolution No. 2005-0019, February 2005.

State Water Board, 2016, *Order WQ 2016-0068-DDW, Water Reclamation Requirements for Recycled Water Use*, June 7, 2016.

State Water Board, 2017, *Draft Part 3 of the Water Quality Control Plan for Inland Surface Waters, Enclosed Bays, and Estuaries of California – Bacteria Provisions and a Water Quality Standards Variance Policy*, June 30, 2017.

USEPA, 2000, *40 CFR Part 131 Water Quality Standards; Establishment of Numeric Criteria for Priority Toxic Pollutants for the State of California; Rule*, Federal Register, Vol. 65, No. 97, May 18, 2000.

USEPA, 2009, *Code of Federal Regulations Title 40, Chapter I, Subchapter C, Part 98 – Mandatory Greenhouse Gas Reporting*, Published on October 30, 2009.

USEPA, 2012, Letter from John Kemmerer, Acting Director, EPA Water Division to Daniel P. Gallagher, DSRSD Operations Manager, *Re: 40 CFR 503.24(k) Request to Harvest Feed Crops from Sewage Sludge Surface Disposal Site*, December 20, 2012.

[9] This document has no title beyond the order and NPDES numbers. As detailed in the text of the chapter, this Regional Board order is the wet weather discharge permit issued jointly to LAVWMA, DSRSD, and the Cities of Pleasanton and Livermore.

[10] This document has no title beyond the order and NPDES numbers. As detailed in the text of the chapter, this Regional Board order is the NPDES Permit for surface water discharge, issued jointly to DSRSD, LAVWMA, and EBDA.

CHAPTER 5
Evaluation Criteria

Both life cycle costs and subjective criteria are considered in this WWTP Master Plan when comparing alternatives and defining the overall recommended program. In general, direct costs are included in the life cycle cost and disregarded when considering the subjective criteria. Conversely, indirect costs are typically considered under the subjective criteria, unless they are well known and could have a substantive impact on the life cycle cost comparison. If identified indirect costs are thought to materially affect the life cycle cost and are reasonably quantifiable, those costs are included in the cost analysis rather than the subjective evaluation.

The specific methods of application and assumptions used when assessing the life cycle costs and subjective criteria are described in this Chapter.

5.1 LIFE CYCLE COSTS

This section presents an overview of the major assumptions applied when developing the life cycle costs for this WWTP Master Plan. The life cycle cost of each project element is defined based on the total capital cost estimated in current dollars (which include construction costs and other project-related capital costs such as design and project administration)[1] and the net present value (NPV) of long-term operation and maintenance (O&M) costs. The NPV can be thought of as a dollar amount that would be invested today to generate the long-term cash flow needed to execute the given alternative, taking into account interest earnings and inflation. The sum of the capital cost and the NPV of the annual O&M costs is referred to herein as the "life cycle cost."

5.1.1 Construction Costs

The assumptions and methods used to prepare the construction cost estimates developed for this WWTP Master Plan are discussed in this section. The topics addressed include:

- Construction Cost Estimate Basis
- Base Construction Cost
- Estimating Contingency
- Other Contractor Costs and Profit
- Construction Contingency

[1] Future replacement and salvage value are not considered for the purposes of this WWTP Master Plan, as the major facilities have an expected life significantly greater than the length of the cost analysis period.

5.1.1.1 Construction Cost Estimate Basis

The cost estimates presented are considered Class 4 estimates, as defined by the AACE International[2] (formerly the Association for the Advancement of Cost Engineering). Cost estimates at this level of planning are necessarily preliminary in nature. The primary purpose of these estimate is to provide the District with a basis for comparing alternatives. The aggregate cost of a particular course of action may also be used for long-range budget planning, with appropriate consideration for the potential variability in project scope, economic factors, and the ongoing evolving construction materials and techniques. Preliminary design and detailed design efforts will be necessary to refine and confirm the estimates presented herein.

With the exception of the construction contingency, the construction costs presented in this WWTP Master Plan represent an engineer's opinion of probable construction cost (OPCC). The total of the component costs, distributed costs, estimating contingency, and other contractor costs and profit was calculated as the engineer's preliminary OPCC. The OPCC plus the construction contingency represent an estimated construction budget. The total capital cost is then calculated as the sum of the engineer's preliminary OPCC, the construction contingency, and other project- related capital costs.

All of the construction costs are estimated in terms of current (2016) dollars.

5.1.1.2 Base Construction Cost

Costs for individual facility components were estimated using a variety of sources. In some cases, rough quantities of materials were estimated, while in others lump sums based on similar project were used.

Distributed costs account for costs that are not included in the component costs at this level of planning. Distributed costs include: Plant paving, grading, and yard piping; miscellaneous mechanical and piping; electrical; and instrumentation and control. Typically, the cost of major components was estimated, and then multipliers were used to account for the related electrical, yard piping, and other distributed costs.

5.1.1.3 Estimating Contingency

An estimating contingency allowance was applied to the sum of the base construction cost and distributed costs to account for cost items that are not identified in the conceptual description of a given alternative. For purposes of this master plan, an allowance of 30 percent was used as estimating contingency, which is typical for planning level cost estimating.

[2] AACE International Recommended Practice No. 18R-97, "Cost Estimate Classification System – as Applied in Engineering, Procurement, and Construction for the Process Industries," March 1, 2016.

5.1.1.4 Other Contractor Costs and Profit

Other contractor costs include taxes on materials and equipment, mark-up on subcontractors, bonds, insurance, mobilization, demobilization, and general overhead. These cost factors, as well as contractor profit are calculated as a multiple of the total base and distributed costs with the estimating contingency, with the exception of the markup on subcontractor costs. The multipliers used are listed in Table 5-1. The amounts for some items, such as the portion subject to subcontractor markup, mobilization or demobilization will vary depending on the nature of the work and the particular contractor selected; nevertheless, the factors provide a reasonable estimate and are useful when used in conjunction with appropriate contingencies.

Table 5-1. Multipliers for Other Contractor Costs and Profit

Item	Multiplier
Tax on Materials[(a)]	9.5%
Contractor's Markup of Subcontractors' Work[(b)]	10%
Mobilization and Demobilization	5%
Contractor's Overhead and Profit	20%
Contractor's General Conditions (Bonds and Insurance, other requirements)	10%

(a) Applied to 50% of the OPCC less the Contractor's markup on Sub-Contractors' Work, Contractor's Overhead and Profit, Mob/Demob, Insurance, Bonds, etc., and Contractor' General Conditions.
(b) Applied to electrical, instrumentation and controls distributed costs only.

5.1.1.5 Construction Contingency

A construction contingency allowance is applied to account for increased costs that may arise during construction due to conditions unforeseen at the time of bidding. A construction contingency of 10 percent of the engineer's preliminary OPCC was used.

5.1.2 Other Project-Related Capital Costs

The District has internal guidelines used in establishing appropriate budget amounts for future capital projects. The non-construction costs (those costs not included in the construction contract and construction contingency), are predicted as a percentage of the estimated construction costs. In general, some of the percentages should be slightly lower for large projects (perhaps totaling 35 percent) as compared to small projects (perhaps totaling 50 percent, or more).

Table 5-2 lists the "rule-of-thumb" values from the guidelines, as well as the values used for this WWTP Master Plan. It is assumed that most or all improvements described in this WWTP Master Plan will be constructed as part of large projects. As shown, an overall multiplier of 35 percent was applied to the engineer's preliminary OPCC to capture the non-construction costs.

Table 5-2. Multipliers for Other Projects Costs (% of Construction Costs)		
Cost Category	District's Rule of Thumb	Master Plan Value[(a)]
Planning	3%	2%
Design	13%	13%
Construction Management	11%	9%
Management & Administration	5%	4%
District Staff Time	8%	7%
Total	**40%**	**35%**

[(a)] Appropriate for elements that are to be included in a large project.

5.1.3 O&M Costs

O&M costs were developed for each alternative and were broken down into labor costs, power costs, chemical costs, replacement costs, and solids hauling and landfill disposal costs. These components of O&M costs were developed as follows:

- **Labor Costs:** Labor costs included annual O&M labor hour requirements for each process and general O&M needed for each alternative. Estimates are based on project team experience and information provided by equipment vendors. The assumed average hourly labor rate was $150. This is a fully loaded rate with salary, benefits, direct overhead, and administrative overhead costs. It represents a rounded average value, and accounts for some hours at overtime rates, as well normal "non-productive" time (e.g., leave time such as holiday, sick, and vacation)[3].
- **Power Costs:** Power costs were determined according to the power demands of duty equipment, estimated annual operating hours, and a power cost of $0.15 per kilowatt hour (kWh) from Pacific Gas & Electric Company. Power costs offset by self-generated power are considered on a case-by-case basis (as appropriate) within the alternatives.
- **Chemical Costs:** Chemical costs were determined according to estimated doses for treatment and chemicals such as polymers and sodium hypochlorite, including process cleaning/renewal operations based on the manufacturer recommendations. Unit chemical costs were provided from sources including the Bay Area Chemical Consortia 2017 rates, GE Power, project team experience and the experience with similar projects.
- **Replacement Costs:** Additional allowances for replacement of major equipment were included where major replacement costs would be expected within the time frame of the analysis.

[3] Average value derived from data provided by District staff; email from Jaclyn Yee, March 22, 2016.

- **Solids Hauling and Disposal Costs:** Several of the alternatives presented in this WWTP Master Plan include costs associated with solids hauling and disposal. These costs are assumed to be $50/wet ton (including both hauling and disposal) based on West Yost experience with other recent Bay Area projects. Hauling costs were assumed to be the same throughout the year. If hauling were to occur to an agricultural land application site, costs may vary depending on the time of year.

All of the applied O&M cost assumptions are summarized in Table 5-3.

Table 5-3. Assumed Unit Costs for Developing Annual Operating Cost

Item	Unit Cost	Unit Basis
Chemical Costs		
Citric Acid	$0.56	per pound
Hydrogen Peroxide	$1.00	per pound
Liquid Alum	$220	per ton
Liquid Oxygen (LOX)	$0.30	per 100 cu-ft
Methanol	$1.8	per gallon
MBR Cleaning Chemicals	$12,750	per MGD per year
Polymer for Solids Dewatering	$7	per active pound
Polymer for Actiflow Process (Winter-Season Phosphorous Removal)	$2.25	per pound
Sulfuric Acid	$5.79	per gallon
Sodium Bisulfite	$0.90	per gallon
Sodium Dodecyl Sulfate	$233	per pound
Sodium EDTA	$61	per pound
Sodium Hydroxide (50% solution)	$1.00	per gallon
Sodium Hypochlorite	$0.40	per gallon
Sodium Triphosphate	$41.95	per pound
Non-Chemical Costs		
Electrical Power	$0.15	per kWh
Labor (including benefits)	$150	per hour
Solids Hauling	$50	per wet ton
MF Membranes	$120	per module per year
RO Membranes	$350	per module per year
UV Lamps	$80	per lamp per year
Granular Activated Carbon (GAC) Media	$2.37	per pound
GAC Media Replacement Cleaning (3 MGD System)	$2,600	per replacement cycle
GAC Media Replacement Cleaning (6 MGD System)	$3,400	per replacement cycle

5.1.4 Present-Worth Analysis

As noted previously, the operating cost evaluation of alternatives is expressed as a NPV, assuming the change in operating cost occurs in the same year the facility is constructed. The NPV of annual operating costs for a given period (n) is estimated based on the present value discount rate (r), which is the amount of interest an investment would earn based on US Treasury Bond rates, and the annual operating costs (A) using the following equation:

$$NPV = \sum_{1}^{n} A \cdot \left(\frac{1}{1+r}\right)^{n}$$

where:

Analysis Period (n): 30 years

Annual Discount Rate (r):[4] 2.68 percent

It should be noted that the NPV of a future capital cost will be lower than the estimated future cost due to the time value of money (i.e., the future cost must be discounted when expressed as a present value). To be clear, capital costs used in this report to compare alternatives are not discounted. The capital costs may still be compared across alternatives on this basis as long as the expenditure will occur in the same (or nearly the same) year for each alternative.

5.2 SUBJECTIVE CRITERIA

This section presents an overview of the non-monetary subjective criteria that are used to compare alternative approaches in this WWTP Master Plan. These subjective criteria were developed in conjunction with District staff and reflect the District's values relative to environmental and social considerations that do not readily translate into a dollar amount. The topics addressed in this section are as follows:

- Subjective Criteria Overview
- Criteria Ranking and Weighting Factors
- Subjective Criteria Summary

[4] Discount rate developed in consultation with District staff, email from Karen Vaden via Judy Zavadil, September 19, 2016.

5.2.1 Subjective Criteria Overview

This section presents an overview of the subjective criteria used in this WWTP Master Plan, which are as follows:

- Reliability
- Flexibility
- Odor Generation Potential
- Maximization of Available Water Storage Volume
- Energy Management
- Chemical Use
- Resource Recovery
- Potential for Ancillary Impacts

5.2.1.1 Reliability

At the core of the District's mission is the need to reliably treat wastewater. Process reliability is considered in three ways:

- Technology Status
- Ease of Operation
- Potential for Construction Impacts

5.2.1.1.1 Technology Status

Processes or treatment trains that incorporate unproven technologies threaten treatment reliability. In comparing alternatives, consideration of relatively new technologies may be justified. Some technologies may be in operation at a few sites and carry enough advantages to be worth further consideration. Nevertheless, more mature technologies are preferred over less proven technologies.

5.2.1.1.2 Ease of Operation

Processes or treatment trains that are difficult to control or that incorporate unproven technologies threaten treatment reliability. Process complexity increases with the number of parameters that must be monitored and controlled to keep the process stable. The increased cost of operating a complex system is included in the life cycle cost analysis. However, complex processes have more potential points of failure, in part due to the number of instruments that must be maintained and functioning properly. Operator and maintenance training is also more extensive, and opportunities for human error increase with increasing complexity. Therefore, in general, simpler, more easily operated alternatives are favored for their improved reliability.

5.2.1.1.3 Potential for Construction Impacts

While the construction period is short compared to the life of the constructed facility, disruptions to ongoing operations due to construction activities can present major risks to process reliability. The extent of potential disruptions for a particular phase of work are considered, as well as the number of potentially disruptive phases over time. Alternatives that have greater "constructability" (ease and simplicity of construction) and that minimize modifications to existing facilities are advantageous. Relative construction costs (generally lower for less disruptive improvements) are evaluated in the life cycle cost analysis.

5.2.1.2 Flexibility

Flexibility is valuable when details about future regulatory constraints or technological advances cannot be predicted with great confidence. Flexibility is considered in three ways:

- Flexibility to incorporate future technological innovations as they are developed and proven reliable
- Flexibility to adapt to future regulatory requirements
- Flexibility for future space planning

5.2.1.2.1 Technological Innovations

New technologies in wastewater treatment, resource recovery and energy management are continuously being developed. Some of these new technologies will eventually be proven reliable enough to be incorporated into the WWTP. Some examples of reasons why a future proven technological development might be considered are reduced energy demands or increased energy production, decreased operating costs, increased safety, and reduced environmental impacts. It is not possible to know which technologies will eventually be desirable, nor where they would be located on the Plant site or how much space they would need. However, it is possible to some degree to identify which alternatives might be more capable of accommodating future innovations. In addition, alternatives that defer investment in facilities to a later date and allow time for new technologies to mature are more accommodating to innovation.

5.2.1.2.2 Regulatory Requirements

As current regulatory activities, such as the Bay Area nutrient regulations resolve, it will likely be necessary to implement changes to portions of the Plant. Alternatives that allow for deferred investment, or that are easily adaptable to different requirements are favored over those that would require major modifications to meet changing discharge limitations (for example).

5.2.1.2.3 Space Planning

Space planning is a key element of the master planning process. Preserving unoccupied physical space helps to maintain flexibility to add processes and ancillary facilities not currently anticipated.

5.2.1.3 Odor Generation Potential

Some treatment processes have a greater potential to generate odors. Designs (and the estimated capacity costs included in the life cycle cost analysis) will incorporate features to control odors. Nevertheless, the presence of odor generating processes increases the risk that odors could be detected beyond the Plant boundaries.

5.2.1.4 Maximization of Available Water Storage Volume

Water storage volume is largely a space planning issue. Yet water storage has multiple benefits that are interrelated to other planning decisions. For example, storage volume available for peak flow equalization reduces the hydraulic capacity that must be built into downstream treatment processes, and increases process reliability. In addition, storage is a critical component with respect to controlling discharges to the LAVWMA/EBDA facilities. Finally, storage also helps protect the environment from unintended discharges, and can decrease facility sizing and operating costs for recycled water operations. Therefore, a separate criterion is used to reflect the quantity of storage volume incorporated into an alternative. Various types of storage may have different value, but the criterion generally applies to all types of storage.

5.2.1.5 Energy Management

Wastewater treatment and reuse requires energy. Raw wastewater contains energy, some of which can be captured and used to generate electricity and heat. Released energy can be used in the treatment process itself and for powering the ancillary facilities, such as the administration and maintenance buildings. Currently, the District uses the majority of biogas generated at the WWTP to generate energy onsite.

The alternatives evaluated in this WWTP Master Plan have varying levels of energy demands, depending on the energy needs and efficiency of particular processes under consideration. Energy costs are included in the life cycle cost analysis. The energy management subjective criterion is related to whether the alternative provides benefits with respect to the potential for capturing wastewater energy, or provides a potential to use available space or facilities to expand energy generating capacity at the site.

5.2.1.6 Chemical Use

Chemical use, whether it be an increase in the quantity of a chemical already in use at the Plant, or introduction of a process that requires use of a new chemical, results in increased costs for purchasing, storing, and handling the chemical. These costs are included in the life cycle cost analysis. However, there are a number of indirect impacts from chemical use at the Plant. Examples of indirect impacts of chemical use include costs associated with procurement, periodic safety training, risks associated with hauling the chemical, work place hazards, additional record keeping and reporting, and environmental impacts associated with offsite manufacturing and transport of the chemical. Price certainty is of value in planning the District's operating budget. Chemical costs are subject to fluctuation over time, so additional chemical use increases the uncertainty in predicting future operating costs.

5.2.1.7 Resource Recovery

Resource recovery focuses on the opportunities to "harvest" the resources associated with wastewater. Resources with viable recovery opportunities include water, nitrogen, phosphorous, and biosolids (a treatment residual).

5.2.1.7.1 Water

Water is the most obvious resource entering the WWTP. The District has already invested in facilities to highly treat the water so that it may be reused. Moreover, the value of, and public interest in, water recycling has increased in recent years. Some technologies are more efficient than others from a water conservation standpoint, and in terms of how much water is "rejected" from the process along with the contaminants being removed (the reject stream). Processes that capture more treated water for potential reuse and delivery to water recycling customers are preferred.

5.2.1.7.2 Nitrogen

Nitrogen is a nutrient that can be beneficially reused in some forms. It is also considered a contaminant in water released to streams and estuaries because it promotes an imbalance in the types of organisms that grow in the affected environment. At present, the only nitrogen from wastewater that is commonly available for beneficial reuse is nitrogen bound up in biosolids. In the future, it may be possible and beneficial to capture and reuse nitrogen separately. Alternatives that provide for nitrogen recover, either now or in the future, are preferred.

5.2.1.7.3 Phosphorous

Phosphorous, like nitrogen, is a nutrient presented in wastewater that can be beneficially reused. It is also considered a contaminant in water released to streams and estuaries because it promotes excessive algae growth. In addition, unlike nitrogen, the worldwide supply of phosphorous is limited, and future shortages could have severe impacts on global food supplies.[5] Technologies exist to separate relatively pure phosphorous from waste streams that have a high enough concentration. However, the potential sources of phosphorous from existing DSRSD facilities are not concentrated enough to make phosphorous removal practical. In addition, the technologies available for harvesting phosphorus are not currently considered economically viable for facilities that are the size of the WWTP. However, as phosphorus harvesting technologies advance, more opportunities for recovering this resource will become available. Alternatives that would help facilitate future phosphorus harvesting are thus preferred.

[5] Nature, 2009, Environment: The Disappearing Nutrient, Vol. 461, October 7, 2009. http://www.nature.com/news/2009/091007/full/461716a.html

5.2.1.7.4 Biosolids

Biosolids are a byproduct of wastewater treatment. There is increasing recognition of biosolids as a resource. The value of biosolids as a resource is affected by how they are treated and the amount of water that remains in the biosolids after treatment. Their value lies in the nutrients they contain (among them nitrogen and phosphorous), their usefulness in improving poor soils, and potentially in their energy content. The monetary value of the energy content and use in agricultural is highly variable and will likely change over time based on market conditions; therefore, this value is included in the subjective evaluation rather than the life cycle cost analysis.

5.2.1.8 Potential for Ancillary Impacts

Ancillary impacts include a variety of social and environmental considerations, such as:

- Offsite vehicular traffic
- GHG production/other air quality impacts
- Risk of groundwater impacts
- Aesthetics/public perception
- Overall carbon footprint

Each alternative may have one or more benefits or constraints related to these considerations. Alternatives that have multiple ancillary benefits would rate higher than those with none or few. Similarly, alternatives that have multiple ancillary impacts would rate lower than those with none or few.

5.2.2 Criteria Ranking and Weighting Factors

When comparing alternatives, each subjective criterion is assigned a rating on a scale of 1 to 5, where a ranking of "5" is considered most favorable, and a ranking of "1" is least favorable. A ranking of "3" indicates the alternative is neutral with respect to the criterion being considered. If a criterion is determined to be inapplicable for a given comparison, it may be ignored, with a value of "0" assigned to all alternatives.

Certain criteria are weighted more heavily than others. The weighting determination was made in collaboration with District staff. Table 5-4 lists each of the subjective criteria along with the applied weighting. The weighting factors are based on the considerations noted in the table.

Table 5-4. Subjective Rating Criteria and Weighting

Criterion	Weighting Factor	Weighting Considerations
Reliability	2.0	Process reliability is of upmost importance and has the highest weighting factor.
Technology Status	2.0	Basing a planning decision on the assumption that an unproven technology will mature adequately introduces considerable risk.
Ease of Operation	3.0	Complexity increases risk of operational problems and introduces the risk that operating and maintenance costs will be higher than expected.
Potential for Construction Impacts	1.0	While the risks related to construction impacts are important and significant, the duration of the risk is relatively short.
Flexibility	1.75	The three measures of flexibility are similar in importance, and flexibility in general is highly weighted given there are three related criteria, with a combined weighting of 5.25.
Technological Innovations	1.75	
Regulatory Requirements	1.75	
Space Planning	1.75	
Odor Generation Potential	1.5	Risks anticipated to be largely mitigated through design, but the consideration remains important in comparing alternatives.
Maximization of Available Water Storage Volume	1.25	Of normal importance, slightly higher weighting than energy management and chemical use due to the multiple benefits of storage capacity.
Energy Management	1.0	Of normal importance
Chemical Use	1.0	Of normal importance
Resource Recovery	0.75	In combination, the resource recovery criteria have a relatively high weighting of 3.0, reflecting the District's values.
Water	0.75	
Nitrogen	0.75	
Phosphorous	0.75	
Biosolids	0.75	
Potential for Ancillary Impacts	0.5	Potentially significant ancillary impacts are typically identified and mitigated in design, and are difficult to reliably identify at this level of planning, so the weighting is low.

An overall subjective ranking for each alternative in this WWTP Master Plan was determined as follows:

1. A ranking is assigned to each criterion or sub-criterion.
2. The ranking is multiplied by the weighting factor to calculate a weighted ranking.
3. For the criterion that have multiple sub-criterion factors (e.g., reliability), a ranking is applied to each sub-criterion and is weighted accordingly, then the average of the weighted rankings is calculated to reflect the overall ranking.
4. The sum of the weighted values calculated for each criterion is compared for the various alternatives under consideration.

5.2.3 Subjective Criteria Summary

Table 5-5 provides a summary of the subjective criteria and their weighting. As shown, some criteria are listed with four categories shown to establish the rating value (from lowest to highest). For the criterion that only indicate two areas (high or low), the rating is established based on the relative magnitude of the benefit or impact.

Table 5-5. Subjective Criteria Summary

Criteria	Weighting	Lowest Rating	Low Rating	High Rating	Highest Rating
Reliability	2.0				
Technology Status	2.0	Incorporates new technology with only a very few operating installations.	Incorporates new technology with a relatively small number of operating installations.	Relies on established technologies.	Relies on established technologies with a long history of reliable operation.
Ease of Operation	3.0	Incorporates numerous control and monitoring points and complex procedures.		Minimizes complexity.	
Potential for Construction Impacts	1.0	Involves complex modifications to operating process and/or numerous disruptive construction phases.	Involves some modifications to operating process.	Minimizes: • Intentional disruption of existing processes. • Risk of unintentional impacts on existing processes. • Potentially disruptive construction phases.	Avoids all disruption of existing processes.
Flexibility	1.75				
Technological Innovations	1.75	• Provides little or no opportunity to incorporate future technological developments. • Involves immediate or near-term commitment to a particular process before anticipated technological improvements can mature.		• Allows for incorporation of new advanced technologies in the future. • Maximizes likelihood that future technological innovations can be incorporated to result in lower capital and/or operating costs. • Defers capital investment to allow time for emerging technologies to mature.	
Regulatory Requirements	1.75	Limits opportunity to insert future unknown processes or modify the liquid and/or solids treatment processes to address new requirements.		Maximizes opportunity to insert future unknown processes or modify the liquid and/or solids treatment processes to address new requirements.	
Space Planning	1.75	Immediately consumes unoccupied or lightly used space within the existing plant boundaries.		Preserves land area within and adjacent to existing facilities to increase capacity, allow for future innovation, and allow for taking advantage of future resource recovery opportunities.	
Odor Generation Potential	1.5	Creates a new potential odor source that may be difficult to completely contain.		Avoids creating a new potential odor source.	Eliminates existing odor source(s).
Maximization of Available Water Storage Volume	1.25	Reduces existing storage by 25 percent or greater	Reduced existing storage volume by some amount	Provides an opportunity to expand storage	Provides an opportunity to significantly expand storage
Energy Management	1.0	Reduces the amount of energy generation capacity at the WWTP		Provides future opportunities for onsite generation of electricity.	Maximizes production of biogas and/or heat capture for onsite generation of electricity and use of heat.
Chemical Use	1.0	Results in increased use of chemicals.		Minimizes use of chemicals.	
Resource Recovery	0.75				
Water	0.75	Results in a greater reject stream and lower quantity, and/or lower quality of recycled water.		Minimizes reject stream while maximizing the quantity, and/or maximizes the quality of recycled water.	
Nitrogen	0.75	Reduces or eliminates the future potential capture of nitrogen in usable forms		Maximizes *opportunity* to capture nitrogen compounds in forms that can be beneficially reused.	
Phosphorus	0.75	Reduces or eliminates the future potential capture of phosphorous in useable forms.		Maximizes *opportunity* to capture phosphorous compounds in forms that can be beneficially reused.	
Biosolids	0.75	Does not allow for, or eliminates the possibility of, reuse of biosolids beneficially.		Provides some biosolids of a quality and in a form that can readily be put to beneficial reuse.	All biosolids of a quality and in a form that can readily be put to beneficial reuse.
Potential for Ancillary Impacts	0.5	Produces multiple increased negative ancillary impacts.		Minimizes or eliminates negative ancillary impacts.	

CHAPTER 6
Liquid Treatment Process Alternatives Evaluation

This Chapter presents an evaluation of the facilities required to meet the long-term liquid treatment process and conveyance needs for the WWTP. Facilities that may be necessary in the future to address emerging nutrient removal regulatory requirements are also identified. To support the identification and development of these potential future facility needs, a series of Visioning Panel Workshops were conducted to gather input and direction from external experts regarding likelihood of various future opportunities and constraints. Meeting notes from these workshops are provided in Appendix E.

The topics addressed in this Chapter include:

- Evaluation Methodology
- Hydraulic Capacity Related Improvements
- Primary Effluent Diversion Improvements
- Primary Treatment Expansion
- Alternatives for Addressing Secondary Clarifier Capacity Limitations
- Potential Future Nutrient Removal Improvements

6.1 EVALUATION METHODOLOGY

This section presents an overview of the approach applied when evaluating the sizing and capacity of the required liquid treatment process alternatives under this Master Plan. The following specific topics are addressed:

- Evaluation Objectives
- Capacity Requirements
- Process Sizing Approach

6.1.1 Evaluation Objectives

The primary objectives of this liquid treatment process evaluation are as follows:

- **Identify Improvements Needed to Address Identified Capacity Deficiencies**[1]: The following liquid process capacity-related improvements were identified in Chapter 3: a) remove a hydraulic bottleneck between the secondary clarifiers and the chlorine contact basin, b) expand the primary treatment capacity, and c) address the secondary clarification capacity constraints. This evaluation identifies the approximate cost, size, location, and layout of facility improvements needed to address each of these recommended improvements.

[1] As noted previously in this Master Plan, the facility needs for the recycled water treatment facilities are being evaluated under a separate effort. Therefore, the recycled water treatment facilities are only discussed herein to the extent that they may be a necessary treatment step under a future surface water discharge scenario.

- **Identify Treatment Facilities Potentially Needed to Meet Future Nutrient Removal Requirements:** As discussed in Chapter 4 (Regulatory Requirements), the Regional Board is working to identify appropriate nutrient treatment goals for facilities that discharge to the San Francisco Bay. This evaluation identifies the approximate cost, size, location, and layout of facility improvements that may be needed to meet potential long-term nutrient removal requirements.

6.1.2 Capacity Requirements

This Mater Plan considers the full range of anticipated influent flow and load conditions, and the effects associated with a treatment unit out of service for maintenance. However, as detailed in Chapter 3 (Existing Facilities), the WWTP has three basins (Holding Basins 1, 2 and 3) that can be used for equalization of primary effluent in the event that the capacity of a downstream process (including the secondary clarifiers) is exceeded. These equalization basins have been used in the past when District staff have been concerned about compromised secondary clarifier performance either due to poor settleablitiy of the secondary process mixed liquor or when flows exceed the downstream hydraulic capacity. Because these equalization basins are available, it would be overly conservative to assume that the treatment processes downstream of the primary sedimentation basins would be designed to accommodate the full range of the anticipated operating conditions with the largest unit out of service.

District staff directed that the Holding Basins continue to be used for primary effluent equalization to the extent that this operation occurs infrequently, and allowance of this operation provides a significant financial benefit in terms of overall facility needs. For example, if equalization can provide the same level of capacity-related reliability as a facility expansion (representing a relative high cost), then equalization should be relied on for this purpose. The following recommended approach was applied in developing the appropriate sizing for the liquid process alternatives:

- The facilities should generally be capable of hydraulically conveying the PHWWF identified in this Master Plan (i.e., 46.7 mgd) from the headworks to the final effluent pump station. However, using a limited amount of equalization would be acceptable downstream of the primary sedimentation basins, if necessary.
- The headworks and primary treatment facilities should provide adequate total capacity to treat the predicted 2035 PHWWF for all anticipated operating conditions.
- The treatment facilities downstream of the primary sedimentation basins should have adequate firm capacity to treat at least the predicted 2035 PHDWF of 23.4 mgd for all anticipated operating conditions.
- Equalization should generally be limited to periods when performance of the treatment facilities downstream of the primary sedimentation basins (and equalization storage) are compromised or these facilities are under maintenance. For example, the secondary clarifiers should be sized to process all the wastewater flow under peak flow conditions if all the clarifiers are online and the plant is exhibited typical settleability conditions. However, if a clarifier is offline, or if the effluent is exhibiting poor settleability, equalization can be used to control flows to a level that can be reliably treated.

There is also some uncertainty regarding potential future peak wet weather flows into the WWTP because the District does not have a long history with reliable influent flow metering. To address this issue, the following additional recommendations have been applied in this evaluation:

- The facilities upstream of primary equalization should allow for processing of flows that exceed the predicted PHWWF. The headworks and hydraulic conveyances up to the primary sedimentation basins already provide more capacity than needed to process the predicted 2035 PHWWF. However, as detailed herein, the primary treatment facilities will need to be expanded to accommodate the predicted PHWWF. Flows exceeding the PHWWF in modest amounts for short periods of time should pass through primary treatment without compromising downstream processes. Thus, equalization of raw influent will only be necessary in an emergency or extreme inflow event that significantly exceeds the predicted PHWWF.
- Some of the current storage capacity provided in Holding Basins 1, 2 and 3 should be left available to equalize primary effluent in the event that flow exceeds the PHWWF identified in this Master Plan. Therefore, only a portion of the available capacity in Holding Basins 1, 2 and 3 should be used to attenuate the predicted PHWWF when determining the size of new processes and conveyances downstream of primary treatment.

A detailed evaluation of primary equalization needs is presented in the *Wastewater Storage Assessment* TM provided as Appendix C. As discussed, the available storage capacity provided in Holding Basins 1 and 2 (approximately 7 million gallons) would be adequate to equalize primary effluent flows upstream of secondary process at the 2035 flow conditions if the flow to the downstream processes is limited to 30 mgd. Thus, designing around a 30 mgd peak flow operating condition meets all the criteria listed above and is recommended.

The recommended design approach requires that the processes downstream of the primary sedimentation basins (and equalization) be capable of treating up to 30 mgd of flow under all reasonably foreseeable operating conditions. In the event that there is an operational issue (such as poor settling sludge), flows above 30 mgd could be equalized in Holding Basins 1 and 2.

6.1.3 Process Sizing Approach

A steady state mass balance model developed in the ENVision program was used to estimate the sizing, capacity, and capacity improvements associated with each of the liquid process alternatives evaluated under this Master Plan. The program calculates the flows and loads (e.g., nitrogen species) throughout the treatment plant for each unit process, and provides a reasonable estimate for the process performance at various points throughout the plant. A more detailed description of the ENVision program and the calculations associated with each major process element in the model is provided in Appendix F-1.

In addition, the secondary clarifier capacities were evaluated using a state point analysis (SPA) model, which provides a graphical means of assessing of the secondary clarifier performance at potential solids loading rate conditions given clarifier-specific dimensions and worst-case assumptions regarding the SVI (Daigger, 1995). SPA is a well-documented and accepted tool for evaluating secondary clarifiers at the Master Planning level. A more detailed description of the SPA is provided in Appendix F-2.

6.2 HYDRAULIC CAPACITY RELATED IMPROVEMENTS

A hydraulic bottleneck has been identified between the secondary clarifiers and the chlorine contact tank that must be addressed. Recommended improvements for addressing the hydraulic constraint downstream of the secondary clarifiers are presented herein. These modifications are common to all alternatives identified in this Master Plan.

6.2.1 Assessment of Potential Improvements

As discussed in Chapter 3 (Existing Facilities), a hydraulic model of the secondary clarifiers through the Effluent Pump Station No 1/Holding Basin 4/LAVWMA facilities was developed in 2014 as part of the *Secondary Effluent Wet Weather Capacity Evaluation* (RMC, 2014). The evaluation also defined improvements for increasing the conveyance capacity downstream of the secondary clarifiers to approximately 74 mgd (the un-equalized PHWWF design capacity defined for the Stage 4 Project). The following specific improvements were identified:

1. Removing the divider wall between the CCT influent channel and CCT, creating an open channel configuration between the two structures.
2. Removing the weir separating influent and effluent sides of chlorine junction box, upstream of the CCT, creating an open channel configuration within the junction box.
3. Removing the divider wall between the CCT effluent channel and Effluent Pump Station No. 1 wet well, creating an open channel configuration between the two structures.
4. Removing or lowering the CCT effluent weir to elevation 313 ft.

The first three modifications listed above are minor, and would have no significant impact on plant operations. However, the fourth modification (removing or lowering the CCT effluent weir) would impact plant operations. Lowering the weir would permanently reduce the CCT modal contact time and an evaluation of whether this reduction would increase the likelihood of effluent limit violations would be needed before such a recommendation could move forward. Removing the CCT effluent weir would require a change in treatment plant operation. Specifically, with the CCT effluent weir removed, the minimum contact volume within the CCT tank would change depending on Effluent Pump Station No. 1 wet well water level (if effluent is being pumped from CCT), water level in Holding Basin 4 (if water is flowing by gravity from the CCT to the holding basin), or water level in the LAWVMA storage ponds (if water is flowing by gravity from the CCT to the LAWVMA ponds. While an operational strategy could be developed, changing CCT water levels would increase the operational complexity significantly.

As discussed in Chapter 2 (Wastewater Flow and Loadings), the 2035 PHWWF is estimated to be 46.7 mgd – which is significantly lower than the 74 mgd capacity needs evaluated in the Secondary Effluent Wet Weather Capacity Evaluation. As part of this Master Planning effort, the District requested that RMC update the hydraulic model to evaluate the capacity under the following conditions:

- Effluent Pump Station 1 is in service,
- The wall between the CCT influent channel and CCT is removed,
- The weir in the chlorine junction box is removed,
- The CCT effluent weir is *not* removed or lowered and,
- The wall between the CCT effluent channel and Effluent Pump Station No. 1 is *not* removed[2].

The updated model, which is depicted in the Master Plan Hydraulic Profile (RMC, 2016) figure provided in Appendix G, was used to evaluate the three scenarios detailed in Table 6-1.

Table 6-1. Summary of the Treatment Objectives and Goals

Condition No.	Flow Basis	Flow Value, mgd	Secondary Clarifiers in Service
1	PHWWF	46.7	Five
2	PHWWF	46.7	Four
3	Peak Flow with Free Discharge Over Effluent Weirs	46.1	Four

As depicted in the Master Plan Hydraulic Profile, the following conclusions can be drawn from the modeling effort:

- With five secondary clarifiers in operation, the effluent weir in Clarifier 5 would be within one inch of being submerged at the updated PHWWF condition.
- With four secondary clarifiers in operation, the effluent weir in Clarifier 4 would be submerged at the PHWWF condition and Clarifier 4 weir would be within one inch of being submerged at a flow of approximately 46.1 mgd.

[2] Note that removing the divider wall between the CCT effluent channel and Effluent Pump Station No. 1 wet well, creating an open channel configuration between the two structures, has no impact on the secondary process hydraulic profile if the effluent weir is also not lowered or removed.

As discussed previously in this Chapter, District staff have stated that primary effluent equalization should continue to be used to the extent that this flexibility allows for cost savings. As shown above, by removing both the wall between the CCT influent channel and CCT and the weir in the chlorine junction box, the District should be able to convey most of the flow through the facility without submerging secondary clarifier effluent weirs. Therefore, even considering the fact that there are uncertainties associated with accuracy limitations of the hydraulic calculations, it would be rare for the flows to reach the hydraulic capacity limitations if the two modest improvements evaluated using the updated hydraulic model are made. Given the District's ability to rely on equalization in rare storm events, it is recommended that only the two improvements evaluated in the recent model be provided.

6.2.2 Facilities Costs

The estimated capital cost for completing these hydraulic improvements are summarized in Table 6-2. These costs were developed following the procedures detailed in Chapter 5 (Evaluation Criteria), and cost estimating details are provided in Appendix H-1.

Table 6-2. Estimated Capital Costs for Constructing Hydraulic Improvements

Project Component	Estimated Cost, million $
Remove Wall at CCT Inlet	0.06
Remove Chlorine Junction Box Weir	0.04
OPCC	0.10
Construction Contingency, 10%	0.01
Total Estimated Construction Cost	0.11
Engineering, Legal, and Administrative Costs, 35%[(a)]	0.04
Total Project Costs	**0.15**

[(a)] Calculated as a percentage of the OPCC

6.3 PRIMARY EFFLUENT DIVERSION IMPROVEMENTS

As documented previously in this Chapter, the Holding Basins will be used for primary effluent equalization. Historically, equalization of primary effluent has not been closely monitored in terms of flows and volumes. However, as influent flows to the WWTP increase, the need to balance storage with treatment will become more critical. At the 2035 design condition, it is estimated that adequate storage capacity will be available for equalization if the primary effluent diversions are limited to only the flows that exceed a threshold of 30 mgd.

The WWTP is not equipped with the facilities needed to control primary effluent diversions to flows above a threshold value. Therefore, improvements to the existing primary effluent diversion facilities are needed to achieve this objective. These modifications are common to all alternatives identified in this Master Plan.

6.3.1 Assessment of Potential Improvements

Currently, primary effluent diversions are controlled by WWTP operations staff by manually opening a slide gate located on the 24-inch primary effluent diversion pipeline (see Figure 3-3). It is recommended that a flow meter and an automated control valve be added to allow for primary effluent flow management to a specific flow rate. The flow meter could be added to the 24-inch diameter primary effluent diversion pipeline Under this scenario, the WWTP SCADA system would use the diversion flow meter and primary influent flow meter data to calculate the flow being directed to the downstream process. Alternatively, a flow meter could be placed on the main 36-inch diameter primary effluent lines and this data could be used to directly measure the downstream flows. Under either case, the control valve would be automatically adjusted to maintain the downstream flow to the desired threshold.

6.3.2 Facilities Cost

For cost estimating purposes, the meter is assumed to be sized to fit on the 36-inch main primary effluent line. A summary of the estimated capital costs associated with constructing the recommended primary effluent diversion equipment is provided in Table 6-3. These costs were developed following the procedures detailed in Chapter 5 (Existing Facilities), and cost estimating details are provided in Appendix H-2.

Table 6-3. Estimated Capital Costs for Primary Effluent Diversion Equipment

Project Component	Estimated Cost, million $
Primary Flow Diversion Improvements	0.14
OPCC	0.14
Construction Contingency, 10%	0.01
Total Estimated Construction Cost	0.15
Engineering, Legal and Administrative Costs, 35%[(a)]	**0.05**
Total Project Costs	**0.20**

[(a)] Calculated as a percentage of the OPCC

6.4 PRIMARY TREATMENT EXPANSION

As documented in Chapter 3 (Existing Facilities), the primary sedimentation basins are heavily loaded, which has resulted in poor suspended solids removals. This section provides an evaluation of primary sedimentation basin improvements needed to address this concern. The following primary treatment topics are addressed in this section:

- Capacity Deficiencies
- Strategy for Evaluating Primary Treatment Improvements
- Required Facilities
- Facilities Layout
- Facilities Costs

6.4.1 Capacity Deficiencies

Capacity limitations in the primary sedimentation basins are only moderately detrimental to process performance because the solids that carry over into the secondary process can generally be handled in that process. However, it is recommended that the primary treatment capacity be expanded for three reasons. First, poor performance of the primaries means that diversion of solids to the anaerobic digesters is reduced, and biogas production is reduced accordingly. Second, solids, and BOD not captured in the primary sedimentation basin process move on to the energy intensive aeration process. Third, the additional solids loading to the aeration basins increases mixed liquor suspended solids (MLSS) concentrations, and subsequently negatively impacts secondary clarifier capacity.

6.4.2 Strategy for Evaluating Primary Treatment Improvements

Based on direction from District staff, it is assumed that the District will construct additional primary sedimentation basins to address the identified capacity deficiencies with primary sedimentation basins. This concept is proven, and injects no new operational concepts or performance uncertainty into the District's primary clarification process. This approach for adding capacity is the most conservative in terms of construction cost and space requirements. Other approaches should be considered in the preliminary design phase. Recommendations for further consideration are presented herein. These other alternatives for addressing the primary sedimentation basin capacity deficiencies will have the same benefits and impacts to downstream processes, and do not otherwise affect the long-term planning process. Therefore, the primary sedimentation basin improvements are common to all alternatives presented in this Master Plan.

These modifications are expected to occur in the near term, and will be common to all other improvements identified in this Master Plan. It is therefore assumed for planning purposes that solids loading to digesters will increase over current levels. It is also assumed that solids loading to the downstream activated sludge system and secondary clarifiers will decrease over current levels, once the primary clarification capacity is increased.

6.4.3 Required Facilities

Table 6-4 provides a summary of 2035 loading rates and hydraulic retention times for the current number of primary sedimentation basins, and loading rates and hydraulic retention times with one to four additional primaries. As shown, the primary sedimentation basin loading rate at the 2035 ADWF conditions is 1,800 gpd/sq-ft, and the peak hydraulic loading rate is 6,100 gpd/sq-ft. These are far above the loading rates typically recommended in reference documents. As presented in the table, three additional primary sedimentation basins (seven total) are needed to provide acceptable loading according to the listed criteria at 2035 flow projections. With seven primary sedimentation basins in total, TSS removals of 60 to 70 percent and BOD removals of up to 35 percent should be readily achievable.

Table 6-4. Primary Treatment Loading and Retention Time at Year 2035 Flows and Loads

Number of Primaries	ADWF Hydraulic Loading Rate, gpd/sq-ft	ADWF Hydraulic Retention Time,[(a)] hours	Peak Hydraulic Loading Rate,[(b)] gpd/sq-ft	Peak Hydraulic Retention Time, hours
4 (Current)	1,800	0.91	6,100	0.26
5	1,400	1.14	4,900	0.33
6	1,200	1.36	4,100	0.39
7	1,000	1.59	3,500	0.46
8	900	1.82	3,100	0.53

[(a)] The recommended hydraulic retention time is 1.5 hours for ADWF flows.
[(b)] The recommended design criteria are 3,600 gpd/sf for peak flows.

Another issue that will need to be evaluated as part of the primary sedimentation basin expansion project is the ability to split flow relatively evenly between the seven sedimentation basins. As shown in Figure 3-3, flow exits Grit Tank 1 flows into the West Primary Influent Channel, and flow exits Grit Tanks 2 and 3 into the East Primary Influent Channel. These two influent channels are connected via the Grit Tank Bypass Channel. From these connected channels, the wastewater enters the four primary sedimentation basins. Weirs upstream of the grit tanks split flow evenly to the tanks, but no other significant flow control is present in the downstream channels or going into the sedimentation basins. As a result, flows are not currently evenly split between the four sedimentation basins. This flow split issue was identified as part of the Plant Dry Weather Process Capacity Study (Brown and Caldwell, 2004). Unless improvements are made to improve the flow split between the grit tanks and the primary sedimentation basins, construction of three additional sedimentation basins will further exacerbate the existing flow split issues. In this study, it was assumed that orifice plates will be utilized at the upstream end (influent) of the primary sedimentation tanks to increase head loss to achieve equal flow split among the tanks.

6.4.4 Facilities Layout

The layout of the three new sedimentation basins is shown on Figure 6-1. As shown, the addition of three sedimentation basins will encroach on the space currently occupied by Holding Basin 1. This encroachment would reduce the total available storage volume in Holding Basin 1 by ten percent, or reduce it from approximately 2.2 Mgal to 2.0 Mgal. Although this is a modest encroachment on Holding Basin 1, the District may want to consider smaller footprint approaches to minimize impacts on storage. Additional discussion of the pros and cons associated with potential alternatives that should be considered as the District implements the primary treatment expansion project are detailed in Chapter 10 (Recommended Plan).

6.4.5 Facilities Costs

A summary of the estimated capital costs associated with constructing three additional primary sedimentation basins is provided in Table 6-5. These costs were developed following the procedures detailed in Chapter 5 (Existing Facilities), and cost estimating details are provided in Appendix H-3. A summary of the annual operation costs are shown in Table 6-6. These costs were also developed following the procedures detailed in Chapter 5, and cost estimating details are provided in Appendix H-1. Table 6-7 provides the NPV of the operating costs over an assumed 30-year period, and the total lifecycle costs calculated as the sum of the estimated capital costs and the NPV of the annual operating costs.

Table 6-5. Estimated Capital Costs for Three Additional Primary Sedimentation Basins

Project Component	Estimated Cost, million $
Primary Sedimentation Basins (3)	5.4
Primary Solids Pumps (3)	1.3
OPCC	6.7
Construction Contingency, 10%	0.7
Total Estimated Construction Cost	7.4
Engineering, Legal and Administrative Costs, 35%[(a)]	2.3
Total Project Costs	**9.7**

[(b)] Calculated as a percentage of the OPCC

Table 6-6. Estimated Annual Operating Costs for Primary Sedimentation Basins

Operating Cost Component	Annual Cost, million $
Chemicals	-
Electrical	0.01
Labor	0.1
Total Annual Cost	**0.1**

Table 6-7. Estimated Lifecycle Costs for Three Additional Primary Sedimentation Basins

Cost Type	Net Present Value, million $
Total Capital Costs	9.7
Total Annual Costs as NPV[(a)]	2.1
Total Present Worth Value (Lifecycle Cost)	**11.8**

[(a)] Assumes 30-year lifecycle

6.5 ALTERNATIVES FOR ADDRESSING SECONDARY CLARIFIER CAPACITY LIMITATIONS

The secondary clarifiers are not capable of treating the full range of flow and SVI conditions that are likely to occur over the Master Planning period. This section presents an evaluation of the alternatives identified for addressing the secondary clarifier capacity deficiencies. The topics addressed include:

- Alternatives Overview
- Alternatives Comparison

6.5.1 Alternatives Overview

The following four alternatives have been identified for addressing the secondary clarifier capacity limitations:

- Additional (fifth) secondary clarifier
- Alum addition to the facultative sludge lagoon (FSL) supernatant to address struvite concerns and facilitate year-round anaerobic selector operation
- Side-stream phosphorus recovery for year-round anaerobic selector operation
- Parallel stream membrane bioreactor (MBR)

6.5.1.1 Additional Secondary Clarifier

The addition of a fifth secondary clarifier is an effective means of reducing clarifier loading and in turn increasing clarifier capacity. If the new clarifier had similar dimensions as the existing largest secondary clarifier (Number 4), the secondary clarification capacity would increase from 24 mgd to 31 mgd during peak flow events (assuming worst-case SVI values of 300 mL/g). While this is a significant increase in capacity, the District would still need to equalize primary effluent if SVIs reach this historic high level. However, when SVI is at 200 mL/g or less, the District should be able to treat the PHWWF if all five aeration basins and clarifiers are in service.

6.5.1.2 Alum Addition for Year-Round Anaerobic Selector Operation

Rather than constructing a new secondary clarifier, it is possible to reliably improve the settleability (i.e., maintain lower SVIs) and, in turn, increase the existing secondary clarifier capacity. Specifically, it has been demonstrated that the SVIs improve significantly when the aeration basins are operated in series mode. While in this mode, all the primary effluent and RAS is fed into the third aeration basin and it operates as an anaerobic selector that is favorable to growth of well-settling biomass (see Figure 3-10). Following the third basin, the water is split between the second and fourth basins, after which it is returned back down the first and fifth basins. Under current operating conditions, the District switches to parallel mode during the winter months when the FSL supernatant is returned to the main plant. This shift in treatment mode is implemented to reduce the potential for struvite formation. By doing so, the third basin is no longer operated as an anaerobic selector and the growth of the well-settling sludge is stifled.

The struvite formation potential during periods when FSL supernatant is returned to the main plant could be nullified by removing the FSL supernatant phosphate load upstream of the activated sludge process. The strategy for removing phosphate in FSL supernatant is to add a metal salt (e.g., alum) to the FSL supernatant to precipitate phosphate, followed by removal as the solids settle in the primaries. The metal salt addition could occur at the FSLs supernatant flume which would facilitate proper mixing and sufficient contact time from the flume to the plant headworks.

The potential plant impacts associated with adding chemicals into the FSL supernatant is summarized in Table 6-8. In general, the added metal salt would not re-solubilize in downstream treatment processes or negatively impact the primary sedimentation basins or digesters. The main impacts from metal salt addition would be a marginal increase in solids production, less ammonia removal than the current mode of operation, and less alkalinity due to consumption by the metal salt.

Table 6-8. Potential Plant-Wide Implications of Adding Metal Salt to the FSL Supernatant

Issue	Impact	Comment
Fate of Chemically Bound Phosphorus in Solids	Phosphate precipitated by metal salt addition is bound and stable. It will not re-solubilize at other locations in the plant.	Recent work has shown that the metal/phosphorus chemistry is closely tied to hydroxide formation, and that covalent bonding to metal hydroxides forms the major mechanism for phosphorus removal. A more detailed discussion on the fundamentals of phosphorus chemical precipitation can be found in Smith et al. (2008).
Impact on Primary Sedimentation Basins	The precipitate formed form metal salt addition has a high settling potential that should be captured in the primaries.	There is little or no concern over impacting the primaries as metal salt addition is common practice (e.g., chemically enhanced primary treatment).
Impact on Anaerobic Digesters	There will be additional phosphorus removal in the digesters from metal salts that eventually end up in the digesters. The extent of additional phosphorus removal should be marginal, but noticeable, while monitoring phosphorus plant discharge levels.	Residual active metal salt will react with phosphorus released due to volatile solids destruction in the digesters. The extent of additional removal is site specific, and dependent on how much metal salt is dosed at the FSL supernatant flume, volatile solids destruction, and other variables. The additional phosphorus captured will end up bound with the solids in the FSLs/DLDs.
Impact on Solids Production	There will be an overall increase in solids produced by the plant.	Metal salt addition produces approximately 0.3 pound of TSS per pound of alum added.
Impact on Ammonia Removal	There will be less overall ammonia removal in the plant due to reduced struvite formation.	Struvite precipitation removes a portion of ammonia at approximately 0.4 pound of Ammonia as N for each pound of Phosphorus precipitated. The extent of impact on performance is governed by how much struvite the plant is currently forming.
Impact on Potential Future Nitrification	There will be less available alkalinity for nitrification.	Metal salt addition consumes alkalinity at 0.5 lb of Alkalinity per pound Alum added. The available effluent alkalinity data is wide ranging with values from approximately 180 to 665 mg/L as CaCO3. In recent years, the extent of data scatter has reduced with an average of approximately 300 to 350 mg/L. Such values suggest that there is sufficient alkalinity for full nitrification and denitrification. Nevertheless, given the limited amount of available data, it is recommended that alkalinity be included in the evaluation if ammonia/total nitrogen removal is required in the future.

6.5.1.3 Side Stream Phosphorus Recovery for Year-Round Anaerobic Selector Operation

In lieu of adding alum to the FSL return flow to chemically bind phosphorus in the plant's biosolids, phosphorus recovery could be used to remove phosphorus from the concentrated biosolids treatment return flows, allowing for year-round anaerobic selector operation. This approach also provides a marketable product in the form of phosphorus fertilizer.

Previous studies completed by the District have demonstrated that phosphorus concentrations in the FSL return flow are not high enough to make phosphorus recovery an economically viable strategy for this side stream. However, it would be possible to provide a WAS stripping process upstream of the existing DAFT process to create a high-strength phosphorus return stream at the main treatment plant site.

The WAS stripping process would consist of a small (approximately 400 square feet) solids blend tank that provides between 30 and 60 minutes of detention time for the combined WAS and primary solids flows. The anaerobic conditions that occur in the tank would be adequate to "strip" the phosphorus from the solids into the liquid portion of the solids stream. Following the DAFT, the liquid return flow would be transferred to a phosphorus recovery process that is also located at the main treatment plant site.

Following an initial review of this concept with District staff, phosphorus recovery as a means of addressing the near-term secondary clarifier capacity issues was ruled out for further consideration for the following reasons:

- There is limited space available near the DAFT, even for small solids blend tank. Moreover, all of the available space in this area should be maintained, to the extent feasible, to allow for future expansion of the cogeneration facilities.
- The current technologies available for phosphorus recovery have been demonstrated to only be economically attractive for much larger facilities than the WWTP. The typical cut-off is at approximately 40 mgd ADWF permitted capacity plants. The economics are not likely to be promising for the WWTP unless the cost of the recovery facilities are reduced or the price of phosphorus increases.
- The only viable location for the phosphorus recovery facilities is within the existing Holding Basins. This space should be maintained, to the extent feasible, until future decisions regarding nutrient removal and potable reuse have been made.

Although phosphorus recovery has been ruled out as a means of meeting the near-term secondary clarifier issues, this concept should not be ruled out as part of the overall long-term strategy for the WWTP. District staff should remain apprised of advances in the technology and look for opportunities to implement a phosphorus recovery strategy, should conditions become more favorable in the future.

6.5.1.4 Parallel Stream Membrane Bioreactor (MBR)

A final option that has been considered is to add a parallel system to treat a portion of the primary effluent. This approach would divert a portion of the wastewater flow and load continuously away from the activated sludge system, thereby reducing the hydraulic and solids loading rates to the secondary clarifiers.

As discussed in Chapter 7 (Potable Reuse Alternatives), the District is considering rehabilitating the 3 mgd (product water flow) MF/RO treatment train that was constructed under the Clean Water Revival project to allow for the creation of a product water that can be used for potable reuse purposes. Therefore, a key driver for considering this alternative is the potential for the new parallel treatment train to meet the pretreatment requirements for a future potable reuse treatment system (in lieu of upgrading the entire treatment process).

The MBR was selected for this alternative because it meets the pre-treatment objectives for a potable reuse system and (once disinfected) provides appropriate water quality for irrigation reuse all within a very compact footprint. The MBR system is a suspended growth treatment process that combines secondary and tertiary processes (i.e., activated sludge, secondary clarification, and filtration)[3]. The MBR consists of two reactors. The first reactor is a suspended growth, activated sludge aeration basin that is used to meet process air requirements. The second reactor is a membrane tank that separates the product water from the activated sludge solids through either hollow-fiber membranes or flat sheets that are immersed in the tank. RAS is returned from the membrane tank to the head end of the aeration basin, and the systems typically operates with a mixed liquor suspended solids concertation of 8,000 to 10,000 mg/L. Fine screens (1-2 millimeter) are installed upstream of the MBR. The fine screen is critical for removing the likes of hair which can negatively impact membrane integrity.

The MBR alternative also requires installation of a new stand-alone blower building separate from the activated sludge blower building. The facility would include the blowers, the new building, overhead crane, 30’ of air piping, and HVAC system for the new building. It is possible that the existing Building D could be repurposed for use as the MBR blower building, which would help to reduce the cost associated with this alternative.

As noted above, the key driver for this alternative is meeting the pretreatment requirements for feeding a future 3 mgd potable reuse treatment system. Therefore, a 4-mgd MBR treatment train is assumed. (Additional discussion regarding the potable reuse system is provided in Chapter 7.) The 4 mgd MBR treatment train should be able to treat flows greater than 4 mgd during short-term, peak flow events. For this analysis, it is assumed that 8 mgd could be treated through the MBR process, if needed, to reduce loadings on the secondary clarifiers.

[3] MBRs also have the ability to treat screened and degritted wastewater, and thus replace primary clarifiers also. However, based on direction received from District staff, this evaluation focused on MBR treatment of primary effluent.

A final benefit of this alternative is that the MBR provides low-level nutrient removal. Thus, if a parallel stream MBR is constructed now, there could be cost savings in the future if nutrient removal facilities become necessary. Additional discussion regarding these benefits are provided later in this Chapter.

6.5.2 Alternatives Comparison

This section presents a comparison of the three viable alternatives discussed above with respect to treatment capacity, layout, lifecycles costs, and the non-economic factors. All of the alternatives identified also provide advantages and disadvantages with respect to potential long-term nutrient removal and potable reuse projects that might occur in the future. Therefore, a recommended strategy for meeting the secondary clarifier capacity needs must consider all of these potential future facilities. A recommended approach to addressing the secondary clarifier capacity limitations is not addressed below, but is instead provided in Chapter 10 (Recommended Plan).

6.5.2.1 Treatment Capacity

The ENVision model was used to evaluate the three alternatives identified for addressing the secondary clarifier capacity limitations. The major assumptions applied in completing this evaluation were as follows:

- Under all three alternatives, the anaerobic selector would be operated during the dry season. However, only the alum addition alternative allows for wet season operation of the anaerobic selector.
- In accordance with the evaluation presented in Chapter 3, the worst case SVI will be 200 ml/g when the anaerobic selector is in operation, and the worst case SVI will be 300 ml/g when the anaerobic selector is not in operation.
- The District will be able to keep all secondary clarifiers in service during peak wet season flow events. Therefore, the wet season operations should be modeled based on all clarifier units being online.
- Major maintenance could occur in the dry season, so dry season operations should be modeled based on one unit out of service.

A summary of the results of the ENVision model analysis are presented in Table 6-9. As shown, all three alternatives can readily treat the PHDWF of 23.4 mgd with the largest unit out of service (and the anaerobic selector in operation). However, the peak flow that can be treated in the wet season varies significantly between the alternatives. The additional secondary clarifier alternative has the lowest capacity and the parallel stream MBR alternative provides the highest capacity. Nevertheless, all three alternatives meet the minimum capacity requirement sofa 30 mgd. Additional detailed results from the ENVision modeling evaluation are provided in Appendix J.

Table 6-9. Secondary Clarifier Treatment Capacity Summary

Parameter	Unit	Additional Secondary Clarifier	Alum Addition for Year-Round Anaerobic Selector Operation	Parallel Stream MBR
Primary Sedimentation Basins				
Solids Capture	%	65		
Aeration Basins (Existing)				
Total Number of Basins		5		
Aerobic SRT	days	2.3		
MLSS (Max Month)	mg/L	1,650-1,750	1,650-1,750	1,100-1,200
Secondary Clarifiers				
Total Number of Units		5	4	4
Dry Season Operations				
Minimum Number of Units Online		4	3	3
Anaerobic Selector Operation?		Yes	Yes	Yes
SVI Basis (worst case)	mL/g	200	200	200
Capacity[a]	mgd	>23.4	>23.4	>23.4
Wet Season Operations				
Minimum Number of Units Online		5	4	4
Anaerobic Selector Operation?		No	Yes	No
SVI Basis (worst-case)	mL/g	300	200	300
Capacity[b]	mgd	30	36.7	43.5
Supernatant Lagoon Treatment				
Alum Demand	gpd	--	330	--

[a] The 2035 PHDWF is 23.4 mgd.
[b] The PHWWF is 46.7 mgd.

6.5.2.2 Facilities Layout

The proposed layout of the three alternatives discussed above is shown on Figure 6-2. Also shown are the location of the expanded primary sedimentation basins (in red). The following information is shown on this figure:

- The additional secondary clarifier facility, shown in green, would be located west of the existing clarifiers. This location was identified previously as the preferred secondary clarifier location.
- The alum addition facilities (shown in orange) would be located at the FSL site (as was noted previously). For this alternative, the District would also operate the aeration basins with an anaerobic selector year-round.
- The proposed MBR location is (shown in yellow) is adjacent to the expanded primary sedimentation basins and would take up a portion of Holding Basin 1. This location allows for minimum conveyances to deliver water from the primary sedimentation basins to the MBR, and deliver MBR effluent to either the existing CCT or to the Microfiltration Transfer Pump Station - from where the effluent can be transferred to the MF Building.

Under the parallel stream MBR alternative, the District would lose an additional 0.5 million gallons of storage in Holding Basin 1, resulting in a total capacity (with no freeboard) of 1.5 million gallons - down from 2.2 million gallons. Combined with the storage loss associated with the primary sedimentation basin expansion, the total storage loss in Holding Basin 1 would be 0.7 million gallons. Therefore, with this alternative, the total amount of available primary effluent storage would decrease from 13.6 million gallons to 12.9 million gallons assuming no freeboard, and from 12.2 million gallons to 11.4 million gallons assuming a one-foot freeboard (see Chapter 3, Existing Facilities). However, it is estimated that the District would only need approximately 7.1 million gallons of primary effluent storage if the treatment plant could reliably treat up to 30 mgd under all conditions (see Appendix C for details). Nevertheless, any loss of available storage is not desired.

6.5.2.3 Lifecycle Costs

A comparison of the capital costs for each of the three alternatives is provided in Table 6-10. These costs were developed following the procedures detailed in Chapter 5 (Evaluation Criteria), and cost estimating details for the three alternatives are provided in Appendix H-4. Annual operation costs and NPV costs are shown in Tables 6-11 and 6-12, respectively. A more detailed breakdown of these operating costs are provided in Appendix I-2. As shown in these tables, the costs associated with alum addition at the FSLs to allow for year-round anaerobic selector operations are significantly lower than the other two alternatives. However, as discussed later in this Chapter, there could be substantial economic benefits associated with the other two alternatives under a future nutrient removal scenario. These benefits should therefore be considered as part of the economic evaluation of these alternatives.

Table 6-10. Estimating Capital Costs of Alternatives for Addressing Secondary Clarifier Capacity Limitations

Cost Component	Estimated Costs, million $		
	Additional Secondary Clarifier	Alum Addition for Year-Round Anaerobic Selector Operation	Parallel Stream MBR
OPCC	9.1	0.5	35.0
Construction Contingency, 10%	0.9	0.1	3.5
Total Estimated Construction Cost	10.0	0.6	38.5
Engineering, Legal and Administrative Costs, 35%[(a)]	3.2	0.2	12.3
Total Project Costs	**13.2**	**0.8**	**50.8**

[(a)] Calculated as a percentage of the OPCC

Table 6-11. Annual Operating Costs of Alternatives for Addressing Secondary Clarifier Capacity Limitations

Operating Cost Component	Estimated Costs, million $		
	Additional Secondary Clarifier	Alum Addition for Year-Round Anaerobic Selector Operation	Parallel Stream MBR
Chemicals	0	0.1	0.1
Electrical	0.8	0.8	1.0
Labor	0.9	0.8	1.1
Total	1.7	1.7	2.2

Table 6-12. Lifecycle Costs of Alternatives for Addressing Secondary Clarifier Capacity Limitations

Cost Component	Estimated Costs, million $		
	Additional Secondary Clarifier	Alum Addition for Year-Round Anaerobic Selector Operation	Parallel Stream MBR
Total Capital Costs	13.2	0.8	50.8
Total Annual Costs as NPV[(a)]	36.4	36.4	47.2
Total Present Worth Value (Lifecycle Cost)	49.6	37.2	98.0

[(a)] Assumes 30-year lifecycle

6.5.2.4 Non-Economic Comparison

Table 6-13 provides a comparison of the alternatives using the non-economic criteria that were discussed in Chapter 5 (Evaluation Criteria).

Table 6-13. Non-Economic Cost Comparison of Alternatives for Addressing Secondary Clarifier Capacity Limitations

Criteria	Weighted Subjective Criteria		
	Additional Secondary Clarifier	Alum Addition for Year-Round Anaerobic Selector Operation	Parallel Stream MBR
Reliability	7.0	5.7	5.0
Flexibility	4.7	8.2	7.0
Odor generation potential	(a)	(a)	(a)
Maximize Available Water Storage Volume	3.8	3.8	2.5
Energy Management	3.0	4.0	3.0
Chemical use	4.0	1.0	2.0
Resource Recovery	2.3	2.6	2.3
Ancillary Impacts	(a)	(a)	(a)
Total Subjective Ranking	**24.7**	**25.2**	**21.8**

(a) Category is not applicable to these alternatives.

As shown in Table 6-13, there is little difference in the overall subjective ranking for the secondary clarifier and alum addition alternatives. However, the parallel stream MBR alternative ranks significantly lower. Additional details regarding this subjective comparison are provided in Appendix K. A few items of note are:

- The additional secondary clarifier alternative ranks highest with respect to reliability because it does not introduce a new treatment process and does not require any modification to current operations.
- The additional secondary clarifier alternative ranks lowest with respect to flexibility, however, because it locks the District into a conventional secondary clarification approach, whereas the other alternatives do not impact the existing secondary process, allowing for more flexibility down the road.
- The parallel stream MBR ranks moderately high with respect to flexibility because it provides pre-treatment for a potable reuse system and accomplishes some nutrient removal.

6.6 POTENTIAL FUTURE NUTRIENT REMOVAL IMPROVEMENTS

It is likely that there will be future requirements that will be applied to the WWTP with respect to nutrient removal. This section presents an overview of the additional liquid treatment process facilities that could be needed to meet these future nutrient removal requirements. The topics addressed include:

- Potential Nutrient Removal Requirements
- Strategy for Evaluating Potential Future Nutrient Removal Facilities
- Facilities Overview
- Facilities Layout
- Lifecycle Costs

6.6.1 Potential Nutrient Removal Requirements

As noted above, a Visioning Panel was convened to help identify improvements that may be necessary in the future, either to address emerging regulatory requirements or to take advantage of emerging resource recovery opportunities. At the Visioning Panel workshops, the following conclusions were made with respect to future nutrient removal requirements:

- As discussed in Chapter 4 (Regulatory Requirements), the Regional Board is considering the need for nutrient removal requirements to be placed on POTWs that discharge to the SF Bay.
- It is anticipated that the 2019 Nutrient Watershed Permit will not include any numeric effluent limits for nutrient constituents.
- The District will significantly reduce nutrient loads to the Bay through the implementation of the recycled water projects current being put into place.
- It is likely that 2024 Nutrient Watershed Permit will require nutrient load reductions.
- The District will likely have five to ten years to comply with any new nutrient reduction requirements.
- There is no certainty with respect to the level of treatment that will be required.
- It is unclear whether nutrient removal would be required year-round or seasonally.
- It is unclear whether nutrient removal would be applied as concentration-based standards or as loading limits.
- The wastewater oxidation that occurs as part of conventional ammonia treatment (also referred to as nitrification) is one option for meeting the pretreatment requirements for a potential future potable reuse treatment system.

- Several promising technologies are currently emerging in the wastewater industry with respect to nutrient removal.
- Because the timing of nutrient removal is uncertain and there is potential for new technologies to be available when nutrient removal is ultimately required, picking a preferred technology or approach to nutrient removal is not recommended at this time.

6.6.2 Strategy for Evaluating Potential Future Nutrient Removal Facilities

Given the uncertainty with potential future requirements and needs, a range of treatment levels have been considered in the Master Plan. Both nitrification for meeting the pretreatment needs of a potable reuse system and the treatment improvements needed to meet the water quality objectives defined under the current BACWA nutrient study are considered. A summary of the specific treatment levels evaluated in the Master Plan are listed in Table 6-14. Additional discussion of wastewater treatment plant nutrient removal fundamentals is provided in Appendix L.

Table 6-14. Summary of the Treatment Objectives and Goals

Treatment Level	Objective	Additional Criteria
Potable Reuse Conditioning[(a)]	• Full nitrification, or Ammonia as N <2 mg/L • Total Nitrogen as N <15 mg/L	Short term exceedances are acceptable and should not have an adverse impact on the potable reuse goals.
BACWA Level 2	• Full nitrification, or Ammonia as N <2 mg/L • Total Nitrogen as N <15 mg/L • Total Phosphorus as P <1 mg/L	Criteria should be met reliably under all anticipated operating conditions.
BACWA Level 3	• Full nitrification, or Ammonia as N <2 mg/L • Total Nitrogen as N <6 mg/L • Total Phosphorus as P <0.3 mg/L	Criteria should be met reliably under all anticipated operating conditions.

[(a)] Conditioning water for potable reuse refers to improving the water quality upstream of MF and RO processes.

In addition to a range of treatment levels, it is assumed that year-round compliance with concentration-based limits will be required under a future regulatory driven scenario. This approach requires a high level of conservatism in the design because the facilities will need to consistently meet an effluent target at a wide range of flow and temperature conditions. It is also understood that the process sizes could likely be reduced if the regulatory requirements are applied seasonally or as load-based limits.

Finally, a conventional approach to nutrient removal that relies on current industry standards would be applied. Similar to the approach for defining future primary sedimentation basin needs, relying on conventional industry standards for nutrient removal is the most conservative in terms of construction cost and space requirements and provides a reasonable basis for long-term planning. However, as regulation are better defined, a range of potential nutrient approaches will need to be considered in future planning phases. Recommendations for studies that can be done over the next ten years to further evaluate potential emerging technologies are presented in Chapter 10 (Recommended Plan).

6.6.3 Facilities Overview

A description of the facilities required to achieve the three nutrient removal scenarios described in Table 6-14 is provided below. The recommend sizing of these facilities was determined using ENVision model described previously in this Chapter, and detailed results from this evaluation are provided in Appendix J. In addition to the additional nutrient removal facilities, it will be necessary to improve the chlorination system to provide a higher level of control. This improvement is also discussed below.

6.6.3.1 Chlorination System Improvements

An improved chlorination system control is needed if the main treatment plant is nitrifying because the chlorination system will be relying on free chlorine instead of chloramines, which are more stable than free chlorine. As discussed in Chapter 3 (Existing Facilities), this improvement would involve modifying the chlorine dosing control strategy to rely on feedback from the current residual monitoring equipment at the contactor, augmented by a chlorine analyzer located at the LAVWMA Junction Structure to provide for chlorine residual concentration data. The first on-line analyzer's target chlorine residual will be adjusted to modify the current chlorine dose if the second on-line analyzer indicates that the current chlorine residual is approaching the required 2 mg/L residual. Similarly, a new secondary chlorine residual monitoring site will be needed at the Effluent Diversion Structure for use when effluent is first diverted to Holding Basin 4. This improvement is common to all of the nutrient removal scenarios discussed below.

6.6.3.2 Potable Reuse Conditioning

To meet the potable reuse conditioning goals, the WWTP would need to include a nitrification process. Nitrification is achieved in an activated sludge process by increasing the solids retention time (SRT)[4] in the aerobic reactors from one to two days to up to five days or greater[5]. Raising dissolved oxygen concentrations in the aeration basins to 2 mg/L also optimizes the nitrification reaction.

SRT levels are increased by reducing the solids wasting rate from the aeration basins, and the result of this action is a more than doubling of the MLSS concentration in the basins. An increase in the basin size is typically needed to avoid MLSS concentrations that are too high readily settle in the secondary clarifiers. Ideally MLSS concentrations are maintained below 3,000 mg/L. Finally, the increased aeration basin MLSS concentrations result in increased solids loading to the secondary clarifiers - thus driving the need to increase capacity of secondary clarifiers.

[4] SRT is the ratio of the total solids in the aeration basin to the total amount of solids wasted and/or discharged from the aeration basin.
[5] SRT requirements are dependent on temperature.

For purposes of this analysis, it has been assumed that the existing basins would be converted to a Modified Ludzack-Ettinger Process (MLE) to achieve nitrification and partial denitrification. The MLE process has an anoxic zone upstream of the aerobic zone, and uses an internal recycle to carry nitrates from the aerobic zone (where nitrification occurs) back to the anoxic zone for denitrification. The advantages of the MLE process over nitrification-only approach are: the denitrification process will result in recovery of some alkalinity to support nitrification, the returned nitrate serves as an electron donor in the nitrification process reducing the overall aeration demands, the presence of an anoxic zone improves settleability characteristics of the MLSS (similar to the anaerobic zone used now).

Finally, under a potable reuse conditioning scenario, occasional breakthroughs of ammonia would not have significant adverse effects on the downstream process and denitrification is only needed to the extent is provides treatment benefits. Therefore, a lower safety factor is appropriate when sizing the facilities for this alternative.

If treating all of the 2035 flows and loads, two additional aeration basins and one additional secondary clarifier would need to be constructed. All of the aeration basins would be modified to provide an anoxic and aerobic zone, and a new mixed liquor return channel with pump station would be constructed around the basins. If a parallel stream MBR system with 4 mgd of capacity were constructed, only one new aeration basin would be needed.

The blower capacity would also need to be expanded if the aeration basins were used to treat all of the 2035 flows and loads. Two additional blowers, one serving as the standby unit, would be required. For purposes of this analysis, it has been assumed that the District would install high speed turbo blowers for this purpose. These blowers could be installed in the existing blower building. If a parallel stream MBR were installed, the activated sludge system aeration system would not require additional blowers. However, additional blowers (including a stand-alone blower building separate from the activated sludge blower building) would be constructed as part of the MBR project.

6.6.3.3 BACWA Level 2

The treatment processes discussed above for the Potable Reuse Conditioning scenario would be further modified to meet the BACWA Level 2 treatment objectives, as follows:

- The aeration basins are further divided to create three zones. The anoxic zone created under the Potable Reuse Conditioning scenario would be converted to an anaerobic zone, and a new anoxic zone would be constructed downstream of the anaerobic zone.
- The addition of the new zone will reduce the overall volume available for nitrification in each basin, requiring a basin expansion. Regardless of whether there is a need to treating all of the 2035 flows and loads or there is a parallel stream MBR system with 4 mgd of capacity, one additional aeration basin is needed beyond the basins identified for Potable Reuse Conditioning scenario described above.

- The new anaerobic zone will allow for biological phosphorus removal in the basins, similar to the current summer-month operations. To avoid the struvite formation that occurs in the anaerobic digester when biological phosphorus removal is being applied, the District could add alum to the FSL return to chemically bind the phosphorus.
- The internal mixed liquor facilities will need to be modified to allow for return flows to the anoxic zone, which is now downstream of the new anaerobic zone.
- The mixed liquor return pumping capacity may also need to be increased to improve denitrification reliability.

No additional blower capacity would be needed beyond what is provided by the Potable Reuse Conditioning alternative.

6.6.3.4 BACWA Level 3

The aeration basin configuration discussed above for the BACWA Level 2 scenario would be further modified to meet the BACWA Level 3 treatment objectives, as follows:

- The aeration basins are further divided to create a total of four zones. This configuration is known as a four-stage Bhardenpho process. The new zones are created at the end of each basin, and are operated as anoxic zones. These zones are about half the size of the upstream anoxic zones, and an external carbon source (such as methanol, which has been assumed for this Master Plan) is added to this second anoxic zone to provide for denitrification.
- To support the new denitrification process, a new methanol addition facility and associated piping would need to be constructed.
- The addition of the new downstream anoxic zone also reduces the overall volume available for nitrification in each basin, requiring a further basin expansion.

No additional blower capacity would be needed beyond what is provided by the Potable Reuse Conditioning alternative.

The analysis demonstrates that the additional basins needed to accommodate the increased treatment cannot reasonably fit within the available treatment area. To address this issue, it has been assumed for purposes of this analysis that an integrated fixed film activated sludge (IFAS) system would be installed in the basins. An IFAS system uses the same treatment principles the suspended growth process, but also has a media installed in the basins to supports the growth of attached microorganisms. With this media in place, many of the organisms that support nitrification and denitrification stay in the reactors – thus reducing the solids separation burden applied to the secondary clarifiers. With an IFAS system installed, an additional two aeration basins will need to be constructed to improve the nitrogen treatment capacity from BACWA Level 2 to BACWA Level 3.

To meet the increased phosphorus removal requirements prescribed under BACWA Level 3, a final solids separation process (such as the ballasted flocculation facilities currently being installed) will be needed. For this analysis, it has been assumed that the ballasted flocculation facilities would be modified to operate year-round, and that all wastewater would be directed through these facilities, where the production capacity of this system is expected to be 18 mgd, with the objective of providing flow to the sand filtration process that could produce up to 16 mgd. Secondary effluent flows would be equalized in Holding Basin 4 upstream of the filter pump station, and treated through the filtration process at a rate of 16 mgd.

On days when the influent flows to the WWTP exceed 16 mgd, a portion of the wastewater may need to bypass the ballasted flocculation and sand filtration facilities. However, such bypasses are expected to be rare and not likely to have an impact on the District's ability to meet the phosphorus requirements.

Finally, under the BACWA Level 3 scenario, biosolids return stream treatment would be recommended since this is a source of high nitrogen loads to the plant. The return stream is typically a low flow (typically a few percent of raw influent) and highly concentrated with nutrients (>500 mg N/L), which is ideal for cost effective and compact nitrogen removal. However, under the current WWTP configuration, the return stream from the FSLs in only seasonal and thus not readily amenable to biological treatment. For purposes of this Master Plan, it has been assumed that the District would either move to year-round FSL return or mechanical solids dewatering under a BACWA Level 3 scenario, thus allowing for year-round return flow treatment using conventional processes. In addition, for purposes of this analysis, a conventional sequencing batch reactor facility located near the FSL site is assumed. The return flow treatment technology recommendation might change to a deammonification technology if mechanical dewatering were implemented and/or if the District was considering a mainstream deammonification process in the future. Additional discussion regarding factors that should be considered in the future with respect to selecting a return stream treatment technology is provided in Chapter 10 (Recommended Plan) of this Master Plan.

6.6.3.5 Summary of Facilities and Capacities

The major ENVision model outputs generated for the three nutrient removal treatment scenarios is provided in Table 6-15. As shown, the available treatment capacities with and without the parallel stream MBR are the same with the exception of the secondary clarifiers. For the secondary clarifiers, the MBR options provides more total clarification capacity.

Table 6-15. Treatment Capacity Summary

Parameter	Units	Treatment All Within Plant			Parallel Stream MBR (4 mgd ADWF capacity)		
		Potable Reuse Conditioning	BACWA Level 2	BACWA Level 3	Potable Reuse Conditioning	BACWA Level 2	BACWA Level 3
Primary Sedimentation Basins							
Number of Units	Number	7					
Solids Capture	%	65					
Aeration Basins (Existing)							
Number of Basins	Number	7	8	10	6	7	9
Aerobic SRT	days	6.5	>8	>8	6.5	>8	>8
MLSS (Max Day)	mg/L	2,900	2,800	2,200[(a)]	2,200	2,000	1,500[(a)]
Mixed Liquor Pumping	% of Influent	200	200	300 – 400	200	200	300 – 400
Methanol Demand	gpd	-	-	1,100	-	-	1,100
Secondary Clarifiers							
Number of Units	Number	5			4		
Firm Capacity	mgd	29	31	26	31	35.5	31.5
Total Capacity	mgd	36	38	32	>36.7[(b)]	>36.7[(b)]	>36.7[(b)]
SVI Basis	mL/g	120	120	200	120	120	200
Supernatant Lagoon Treatment							
Alum Demand	gpd	--	330	330	--	330	330
Ballasted Flocculation Treatment							
Alum Demand	gpd	--	--	2,600	--	--	2,600
Polymer Demand	gpd	--	--	30	--	--	30

(a) Assumed an IFAS system is installed in all basins.

(b) Assumed the MBR can handle peak flows of approximately 10 mgd, which translates to a peak wet weather flow of less than 36.7 in the activated sludge basins (46.7 mgd – 10 mgd = 36.7 mgd).

6.6.4 Facilities Layout

The layout of the treatment facilities will vary depending on what (if any) secondary clarifier capacity improvements are provided in an earlier phase. Three figures were created to demonstrate the potential facilities layout, depending on what is assumed to be the first phase of construction, as follows:

- Figure 6-3 shows the layout for the Additional Secondary Clarifier progression
- Figure 6-4 shows the facilities layout for the Alum Addition for Year-Round Anaerobic Selector Operation progression, and
- Figure 6-5 shows the layout for the Parallel Stream MBR progression.

Moreover, it is unclear at this time when the District will need to construct nutrient removal facilities and whether it will be necessary to construct facilities to address the secondary clarifier limitations before addressing the nutrient removal issues. For example, if the District moves forward with a potable reuse project in the next few years, nitrification could be needed to support this operation, thus pushing nitrogen removal timeline ahead of the anticipated regulatory timeline. Additional discussion regarding the recommended approach for implementing the project needed to address the secondary clarification issues in concert with making necessary nutrient removal upgrades is provided in Chapter 10 (Recommended Plan).

6.6.5 Lifecycle Costs

The costs associated with upgrading the plant to meet the nutrient removal targets will also depend on what project (if any) is constructed to address the secondary clarifier capacity limitations. Thus, three different sets of capital and operating cost estimates were developed for the nutrient upgrades, as follows:

- Table 6-16 provides the estimated costs to upgrade the plant to meet the Potable Reuse Conditioning, BACWA Level 2 and BACWA Level 3 requirements assuming an alum addition facility is installed as an earlier project.
- Table 6-17 provides the estimated costs to upgrade the plant to meet the Potable Reuse Conditioning, BACWA Level 2 and BACWA Level 3 requirements assuming a secondary clarifier is installed as an earlier project.
- Table 6-18 provides the estimated costs to upgrade the plant to meet the Potable Reuse Conditioning, BACWA Level 2 and BACWA Level 3 requirements assuming a parallel stream MBR is installed as an earlier project.

Details regarding these cost estimates are provided in Appendices H-5-i, H-5-ii, and H--iii, respectively. As shown in these tables, the money that would be saved by choosing the alum addition project as an earlier phase (to address the secondary clarifier capacity issues) would ultimately need to be spent to meet the nutrient removal objectives.

Table 6-16. Estimated Cost for Additional Nutrient Removal Facilities for the Additional Secondary Clarifier Progression

Project Component	Additional Secondary Clarifier[a], million $	Estimated Additional Costs, million $		
		Potable Reuse Conditioning	BACWA Level 2	BACWA Level 3
Secondary Clarifier	9.1	-	-	-
Alum Addition at FSLs	-	-	0.5	0.5
Aeration Basins	-	13.1	17.6	24.4
MLE/IFAS Air Supply	-	2.7	2.7	2.7
IFAS Media	-	-	-	5.7
Methanol Addition	-	-	-	1.4
Chlorination System Improvements	-	0.1	0.1	-
High Strength Return Stream Treatment	-	-	-	10.1
Tertiary Facility Modifications	-	-	-	4.2
OPCC	9.1	15.9	20.9	49.0
Construction Contingency, 10 %	0.9	1.6	2.1	4.9
Total Estimated Construction Cost	10.0	17.5	23.0	53.9
Engineering, Legal and Administrative Costs, 35%[b]	3.2	5.6	7.3	17.2
Total Project Costs	13.2	23.1	30.3	71.1
Total Project Costs with Initial Project		**36.3**	**43.5**	**84.3**

[a] From Table 6-9.
[b] Calculated as a percentage of the OPCC

Table 6-17. Estimated Cost for Additional Nutrient Removal Facilities for the Alum Addition for Year-Round Anaerobic Selector Operation Progression

Project Component	Alum Addition for Year-Round Anaerobic Selector Operation[a], million $	Estimated Additional Costs, million $		
		Potable Reuse Conditioning	BACWA Level 2	BACWA Level 3
Secondary Clarifier	-	9.1	9.1	9.1
Alum Addition at FSLs	0.5	[b]	-	-
Aeration Basins	-	13.1	17.6	24.4
MLE/IFAS Air Supply	-	2.7	2.7	2.7
IFAS Media	-	-	-	5.7
Methanol Addition	-	-	-	1.4
Chlorination System Improvements	-	0.1	0.1	-
High Strength Return Stream Treatment	-	-	-	10.1
Tertiary Facility Modifications	-	-	-	4.2
OPCC	0.5	25.0	29.5	57.6
Construction Contingency, 10%	0.05	2.5	3.0	5.8
Total Estimated Construction Cost	0.6	27.5	32.5	63.4
Engineering, Legal and Administrative Costs, 35%[c]	0.2	8.8	10.3	20.2
Total Project Costs	0.8	36.3	42.8	83.6
Total Project Costs with Initial Project		**37.1**	**43.6**	**84.4**

[a] From Table 6-9.
[b] Alum addition facilities would be a stranded asset under this scenario.
[c] Calculated as a percentage of the OPCC

Table 6-18. Estimated Cost for Additional Nutrient Removal Facilities for the Parallel Stream MBR Progression

Project Compenent	Parallel Stream MBR[a], million $	Estimated Additional Costs, million $		
		Potable Reuse Conditioning	BACWA Level 2	BACWA Level 3
Secondary Clarifier	-	-	-	-
Alum Addition at FSLs	-	-	0.5	0.5
MBR Facilities	35.0	-	-	-
Aeration Basins	-	8.5	13.1	21.0
Methanol Addition	-	-	-	1.4
Chlorination System Improvements	-	0.1	0.1	-
High Strength Return Stream Treatment	-	-	-	10.1
Tertiary Facility Modifications	-	-	-	4.12
OPCC	35.0	8.6	13.7	37.2
Construction Contingency, 10%	3.5	0.9	1.4	3.7
Total Estimated Construction Cost	38.5	9.5	15.1	40.9
Engineering, Legal and Administrative Costs, 35%[b]	12.3	3.0	4.8	13.0
Total Project Costs	50.8	12.5	19.9	53.9
Total Project Costs with Initial Project		**63.3**	**70.7**	**104.7**

[a] From Table 6-9.
[b] Calculated as a percentage of the OPCC

Annual operation costs for each of the nutrient removal scenarios are dependent on whether a parallel stream MBR is being used to accomplish a portion of the wastewater treatment. (Note that the parallel stream MBR would meet BACWA Level 3 standards). A summary of the operating costs with and without the parallel stream MBR are shown in Table 6-19. As shown, the annual operating costs are slightly greater with the parallel stream MBR process. A more detailed breakdown of these operating costs are provided in Appendix I-3.

Finally, the estimated lifecycle costs for upgrading the WWTP to achieve the range of nutrient removal requirements identified for this Master Plan, grouped based on what project is assumed to be implemented to address the secondary clarifier capacity issues, is shown in Table 6-20. As shown, the parallel stream MBR option is significantly more expensive than the other two alternatives. However, the differences in costs are reduced as the treatment level increased. This is partially due to the fact that that MBR process will achieve BACWA Level 3 standards. As discussed in Chapter 7 (Potable Reuse Alternatives Evaluation), there could be substantial economic benefits associated with the parallel MBR process under a future potable reuse treatment scenario. These benefits should therefore be considered as part of the economic evaluation of these alternatives.

Table 6-19. Annual Operating Costs for Nutrient Removal Facilities, million $

Operating Cost Component	Potable Water Reuse		BACWA Level 2		BACWA Level 3	
	Without MBR	With Parallel Stream MBR	Without MBR	With Parallel Stream MBR	Without MBR	With Parallel Stream MBR
Chemical	0	0.05	0.07	0.1	2.4	2.3
Electrical	0.9	1.2	1.0	1.2	1.2	1.3
Labor	1.1	1.3	1.1	1.4	1.2	1.4
Total Annual Cost	**2.0**	**2.55**	**2.2**	**2.7**	**4.8**	**5.0**

Table 6-20. Lifecycle Costs of Additional Nutrient Removal Facilities, million $

Cost Component	Potable Water Reuse	BACWA Level 2	BACWA Level 3
Additional Secondary Clarifier Progression			
Total Project Costs	36.3	43.5	84.3
NPV of Annual Cost	42.9	47.2	102.9
Lifecycle Costs	**79.2**	**90.7**	**187.2**
Alum Addition for Year-Round Anaerobic Selector Operation Progression			
Total Project Costs	37.1	43.6	84.4
NPV of Annual Cost	42.9	47.2	102.9
Lifecycle Costs	**80**	**90.8**	**187.3**
Parallel Stream MBR Progression			
Total Project Costs	63.2	70.7	104.7
NPV of Annual Cost	54.7	57.9	107.2
Lifecycle Costs	**118**	**128.6**	**211.9**

6.7 REFERENCES

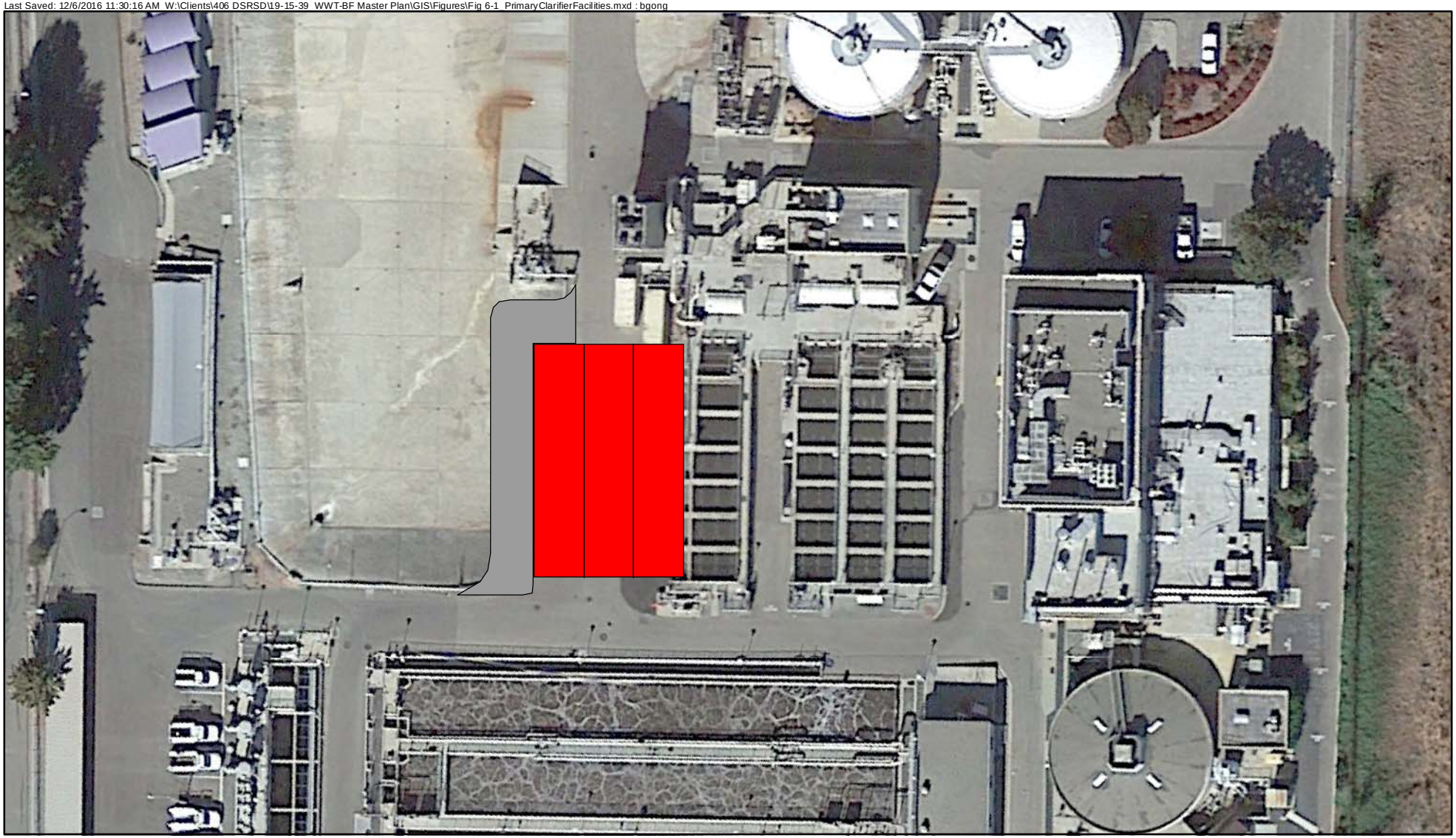
Brown and Caldwell (2004) *Plant Process Capacity Study: Dry Weather Capacity Final Report*, prepared for Dublin San Ramon Services District, December 2004

Daigger, G.T., (1995) *Development of Refined Clarifier Operating Diagrams Using an Updated Settling Characteristics Database.* Water Environment Research, Vol. 67, No. 1, p. 95, January/February 1995.

Smith, D. S., Takács, I., Murthy, S., Daigger, G.T., and Szabo, A. (2008) *Phosphate Complexation Model and its Implications for Chemical Phosphorous Removal*. Wat Environ. Res., 80(5):428-438.

RMC, (2014). *Technical Memorandum: Secondary Effluent Wet Weather Capacity Evaluation*. prepared for Dublin San Ramon Services District, August 2014

Last Saved: 12/6/2016 11:30:16 AM W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\GIS\Figures\Fig 6-1_PrimaryClarifierFacilities.mxd : bgong



Symbology

- Primary Sedimentation Basin
- Additional Paved Area

0 30 60

Scale in Feet

WEST YOST ASSOCIATES

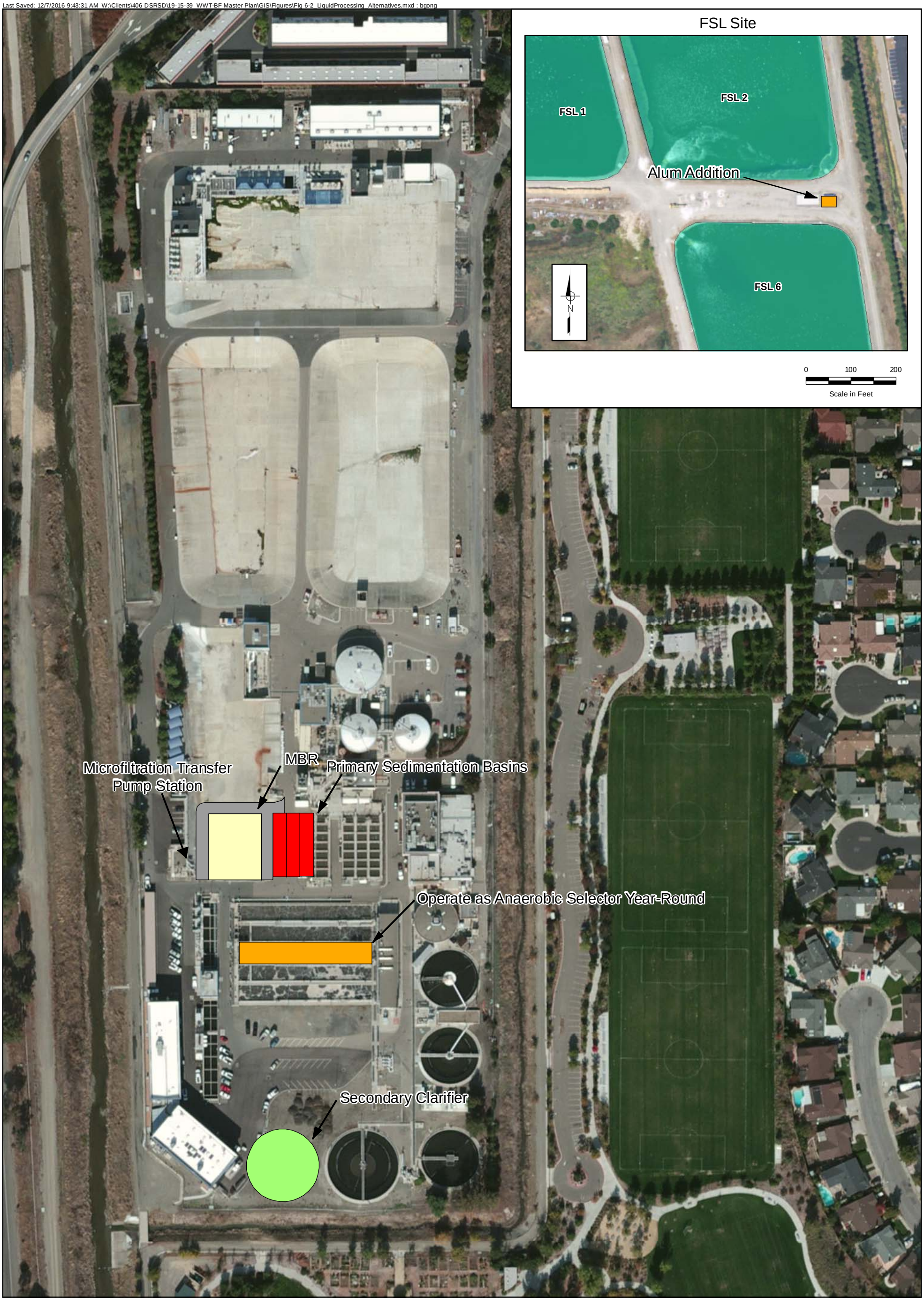
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Figure 6-1

Conceptual Layout of Primary Sedimentation Basin Facility Expansion

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- Primary Sedimentation Basin Expansion
- Alternative A: Additional Secondary Clarifier
- Alternative B: Alum Addition for Year-Round Anaerobic Selector Operation
- Alternative C: Parallel Flow MBR
- Additional Paved Area

N

0 70 140

Scale in Feet

WEST YOST ASSOCIATES

HDR

Figure 6-2

Alternatives for Addressing Secondary Clarifier Capacity Limitations

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Connect to Existing Outfall Pipeline

Tertiary Facility Modifications (Modify Pumping Station to Allow for Year-Round Filtration and Split Flow Discharge)

Ballasted Flocculation (Under Construction)

FSL Site



Methanol Addition

Primary Sedimentation Basins

Mixed Liquor Return Channel

Aeration Basins

Secondary Clarifier

Symbology

- Facilities Required to Meet Current Treatment Requirements
- Facilities Added to Provide Potable Reuse Conditioning
- Facilities Added to Provide BACWA Level 2
- Facilities Added to Provide BACWA Level 3
- Additional Paved Area

N

0 70 140

Scale in Feet

WEST YOST ASSOCIATES

HDR

Figure 6-3

Additional Secondary Clarifier Progression

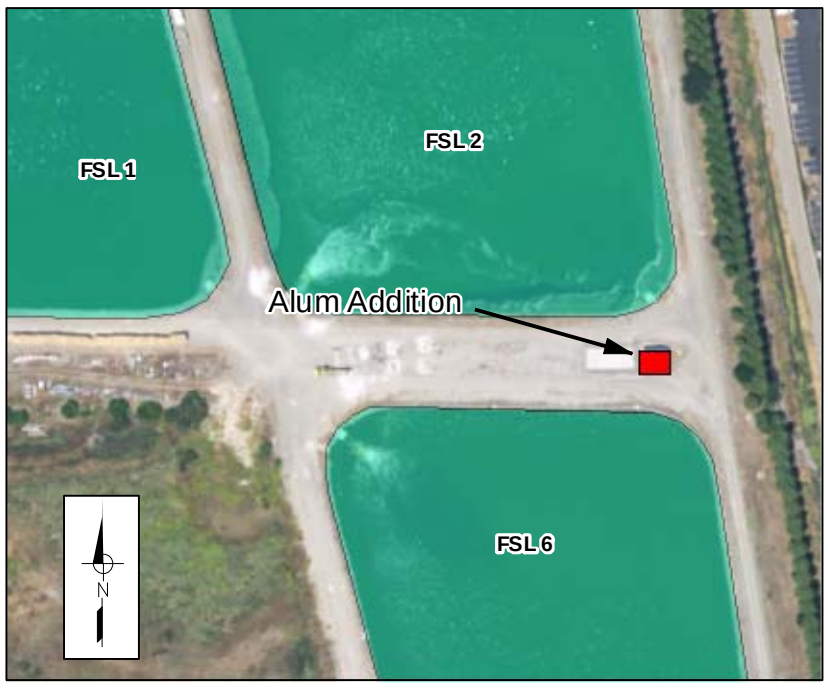
Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

Connect to Existing Outfall Pipeline

Tertiary Facility Modifications (Modify Pumping Station to Allow for Year-Round Filtration and Split Flow Discharge)

Ballasted Flocculation (Under Construction)

FSL Site



Methanol Addition

Primary Sedimentation Basins

Mixed Liquor Return Channel

Aeration Basins

Secondary Clarifier

Symbology

- Facilities Required to Meet Current Treatment Requirements
- Facilities Added to Provide Potable Reuse Conditioning
- Facilities Added to Provide BACWA Level 2
- Facilities Added to Provide BACWA Level 3
- Additional Paved Area

N

0 70 140

Scale in Feet

WEST YOST ASSOCIATES

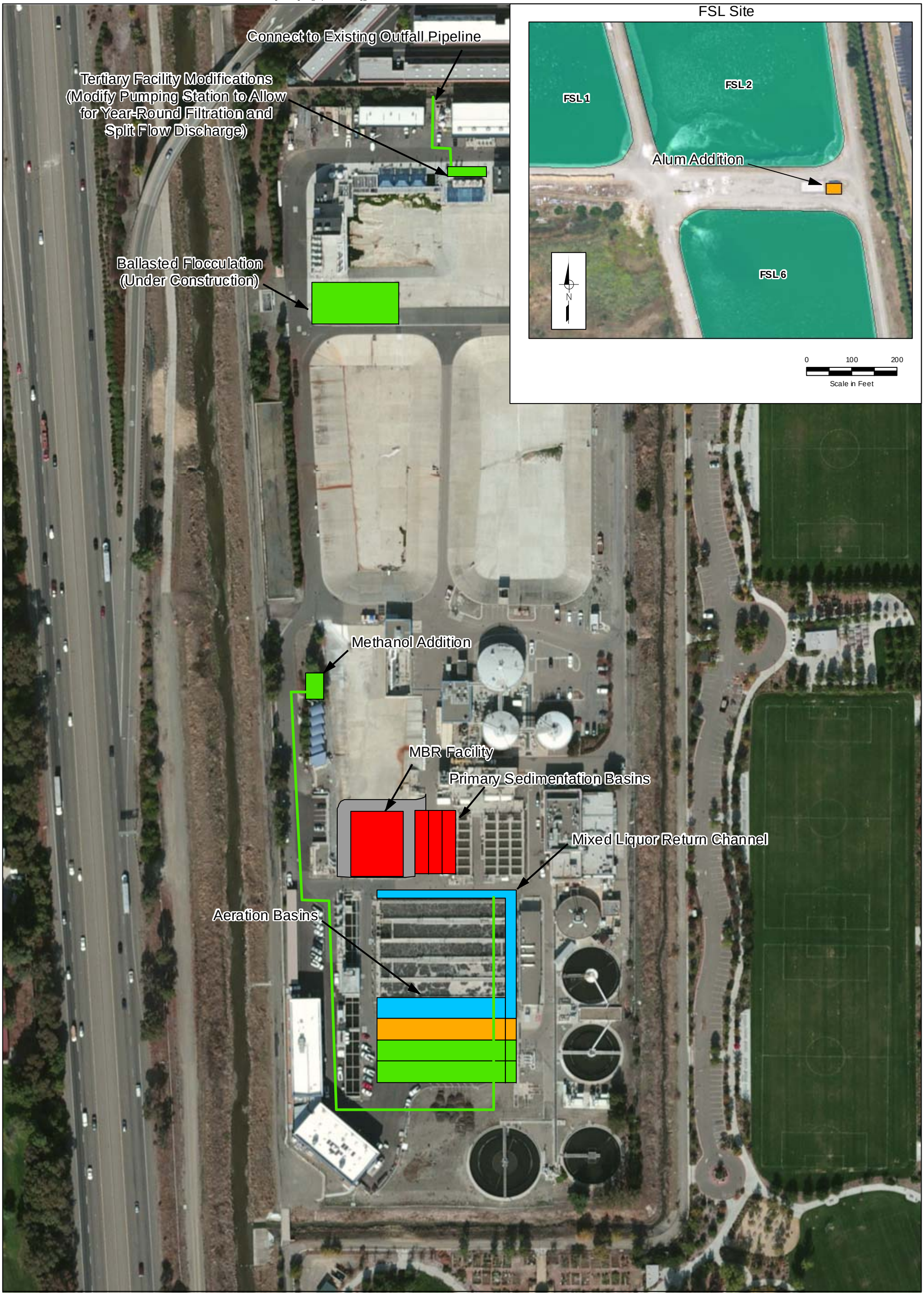
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Figure 6-4

Alum Addition for Year-Round Anaerobic Selector Operation Progression

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- Facilities Required to Meet Current Treatment Requirements
- Facilities Added to Provide Potable Reuse Conditioning
- Facilities Added to Provide BACWA Level 2
- Facilities Added to Provide BACWA Level 3
- Additional Paved Area

N

0 70 140

Scale in Feet

WEST YOST ASSOCIATES

HDR

Figure 6-5

Parallel Flow MBR Progression

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

CHAPTER 7
Potable Reuse Treatment Evaluation

This Chapter presents an evaluation of the treatment facilities that could be required at the WWTP to provide for a potential future potable reuse project that relies on effluent from the District's facilities. The specific topics addressed in this section include:

- Evaluation Methodology
- 3.0 MGD Full Advanced Treatment (FAT) Facilities
- Pretreatment Alternatives for 3.0 MGD FAT Facilities
- 3.0 MGD Raw Water/Treated Water Augmentation Treatment Facilities
- 9.0 mgd Raw Water/Treated Water Augmentation Treatment Facilities

As noted previously, a series of Visioning Panel Workshops were conducted to bring together external experts to provide direction regarding potential future opportunities and constraints for the WWTP, and a key subject area of those discussions was potable reuse treatment. A description of each workshop and what was covered is provided in **Appendix E**.

7.1 EVALUATION METHODOLOGY

This section presents an overview of the approach applied when evaluating the sizing and capacity of the potential future potable reuse treatment alternatives under this Master Plan. The following specific topics are addressed:

- Evaluation Objectives
- Capacity Requirements
- Potable Reuse Classification in California
- Potable Reuse Regulations in California

7.1.1 Evaluation Objectives

The two objectives of this potable reuse treatment facilities evaluation are as follows:

- **Identify Improvements Needed to Rehabilitate the CWR Treatment Facilities for Potable Reuse:** The CWR project included MF, RO, and UV facilities, which were designed to produce water suitable for potable reuse through groundwater augmentation (although statewide regulations were not defined at the time). The existing CWR MF/UV facilities are currently used seasonally, and/or as backup facilities for the District's larger capacity sand filtration and associated UV recycled water facilities. The CWR RO facility has been decommissioned and is no longer in use. (See Chapter 3 for further details.) The District could potentially recuperate some of the investment made in the CWR facilities by rehabilitating them to allow for potable reuse treatment at the WWTP. Therefore, one objective of this evaluation is to identify the approximate cost, size, location, and layout of treatment facilities needed to allow for different levels of potable reuse by rehabilitating the CWR reuse treatment system.

- **Identify Treatment Facilities Needed to Increase Potable Reuse Capacity to Minimize Discharges to the Bay:** The District's 2016 Strategic Plan identified the elimination of discharge to the Bay 365 days per year as a specific goal. An evaluation of the District's anticipated Bay discharges, with currently planned reuse projects, is provided in **Appendix C**. As detailed, the District has plans for reusing the majority of treated effluent for irrigation purposes during the summer months, but flows are available during the months when irrigation demands are low. Thus, a second objective of this effort is to define the approximate cost, size, location, and layout of the treatment facilities needed to allow for different levels of potable reuse of the majority of the District's available treated wastewater.

7.1.2 Capacity Requirements

The CWR system was originally designed to produce 3.0 mgd of high quality treated effluent for potable reuse purposes. Part of this evaluation is to determine what would be required to bring these facilities to current standards for potable reuse. Thus, a 3.0 mgd treatment capacity is assumed as the first capacity level of interest.

An evaluation of average year and wet-weather year storage needs for the WWTP is detailed in **Appendix C**. As part of that evaluation, an optimum level of recycled water production has been identified to maximize reuse. A recycled water capacity of about 9 mgd is recommended to use the majority of available flow. The District would still need to discharge some peak wet-weather flows. However, a recycled water capacity greater than this breakpoint would not likely be economical given the limited number of days that the added capacity would be used. For these reasons, the second capacity level of interest is 9.0 mgd.

7.1.3 Potable Reuse Classification in California

Potable reuse involves the use of highly treated wastewater to augment a drinking water supply. In California, potable reuse systems are generally classified with regards to extent of an environmental or other physical barrier between the potable reuse treatment system and the potable reuse activity. Currently, regulations are being developed to define potable reuse by the following four classifications:

- **Groundwater Augmentation**: where a potable reuse water supply is added to groundwater either via surface application (spreading) or subsurface application (direct injection),
- **Reservoir Augmentation**: where a potable reuse water supply is added to a reservoir that is used as a drinking water supply,
- **Raw Water Augmentation**: where a potable reuse water supply is blended with other raw water supplies and sent directly to a potable water treatment plant, and
- **Treated Water Augmentation**: where a potable reuse water supply is discharged directly to a potable water distribution system.

The evaluation detailed in this Chapter is not intended to identify what type of potable reuse classification that the District should implement. Moreover, in parallel to this Master Plan, the District is participating in a separate study to evaluate potable reuse on a region-wide basis. As further discussed in Chapter 10, the District will need to consider the results from both this Master Plan and the current regional study before confirming a preferred approach to potable reuse.

7.1.4 Potable Reuse Regulations in California

Potable reuse regulations are rapidly evolving in California. An overview of the current status of regulatory requirements applicable to each of the four types of potable reuse projects discussed above is provided on Figure 7-1. As shown, allowing for a more direct path from treated effluent into a potable water supply requires increasing levels of treatment. Additional details regarding what is known about current potable reuse regulations as the they pertain to the potential treatment improvements at the WWTP are provided below.

7.1.4.1 Groundwater Augmentation

The State Water Board Division of Drinking Water (DDW) adopted regulations for groundwater recharge in 2014. These requirements specify treatment standards to ensure adequate removal of organic and inorganic contaminants found in secondary effluent, and removal/inactivation of pathogens. The pathogen reduction requirements for groundwater augmentation are defined in terms of orders of magnitude (logarithm base 10, or "log") reduction/inactivation of organisms, as follows:

- 12-log reduction of enteric viruses
- 10-log reduction of *Giardia* cysts
- 10-log reduction of *Cryptosporidium* oocysts

In addition to the above standards, California regulations dictate that a groundwater augmentation treatment system include multiple barriers to provide resiliency against failure of individual barriers, and that at least three separate treatment processes are required. These regulations further specify a FAT process to achieve the treatment goals, as follows:

- **Primary treatment and a secondary (oxidation) processes:** This treatment is met by the existing WWTP.
- **Membrane Filtration:** Membrane filtration by either MF or UF provides a high level of particle removal filtration upstream of the RO process. The distinction between MF and UF is related to the pore size of the membranes, with MF membranes having larger, nominal 1 micrometer (micron), pore size, and UF membranes having a nominal 0.1 micron pore size.
- **RO Process:** RO is a separation process similar in concept to the membrane filtration process, but it involves removal of smaller pollutants, including dissolved organic compounds, and salts that typically pass through membrane filers. Because RO membranes reject a large fraction of dissolved pollutants, another critical difference is that this process results in a concentrated brine reject stream.

- **Advanced Oxidation Process (AOP):** AOP breaks down complex organic compounds with strong oxidizing chemicals. The most common AOP oxidants used for wastewater treatment are hydroxyl radicals. Hydroxyl radical(s) are typically produced by adding ozone to water with a high pH, by adding ozone to water that is also exposed to UV light energy, by adding hydrogen peroxide to water that is also exposed to UV light energy, or by adding ozone to water in combination with hydrogen peroxide.

7.1.4.2 Reservoir Augmentation

Regulations are currently under development for reservoir augmentation, and are expected to be adopted within the next year. Exact requirements of raw water/treated water augmentation are not known at this time. However, several agencies in California are moving forward with reservoir augmentation projects, and DDW has provided guidance on the regulatory expectations for these types of projects.

It is currently understood that additional organics removal and pathogen log reduction/inactivation will need to be achieved beyond the FAT process. The most advanced project to date, which is being developed in San Diego, includes an ozonation plus biologically active carbon (BAC) process in addition to the FAT process.

7.1.4.3 Raw Water/Treatment Water Augmentation

The feasibility of developing regulations for raw water/treated water augmentation was recently reviewed by an expert panel for the State Water Board, and a feasibility report was issued in December 2016. This feasibility report indicates that developing regulations for raw/treated water augmentation is feasible from a technical standpoint. Therefore, the State Water Board is expected to move forward with developing regulations for raw/treated water augmentation. However, the exact nature and timing of these regulations are currently uncertain.

Potential requirements for a future raw/treated water augmentation were discussed as part of the Visioning Panel process, and the following conclusions were made:

- Requirements to meet specific log reduction/inactivation levels will likely increase as the barrier between the potable reuse water supply source and the reuse system becomes smaller.
- Exact requirements of raw/treated water augmentation cannot be defined at this time. However, it is expected that treatment beyond the FAT process with ozonation plus BAC will be required.
- Additional microbial log reduction/inactivation could be achieved through increased disinfection (either chlorine or UV).
- Pretreatment processes such as conventional NDN or MBR may also be considered for log removal credit.

- Additional monitoring requirements are expected, and, depending on what parameters need to be monitored, it may be necessary to provide short-term storage of treated recycled water for 8 to 24 hours to provide time for analysis. Multiple storage tanks would be needed to permit storing treated product water in one tank while samples are being analyzed while the other tank is being emptied for potable reuse (if monitoring results are favorable).
- The timing of when raw/treated water augmentation regulations will be adopted by DDW is currently not known, but it is likely that demonstration projects will be developed in concert with development of the regulations, similar to the previously adopted groundwater augmentation regulations or to the current reservoir augmentation regulations.
- In the absence of state-wide regulations, potable reuse projects can move forward with site-specific discharge permits. Based on the history of recycled water regulations in California, the regulatory staff is expected to be become more accepting of potable reuse projects as individual projects move forward.
- There are current efforts in the potable water reuse industry to identify combinations of water treatment processes that do not include RO filtration to achieve the log reduction/inactivation goals. If a justification for a change in current DDW policy regarding the need to include RO filtration is accepted, this change could significantly reduce capital and operating costs associated with potable reuse.

7.2 3.0 MGD FAT FACILITIES

This section presents an evaluation of the facilities required to upgrade/improve the 3.0 mgd capacity CWR facilities to meet the definition of a FAT facility. The topics addressed include:

- Strategy for Rehabilitation of CWR Facilities to Satisfy FAT Requirements
- Facilities Overview
- Facilities Layout
- Lifecycle Costs

7.2.1 Strategy for Rehabilitation of CWR Facilities to Satisfy FAT Requirements

One of the overall objectives of the potable reuse evaluation is to identify improvements needed to rehabilitate and upgrade the CWR facilities to allow for potable reuse. Based on the current information available regarding potential future potable reuse regulations, a FAT system is the backbone process that would be required for any potable reuse project. Therefore, a key objective of this evaluation is to define what improvements would be needed to upgrade the CWR system to meet the criteria of the FAT process.

It should be noted that this evaluation is not focused on determining the most cost-effective technologies to achieve the FAT process. Instead, typical FAT technologies are evaluated with the assumption that the District would further evaluate specific technologies and approaches in a later planning phase.

7.2.2 Facilities Overview

A schematic of the typical configuration for a FAT system downstream of secondary treatment is provided on Figure 7-2. As shown, the primary treatment components of a FAT process include MF/UF membrane filtration, RO, and AOP. The CWR treatment system includes an MF, RO and UV disinfection process. This section addresses the CWR system improvements needed to meet the requirements that have been established for a FAT process.

A detailed assessment of the overall rehabilitation needs for the CWR systems was beyond the scope of this Master Plan. Therefore, this analysis includes the following assumptions with respect to upgrading the CWR facilities to meet the FAT treatment process requirements:

- The primary and secondary processes at the WWTP meet the basic pretreatment requirements for the FAT process, and were adequately addressed in Chapter 6.
- The existing MF Transfer Pump Station and 14-inch diameter discharge pipeline would continue to be used to convey secondary effluent to the CWR's MF/RO Building.
- CWR's existing MF membranes are near the end of their expected service life and thus need replacement.

- The RO membrane housing units currently do not have RO elements installed, but the supporting RO system infrastructure is still there. However, the supporting infrastructure has been in place for over 20 years and has not been used for most of this time. Therefore, it is assumed that new RO membranes would be installed in the existing modules, and the associated pump station, pipes, valves, and accessories would be rehabilitated or replaced.
- The original UV disinfection system is not suitable for AOP service. Therefore, a new AOP system is required. For this Master Plan, it is assumed that a combined ozone and UV process will be used, referred to herein as a "UV/AOP" system. The system would include an ozone process and a closed closed-vessel UV system.
- The RO brine stream would be discharged to the LAVWMA outfall.

In addition to the above assumptions, an additional 20 percent estimating contingency has been applied to the MF and RO rehabilitation costs to account for the significant number of uncertainties associated with the CWR rehabilitation. Thus, the total estimating contingency applied to the evaluation of the CWR rehabilitation cost is 50 percent.

It is also likely that the existing Product Water Pump Station, which is currently not in use, could be repurposed for distributing the treated potable reuse water to its ultimate discharge location. However, specific costs for modifications to the Product Water Pump Station have not been evaluated because the final location for effluent intended for potable reuse has not been established.

7.2.3 Facilities Layout

The proposed layout of the facilities described above is shown on Figure 7-3. As shown, the only completely new component is the UV/AOP system and connecting pipelines. This size of the AOP system is relatively small, and it could be located within the footprint of some of the parking spaces adjacent to Holding Basin 4.

7.2.4 Lifecycle Costs

A summary of the capital costs for the 3.0 mgd FAT facilities is provided in Table 7-1. These costs were developed following the procedures detailed in Chapter 5, and cost estimating details are provided in **Appendix H-6**. Annual operation costs and NPV costs for these facilities are shown in Tables 7-2 and 7-3, respectively. A more detailed breakdown of the operating costs is provided in **Appendix I-4**. Table 7-3 also shows the total present worth value of the 3.0 mgd FAT facilities and the estimated cost per acre-foot of recycled water produced, assuming the system is operated between October 15 and April 30 of each year.

Table 7-1. Estimated Cost for 3.0 mgd FAT Facilities

Project Component	Estimated Cost, million $
MF Membrane System	3.8
RO Membrane System	8.8
UV/AOP System	2.3
OPCC	**14.9**
Construction Contingency, 10%	1.5
Total Estimated Construction Cost	**16.4**
Engineering, Legal and Administrative Costs, 35%[(a)]	5.2
Total Project Costs	**21.6**

[(a)] Calculated as a percentage of the OPCC

Table 7-2. Annual Operating Costs for 3.0 mgd FAT Facilities

Type	Annual Cost, million $
Chemical	0.4
Electrical	0.4
Labor	0.1
Sampling and Analytical	0.1
Replacements (e.g., UV lamps, membrane modules)	0.1
Total Annual Cost	**1.1**

Table 7-3. Summary of Net Present Value for 3.0 mgd FAT Facilities[(a)]

Project Component	Estimated Cost, million $
Total Capital Costs	21.6
Total Annual Costs (NPV)	24.4
Total Present Worth Value (Lifecycle Cost)	**46.0**
Lifecycle Cost per Acre-Foot Delivered[(a)]	900

[(a)] Unit lifecycle costs are based on a 30-year present worth value, and an estimated production volume of 591 Mgal (or 1,800 acre-feet) per year for 30 years.

7.3 PRETREATMENT ALTERNATIVES FOR 3.0 MGD FAT FACILITIES

Pretreatment beyond the conventional activated sludge secondary treatment process required by current regulations for groundwater recharge has been shown to improve performance of the MF system, and could result in an overall lower cost. In addition, as discussed previously in this Chapter, treatment beyond the FAT process is expected to be necessary for more advanced potable reuse scenarios, like reservoir augmentation. Thus, a pretreatment process installed to improve MF performance could potentially satisfy the additional treatment requirements for an advanced potable reuse projects. This section details the following with regards to alternatives for pretreatment upstream of the FAT process:

- Strategy for Evaluating FAT Process Pretreatment Facilities
- Alternatives Overview
- Comparison of Alternatives

7.3.1 Strategy for Evaluating FAT Process Pretreatment Facilities

The purpose of a FAT process pretreatment system is to further oxidize organic compounds that tend to foul MF and UF membranes. With pretreatment, the permeate flux through the membranes is enhanced, and the operating duration between clean-in-place tasks is increased making the facilities easier to operate. Therefore, pretreatment can result in fewer membrane treatment trains to achieve the same treatment capacity, and can lower electrical, chemical, and maintenance costs. Thus, to assess the benefits of a pretreatment system, these cost savings must be quantified.

The following technologies are considered in the Master Plan for pretreatment upstream of the FAT process:

- Ozonation
- Ozonation plus BAC
- Nitrification/Denitrification (NDN)

The estimated MF/UF membrane flux rates without pretreatment and with these potential pretreatment options considered herein are presented in Table 7-4. The flux values shown in Table 7-4 were used herein to size the MF system for each alternative, where fewer MF membrane modules are required with higher flux rates and the number of modules assumed affects both the capital and operating cost for a given alternative.

Table 7-4. Estimated MF/UF Membrane Flux by Pretreatment Method

Pretreatment Method	MF/UF Membrane Flux, gpd/square feet
None (non-nitrified secondary effluent)	17
Ozonation	21[(a)]
Ozonation plus BAC	28
Nitrogen/Denitrification (NDN)	21[(b)]

(a) Information is not available on enhanced flux of MF/UF with pretreatment by ozonation (without BAC). The flux with pretreatment by these methods is assumed to be higher than with no pretreatment. The lowest flux with pretreatment, that for conventional NDN, is shown, and has been used for cost estimating purposes.

(b) Assumed to include conventional NDN and MBR processes.

7.3.2 Alternatives Overview

This section provides an overview of the following FAT facilities pretreatment alternatives considered under this Master Plan:

- No additional pretreatment
- Ozonation
- Ozonation plus BAC
- Conventional (full-scale) NDN
- Parallel stream MBR (for NDN of only the portion of flow being sent to the potable reuse treatment facilities)

Schematics of the FAT systems that include each of these pretreatment options are shown on Figure 7-4. These schematics also include flow at different points in the treatment train, based on a nominal net 3.0 mgd production capacity for the system. Each of these alternatives is further discussed below.

7.3.2.1 No Additional Pretreatment

As noted previously, there is no requirement for additional pretreatment of flows upstream of the FAT process for groundwater augmentation. However, without pretreatment, the performance of the MF membranes will likely be compromised, net production diminished, and the overall system maintenance will be more extensive than if pretreatment were provided. In addition, if more advanced potable reuse beyond groundwater augmentation is desired, additional pretreatment is expected to be necessary.

7.3.2.2 Ozonation

Ozonation has been demonstrated to be an effective oxidizing process upstream of the FAT process to improve MF operations. The major structural components of an ozonation system include a cryogenic liquid oxygen (LOX) storage tank, a LOX-gas vaporization system for gaseous oxygen (GOX) delivery, ozone generators and associated power supply units, an ozone contactor (the treatment unit), and an off-gas ozone destruction system. The ozone generator use a high-voltage current to create ozone from GOX. The ozone gas is then added to the wastewater stream in the contactor to react with the organic molecules and other contaminants. Afterward, the contactor off-gas system uses a catalyst to convert the remaining ozone back to oxygen gas.

7.3.2.3 Ozonation plus BAC

Removal of organic contaminants can be further enhanced if the ozone process followed by a granular activated carbon (GAC) filtration process. GAC is commonly used to remove organic chemicals and is also effective in removing many oxidation byproducts. Ozonation produces biodegradable organic chemicals, so ozonation upstream of a GAC process results in the formation of a microbial biofilm on the GAC media. This biologically enhanced filtration process is referred to as BAC. In a BAC process, the GAC media will remove larger, more complex of organic compounds, and the biofilm consumes the smaller organic compounds that have been broken down by the ozone process, as well as some of the compounds absorbed by the GAC media.

There is also anecdotal information to suggest that ozonation can produce organic chemicals or oxidation byproducts that degrade polymeric membrane materials used for MF/UF. Therefore, a BAC process that removes the organic chemicals and oxidation byproducts prior to MF/UF filtration can potentially extend the life of the MF/UF membranes.

7.3.2.4 Conventional NDN

The NDN treatment process also provides additional oxidation of wastewater upstream of the FAT process. The facilities required to achieve stable NDN via a conventional process were described under the "Potable Reuse Conditioning" alternative presented in Chapter 6 (Liquid Treatment Process Alternatives Evaluation). As discussed, this alternative would require the addition of two aeration basins, two additional blowers, a mixed liquor return channel, and a new secondary clarifier.

7.3.2.5 Parallel Stream MBR

NDN can also be accomplished via an MBR process. An MBR facility could be installed instead of conventional facilities to achieve NDN and would have two advantages: a smaller footprint and another membrane process to provide additional removal of constituents that could interfere with the downstream membrane processes (and potentially increase removal credit).

The facilities required to provide MBR pretreatment were described under the "Parallel Stream MBR" alternative presented in Chapter 6 (Liquid Treatment Process Alternatives Evaluation). As discussed, a 4-mgd MBR system that treats effluent drawn from the primary sedimentation basins and discharges to the existing MF Transfer Pump Station is assumed.

7.3.3 Comparison of Alternatives

This section presents a comparison of the alternatives discussed above with respect to layout, lifecycles costs, and the non-economic factors. The alternatives identified also provide advantages and disadvantages with respect to potential long-term potable reuse treatment and nutrient removal treatment needs. Therefore, a recommended strategy for pretreatment must consider all these potential future facilities. Thus, a recommended approach to addressing the pretreatment needs is not addressed below, but is instead provided in Chapter 10 (Recommended Plan) of this Master Plan.

7.3.3.1 Facilities Layout

The potential layout of the 3.0 mgd FAT facilities, along with the various facilities required for each of the pretreatment alternatives, is presented on Figure 7-5. The specific locations shown were chosen based on the follow factors:

- There should be adequate space for the ozone system facilities (shown in green) near the existing MF Transfer Pump Station. This facility would include a LOX storage tank, ozone generators and power facilities, ozone gas transfer system and contactor, and the contactor off-gas system facilities.
- A portion of the existing 14-inch diameter MF Feed pipeline (shown in red) could be used to provide a portion of the required ozone contact time.
- A BAC facility (shown in green) could be located at the northwest corner of the main treatment site, near the existing MF/RO building. This location will allow the ozonated effluent to be diverted and treated upstream of the MF/RO facilities.
- Nitrogen removal would require modifications to the existing aeration basins, and expansion of the basins and secondary clarifiers (shown in orange).
- A new parallel stream MBR facility (shown in yellow) would be constructed within a portion of Basin 1, reducing the overall storage capacity provided at the WWTP. The approximate footprint for the MBR facility is 10,000 square feet.

7.3.3.2 Lifecycle Costs

A summary of the capital costs for each of the 3.0 mgd FAT facilities pretreatment alternatives is provided in Table 7-5. These costs were developed following the procedures detailed in Chapter 5, and cost estimating details are provided in **Appendix H-7**. Annual operation costs and NPV costs for each of these alternatives are shown in Tables 7-6 and 7-7, respectively. Unit costs per acre-foot of delivered water are also included in Table 7-7. These unit costs are useful for comparison with alternative water supplies. A more detailed breakdown of these operating costs is provided in **Appendix I-4**.

Table 7-5. Estimated Cost for 3.0 mgd FAT Facilities Pretreatment Alternatives

Project Component	Estimated Costs by Pretreatment Alternative, million $				
	No Pretreatment[a]	Ozonation	Ozonation plus BAC	Full NDN	Parallel Stream MBR
MF Membrane System	3.8	1.8	1.6	1.8	1.8
RO Membrane System	8.8	8.8	8.8	8.8	8.8
UV/AOP System	2.3	2.3	2.3	2.3	2.3
Ozone Pretreatment	-	7.4	7.4	-	-
BAC Filter Pretreatment	-	-	4.4	-	-
Full-Scale NDN	-	-	-	25.0	-
MBR Treatment System	-	-	-	-	35.0
OPCC	**14.9**	**20.3**	**24.5**	**37.9**	**47.9**
Construction Contingency, 10%	1.5	2.0	2.5	3.8	4.8
Total Estimated Construction Cost	**16.4**	**22.3**	**27.0**	**41.7**	**52.7**
Engineering, Legal and Administrative Costs, 35%[a]	5.2	7.1	8.6	13.3	16.8
Total Project Costs	**21.6**	**29.4**	**35.6**	**55.0**	**69.5**

[a] From Table 7-1
[b] Calculated as a percentage of the OPCC

Table 7-6. Annual Operating Costs for 3.0 mgd FAT Facilities Pretreatment Alternatives

Type	Annual Cost by Pretreatment Alternative, million $				
	No Pretreatment	Ozonation	Ozonation plus BAC	Full NDN	Parallel Stream MBR
Chemical	0.4	0.4	0.4	0.4	0.4
Electrical	0.4	0.5	0.5	1.3	1.5
Labor	0.1	0.2	0.2	1.2	1.3
Sampling and Analytical	0.1	0.1	0.1	0.1	0.1
Replacements (e.g., UV lamps, membrane modules)	0.1	0.1	0.1	0.1	0.1
Total Annual Cost	**1.1**	**1.3**	**1.3**	**3.1**	**3.4**

Table 7-7. Summary of Net Present Value for 3.0 mgd FAT Facilities Pretreatment Alternatives

Project Component	Estimated Cost by Pretreatment Alternative, million $				
	No Pretreatment	Ozonation	Ozonation plus BAC	Full NDN	Parallel Stream MBR
Total Capital Costs	21.6	29.4	35.6	55.0	69.5
Total Annual Costs (NPV)	24.4	26.2	27.4	67.5	71.9
Total Present Worth Value (Lifecycle Cost)	**46.0**	**55.6**	**63.0**	**122.5**	**141.4**
Lifecycle Cost per Acre-Foot Delivered[(a)]	900	1,000	1,200	2,200	2,600

[(a)] Unit lifecycle costs are based on a 30-year present worth value, and an estimated production volume of 591 Mgal (or 1,800 acre-feet) per year for 30 years.

As shown in the tables above, the parallel stream MBR process would entail the most expensive capital costs, and conventional NDN would also entail relatively large capital investment. However, as discussed in Chapter 6, there could be substantial economic benefits associated with the NDN alternatives under a future nutrient removal scenario. These benefits should therefore be considered as part of the economic evaluation of these alternatives.

7.3.3.3 Non-Economic Comparison

The non-economic criteria discussed in Chapter 5 can be used to compare the pretreatment alternatives. Subjective rankings are presented in Table 7-8. (Not all of the ranking categories discussed in Chapter 5 are relevant, as noted in the table.)

Table 7-8. Non-Economic Comparison of Pretreatment Alternatives Upstream of FAT Process

Criteria	Weighted Subjective Criteria				
	No Pretreatment	Ozonation	Ozonation plus BAC	Full NDN	Parallel Stream MBR
Reliability	4.0	4.3	4.3	5.0	5.3
Flexibility	6.4	5.8	5.8	3.5	5.3
Odor generation potential	(a)	(a)	(a)	(a)	(a)
Maximize Available Water Storage Volume	3.8	3.8	3.8	3.8	2.5
Energy Management	(a)	(a)	(a)	(a)	(a)
Chemical use	2.0	2.0	2.0	3.0	2.0
Resource Recovery	1.5	2.3	3.0	2.3	2.3
Ancillary Impacts	1.5	1.0	1.0	1.5	1.5
Total Subjective Ranking	**19.2**	**19.2**	**19.9**	**19.1**	**18.9**

(a) Category is not applicable to these alternatives.

As shown in Table 7-8, all the alternatives had similar ranking, with ozonation plus BAC ranking slightly higher than the other alternatives. Additional details regarding this subjective comparison are provided in **Appendix K-2**. A few items of note are:

- The full NDN and parallel MBR alternatives rank highest with respect to reliability because these are more established wastewater treatment technologies.
- The no pretreatment, ozonation, and ozonation/BAC alternatives rank the highest with respect to flexibility, primarily because they do not require the District to commit to a nutrient removal treatment strategy at this time. In addition, the Ozone plus BAC options ranked the highest for flexibility for meeting future regulations because this is a recognized pretreatment option for reservoir augmentation.
- Chemical use was another category with differences among the alternatives. The two ozonation alternatives (with and without BAC) would require routine chemical usage (LOX purchase) and additional electrical power use. The no pretreatment would require a lot more cleaning of the MF system membranes and the MBR option would require cleaning of the MBR membranes and the MF system membranes, which will require more chemical use for maintenance than the other options. Thus, the only option that does not require additional chemical use is the Full NDN process.
- The ozone plus BAC option ranked highest for resources recovery because this alternative lends itself to a reservoir augmentation project.
- The options with ozone also ranked lowest for ancillary impacts because of issues associated with off-gas management and increased truck traffic for LOX deliveries.

7.4 3.0 MGD RAW WATER/TREATED WATER AUGMENTATION TREATMENT FACILITIES

This section presents an evaluation of improvements that may be needed at the WWTP to produce 3.0 mgd of recycled water meeting future raw water or treated water augmentation requirements. The topics addressed include:

- Strategy for Evaluating Raw Water/Treated Water Augmentation Treatment Facilities
- Facilities Overview
- Facilities Layout
- Lifecycle Costs

7.4.1 Strategy for Evaluating Raw Water/Treated Water Augmentation Treatment Facilities

Given the uncertainty with potable reuse regulations, it is not possible to define the treatment requirements of a raw water and treated water augmentation project. Therefore, reasonable estimates of potential treatment trains have been defined herein based on currently available information, research being conducted within in the potable reuse industry, and input from the Vision Plan members. The potential treatment scenarios defined through this assessment are as follows:

- **Raw Water Augmentation Scenario:** In addition to the FAT facilities, the treatment train will include an NDN pretreatment process and ozonation plus BAC.
- **Treated Water Augmentation Scenario:** The treatment train will include the facilities for raw water augmentation plus a final disinfection process that meets current standards prescribed for unrestricted irrigation reuse. In addition, monitoring requirements will drive the need for onsite storage of at least a full day of flow to allow for testing of potable recycled water prior to discharge.

7.4.2 Facilities Overview

This section provides an overview of the facilities that would be constructed to produce 3.0 mgd of recycled water under the raw water and treated water augmentation scenarios discussed above. Schematics of these scenarios are shown on Figure 7-6, which also and includes flows at different points in the treatment train, based on a nominal net 3.0 mgd production capacity for the system. These facilities are further discussed below.

7.4.2.1 3.0 MGD Raw Water Augmentation Facilities

Facilities included in the raw water augmentation scenario are a combination of the facilities previously discussed in this Chapter, as follows:

- The facilities and improvements required to upgrade the 3.0 mgd CWR system to meet the regulatory requirements for a FAT process
- The ozonation plus BAC pretreatment facilities constructed ahead of the upgraded 3.0 mgd CWR FAT process

- NDN pretreatment, either through a full-scale NDN at the main WWTP site or through a parallel stream MBR

7.4.2.2 3.0 MGD Treatment Water Augmentation Facilities

In addition to the raw water augmentation treatment facilities discussed above, it is assumed that following facilities would be needed to support a treated water augmentation project:

- A chlorine disinfection process that provides a modal contact time of 90 minutes.
- Two, 3.0 Mgal tanks would be provided. Each day, one tank would be filled while the second tank is tested and released. The next day, the other tank would be filled, while appropriate monitoring would be performed on water in the first tank. If monitoring results are unfavorable for treated water augmentation, stored water could be either discharged to surface water under the District's NPDES Permit (assuming the quality is adequate) or returned to the treatment facilities for additional treatment.
- The existing MF Product Water Pump Station would be retrofitted to convey effluent from the UV/AOP process to the disinfection and storage facilities.
- A new, 12-inch diameter potable reuse effluent pipeline would be constructed between the Product Water Pump Station and the new facilities.

A new treated water pump station would also be required downstream of the new chlorination and storage facilities to convey the treated water to the reuse site. However, similar to the previous discussion of the FAT facilities, a cost for this pump station is not included in the analysis because the ultimate location for discharge of the treated potable reuse water is not defined.

7.4.3 Facilities Layout

A potential layout of the potential raw water and treated water augmentation facilities is presented on Figure 7-7. Due to limited space at the main WWTP site, the disinfection and storage facilities required for treated water augmentation are assumed to be located at the DLD site. Therefore, the new conveyance pipeline would bring water from the Product Water Pump Station, across Stoneridge Drive, to the DLD site.

7.4.4 Lifecycle Costs

The costs associated with upgrading the CWR facilities to provide for the raw water and treated water augmentation will depend on what (if any) potable reuse treatment facilities are already installed. Thus, five different sets of capital and operating cost estimates were developed for the 3.0 mgd raw/treated water augmentation upgrades, as follows:

- Table 7-9 provides the estimated costs to provide 3.0 mgd production of recycled water meeting raw water and treated water augmentation requirements, assuming no pretreatment is installed with the FAT facilities.

- Table 7-10 provides the estimated costs for 3.0 mgd raw water and treated water augmentation upgrades assuming ozonation pretreatment process is installed with the FAT facilities.
- Table 7-11 provides the estimated costs for 3.0 mgd raw water and treated water augmentation upgrades assuming ozonation plus BAC pretreatment processes are installed with the FAT facilities.
- Table 7-12 provides the estimated costs for 3.0 mgd raw water and treated water augmentation upgrades assuming full NDN pretreatment process is installed with the FAT facilities.
- Table 7-13 provides the estimated costs for 3.0 mgd raw water and treated water augmentation upgrades assuming a parallel stream MBR pretreatment process is installed with the FAT facilities.

Additional details regarding these capital cost estimates are provided in **Appendix H-8**.

Annual operating cost estimates have also been developed for each of the raw and treated water augmentation alternatives, based on the general assumptions detailed above and process-specific cost factors acquired from equipment vendors. Operating costs are summarized in Table 7-14 for 3.0 mgd raw and treated water augmentation facilities with either full NDN or a parallel stream MBR. Additional details regarding these cost estimates are provided in **Appendix I-5**.

Table 7-15 presents a summary of the NPV of annual operating costs and capital costs for all of alternatives.

Table 7-9. Estimated Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities for the 3.0 mgd No Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities, [a] million $	Estimated Additional Cost, million $	
		Raw Water Augmentation[b]	Treated Water Augmentation[c]
MF Membrane System	3.8	-	-
RO Membrane System	8.8	-	-
UV/AOP System	2.3	-	-
Ozone Pretreatment	-	7.4	7.4
BAC Filter Pretreatment	-	4.4	4.4
Full-Scale NDN	-	25.0	25.0
Pumping and Conveyance to DLD Site	-	-	1.3
Final Effluent Chlorination	-	-	0.7
Treated Water Storage/Pump Stations	-	-	28.8
OPCC	**14.9**	**36.8**	**67.6**
Construction Contingency, 10%	1.5	3.7	6.8
Total Estimated Construction Cost	**16.4**	**40.5**	**74.4**
Engineering, Legal and Administrative Costs, 35%[d]	5.2	12.9	23.7
Total Project Costs	**21.6**	**53.4**	**98.1**
Total Project Costs with Initial Project		**75.0**	**119.7**

[a] From Table 7-1
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-10. Estimated Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities for the 3.0 mgd Ozonation Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		Raw Water Augmentation[b]	Treated Water Augmentation[c]
MF Membrane System	1.8	-	-
RO Membrane System	8.8	-	-
UV/AOP System	2.3	-	-
Ozone Pretreatment	7.4	-	-
BAC Filter Pretreatment	-	4.4	4.4
Full-Scale NDN	-	25.0	25.0
Pumping and Conveyance to DLD Site	-	-	1.3
Final Effluent Chlorination	-	-	0.7
Treated Water Storage	-	-	28.8
OPCC	**20.3**	**29.4**	**60.2**
Construction Contingency, 10%	2.0	2.9	6.0
Total Estimated Construction Cost	**22.3**	**32.3**	**66.2**
Engineering, Legal and Administrative Costs, 35%[d]	7.1	10.3	21.1
Total Project Costs	**29.4**	**42.6**	**87.3**
Total Project Costs with Initial Project		**72.0**	**116.7**

(a) From Table 7-5
(b) Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
(c) Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
(d) Calculated as a percentage of the OPCC

Table 7-11. Estimated Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities for the 3.0 mgd Ozonation and BAC Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		Raw Water Augmentation[b]	Treated Water Augmentation[c]
MF Membrane System	1.6	-	-
RO Membrane System	8.8	-	-
UV/AOP System	2.3	-	-
Ozone Pretreatment	7.4	-	-
BAC Filter Pretreatment	4.4	-	-
Full-Scale NDN	-	25.0	25.0
Pumping and Conveyance to DLD Site	-	-	1.3
Final Effluent Chlorination	-	-	0.7
Treated Water Storage	-	-	28.8
OPCC	**24.5**	**25.0**	**55.8**
Construction Contingency, 10%	2.5	2.5	5.6
Total Estimated Construction Cost	**27.0**	**27.5**	**61.4**
Engineering, Legal and Administrative Costs, 35%[d]	8.6	8.8	19.5
Total Project Costs	**35.6**	**36.3**	**80.9**
Total Project Costs with Initial Project		**71.9**	**116.5**

(a) From Table 7-5
(b) Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
(c) Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
(d) Calculated as a percentage of the OPCC

Table 7-12. Estimated Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities for the 3.0 mgd Full-Scale NDN Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		Raw Water Augmentation[b]	Treated Water Augmentation[c]
MF Membrane System	1.8	-	-
RO Membrane System	8.8	-	-
UV/AOP System	2.3	-	-
Ozone Pretreatment	-	7.4	7.4
BAC Filter Pretreatment	-	4.4	4.4
Full-Scale NDN	25.0	-	-
Pumping and Conveyance to DLD Site	-	-	1.3
Final Effluent Chlorination	-	-	0.7
Treated Water Storage	-	-	28.8
OPCC	**37.9**	**11.8**	**42.6**
Construction Contingency, 10%	3.8	1.2	4.3
Total Estimated Construction Cost	**41.7**	**13.0**	**46.9**
Engineering, Legal and Administrative Costs, 35%[d]	13.3	4.1	14.9
Total Project Costs	**55.0**	**17.1**	**61.8**
Total Project Costs with Initial Project		**72.1**	**116.8**

[a] From Table 7-5
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-13. Estimated Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities for the 3.0 mgd Parallel Stream MBR Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		Raw Water Augmentation[b]	Treated Water Augmentation[c]
MF Membrane System	1.8	-	-
RO Membrane System	8.8	-	-
UV/AOP System	2.3	-	-
Ozone Pretreatment	-	7.4	7.4
BAC Filter Pretreatment	-	4.4	4.4
MBR Treatment System	35.0	-	-
Pumping and Conveyance to DLD Site	-	-	1.3
Final Effluent Chlorination	-	-	0.7
Treated Water Storage	-	-	28.8
OPCC	**47.9**	**11.8**	**42.6**
Construction Contingency, 10%	4.8	1.2	4.3
Total Estimated Construction Cost	**52.7**	**13.0**	**46.9**
Engineering, Legal and Administrative Costs, 35%[d]	16.8	4.1	14.9
Total Project Costs	**69.5**	**17.1**	**61.8**
Total Project Costs with Initial Project		**86.6**	**131.3**

[a] From Table 7-5
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-14. Annual Operating Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities, million $

Component	Raw Water Augmentation		Treated Water Augmentation	
	With Full-Scale NDN and Ozonation plus BAC	With MBR and Ozonation plus BAC	With Full-Scale NDN and Ozonation plus BAC	With MBR and Ozonation plus BAC
Chemical	0.4	0.5	0.4	0.5
Electrical	1.4	1.5	1.4	1.5
Labor	1.3	1.3	1.3	1.3
Sampling and Analytical	0.1	0.1	0.2	0.2
Equipment Replacement	0.1	0.1	0.1	0.1
Total Annual Cost	**3.3**	**3.5**	**3.4**	**3.6**

Table 7-15. Estimated Lifecycle Costs for 3.0 mgd Raw Water and Treated Water Augmentation Facilities, million $

Type	Raw Water Augmentation	Treated Water Augmentation
3.0 mgd FAT Facility w/ No Pretreatment		
Total Project Costs	75.0	119.7
NPV of Annual Cost	70.8	72.8
Lifecycle Costs	**145.8**	**192.5**
3.0 mgd FAT Facility w/ Ozone Pretreatment		
Total Project Costs	72.0	116.7
NPV of Annual Cost	70.8	72.8
Lifecycle Costs	**142.8**	**189.5**
3.0 mgd FAT Facility w/ Ozone plus BAC Pretreatment		
Total Project Costs	71.9	116.5
NPV of Annual Cost	70.8	72.8
Lifecycle Costs	**142.7**	**189.3**
3.0 mgd FAT Facility w/ Full-Scale NDN Pretreatment		
Total Project Costs	72.1	116.8
NPV of Annual Cost	70.8	72.8
Lifecycle Costs	**142.9**	**189.6**
3.0 mgd FAT Facility w/ MBR Pretreatment		
Total Project Costs	86.6	131.3
NPV of Annual Cost	75.0	77.3
Lifecycle Costs	**161.6**	**208.6**

As shown in Table 7-15, the total present value cost to go to raw water or treated water augmentation is the essentially the same for all of the pretreatment alternatives, with the exception of the Parallel Stream MBR pretreatment alternative. The capital costs associated with the MBR pretreatment approach would be approximately $15 million more than the others evaluated. Furthermore, the operating costs for the parallel stream MBR are higher than the operating costs for the full-scale NDN. Therefore, the lifecycle costs differences between these two approaches are approximately $20 million.

Table 7-16 shows the estimated cost per acre-foot of recycled water produced. As shown, the cost would be approximately $2,200 per acre-foot produced for raw water augmentation and approximately $3,000 per acre-foot produced for treated water augmentation.

Table 7-16. Estimated Costs per Acre-Foot of 3.0 mgd Raw Water and Treated Water Augmentation Water Production Facility, million $

Type	Raw Water Augmentation	Treated Water Augmentation
Range of Lifecycle Costs	143 - 162	189 - 209
Lifecycle Cost per Acre-Foot Delivered[(a)]	2,600 – 3,000	3,500 – 3,900

[(a)] Unit lifecycle costs are based on a 30-year present worth value, and an estimated production volume of 591 Mgal (or 1,800 acre-feet) per year for 30 years.

7.5 9.0 MGD RAW WATER/TREATED WATER AUGMENTATION TREATMENT FACILITIES

This section presents an evaluation of specific treatment requirements for production of up to 9.0 mgd of recycled water meeting the potable reuse requirements detailed above. The topics addressed include:

- Strategy for Evaluating Potential 9.0 mgd Potable Reuse Facilities
- Facilities Overview
- Facilities Layout
- Lifecycle Costs

7.5.1 Strategy for Evaluating Potential 9.0 mgd Potable Reuse Facilities

The evaluation of the 9.0 mgd raw water and treated water augmentation facilities presented in the Master Plan relies on the following major assumptions:

- For the 9.0 mgd condition, only raw water or treated water augmentation scenario would be applicable.
- The same treatment levels defined previously for raw water and treated water augmentation under the 3.0 mgd flow condition would be required for the 9.0 mgd flow condition.

- The 3.0 mgd flow condition facilities would be initially be constructed, and expansion to 9.0 mgd would involving constructing some facilities with a capacity of approximately 6.0 mgd.

7.5.2 Facilities Overview

This section includes an overview of the potential treatment facilities that would be needed to support the expansion from a 3.0 mgd potable reuse treatment system to a facility that can produce 9.0 mgd of recycled water for either raw water and treated water augmentation.

A schematic of the 9.0 mgd raw water augmentation facilities, based on the treatment train previously described in this Chapter, is provided on Figure 7-8. These schematics also and includes flows at different points in the treatment train, based on a nominal net 9.0 mgd production capacity for the system.

7.5.2.1 9.0 MGD Raw Water Augmentation Facilities

As discussed previously in this Chapter, the treatment train for raw water augmentation is assumed to include the FAT facilities, ozonation plus BAC pretreatment, and NDN treatment.

Due to limited space at the main WWTP site near the existing CWR facilities, two separate FAT systems (one with 3.0 mgd capacity and one with 6.0 mgd output capacity) will be needed for the 9.0 mgd condition. This analysis specifically assumes that the following improvements would be needed:

- New FAT facilities with 6.0 mgd net capacity (and 8.0 mgd input flow) would be constructed at the DLD site.
- Diversion facilities would be needed to bring 8.0 mgd of secondary effluent from the LAVWMA discharge pipeline to the new FAT facilities at the DLD site.
- If full-scale NDN facilities are not previously constructed, then two additional aeration basins, two additional blowers, a mixed liquor return channel, and a new secondary clarifier would be required.
- If a parallel stream MBR is previously constructed, then the full-scale NDN facilities would only require one additional aeration basin and no blower expansion. See Chapter 6 for additional details.
- If not previously constructed to support the 3.0 mgd system, a potable reuse product pump station with a 9.0 mgd capacity would be needed to convey the treated water flows to the reuse location. However, similar to the previous alternative, the cost of this facility is not included.
- The existing Product Water Pump Station and a new 12-inch pipeline would be needed to convey the treated water from the 3.0 mgd facilities to the new 9.0 mgd product water pump station.

Similarly, two separate ozonation plus BAC pretreatment systems will be needed. This is because the main WWTP plant site would house the 3.0 mgd FAT facilities, and pretreatment for this system must be provided upstream. However, the site does not have adequate space for the 9.0 mgd ozonation or BAC pretreatment facilities area near (and upstream of) the existing CWR MF/RO building site where the 3.0 mgd FAT facilities would be constructed. Therefore, the second 6.0 mgd capacity system would be constructed at the DLD site along with the 6.0 mgd FAT facilities.

The additional ozonation and BAC pretreatment facilities required to provide 9.0 mgd of water treated to raw water augmentation standards as a function of which 3.0 mgd potable reuse scenarios had previously been implemented, as summarized in Table 7-17.

Table 7-17. Additional Pretreatment Capacity Needed for 9.0 mgd Raw Water Augmentation

Project Component	Initial 3.0 mgd Pretreatment Alternative					
	No Pretreatment	Ozonation	Ozonation plus BAC	Full NDN	Parallel Stream MBR	Raw Water Augmentation
Ozone	3.0 + 6.0	6.0	6.0	3.0 + 6.0	3.0 + 6.0	6.0
BAC	3.0 + 6.0	3.0 + 6.0	6.0	3.0 + 6.0	3.0 + 6.0	6.0

7.5.2.2 9.0 mgd Treated Water Augmentation Facilities

The 9.0 mgd capacity treated water augmentation facilities would include:

- The 9.0 mgd capacity raw water augmentation facilities identified above,
- A 9.0 mgd capacity chlorine disinfection process with 90 minutes of modal contact time. If 3.0 mgd of chlorination capacity has already been installed, then an additional 6.0 mgd of chlorination capacity would be needed. However, if no potable reuse treatment disinfection capacity had previously been installed, then a 9.0 mgd chlorination facility would be constructed at the DLD site.
- 9.0 mgd of treated water storage capacity, consisting of three, 3.0 Mgal tanks. Each tank would be filled during an 8-hour period, monitored during the following 8 hours, and then discharged to the treated water system or to waste during the final 8-hour period, such that 9.0 mgd of water could be provided for treated water augmentation each day (assuming monitoring results are favorable). As under the 3.0 mgd scenario, if monitoring results are unfavorable for treated water augmentation, stored water could be either discharged to surface water under the District's NPDES Permit (assuming the quality is adequate) or returned to the treatment facilities for additional treatment. If a 3.0 mgd treated water augmentation scenario has previously been constructed, then only one additional tank would be required.

7.5.3 Facilities Layout

Potential layouts for the 9.0 mgd raw water and treated water augmentation facilities are provided on Figure 7-9. As discussed previously, most of the potable reuse treatment facilities would be located at the DLD site. However, the 3.0 mgd capacity FAT system and of ozonation and BAC pretreatment system would be located at the main WWTP site. To achieve this flow split, some secondary effluent flows would be directed to the 3.0 mgd system using the existing MF Feed pump station. The flows for the additional 6.0 mgd production system would be diverted from the LAVWMA pipeline.

7.5.4 Lifecycle Costs

The costs associated with upgrading the WWTP for the 9.0 mgd raw water and treated water augmentation alternatives described above will depend on what (if any) 3.0 mgd potable reuse facilities have been installed. Therefore, capital cost estimates were developed for the 9.0 mgd raw water and treated water augmentation alternatives assuming various progressions from the District's initial 3.0 mgd potable reuse project. These cost estimates are summarized in the following tables:

- Table 7-18: 3.0 mgd FAT facility with no pretreatment progression
- Table 7-19: 3.0 mgd FAT facility with ozonation pretreatment progression
- Table 7-20: 3.0 mgd FAT facility with ozonation plus BAC pretreatment progression
- Table 7-21: 3.0 mgd FAT facility with full-scale NDN pretreatment progression
- Table 7-22: 3.0 mgd FAT facility with a parallel stream MBR pretreatment progression
- Table 7-23: 3.0 mgd raw water augmentation potable reuse treatment facility progression
- Table 7-24: 3.0 mgd treated water augmentation potable reuse treatment facility progression

The cost estimates presented in the above tables were developed following the procedures described in Chapter 5. Additional details are provided in **Appendix H-8, H-9**, and **H-10**, respectively.

Table 7-18. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd FAT Facility with No Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[b]	9.0 mgd Treated Water Augmentation[c]
MF Membrane System	3.8	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	-	20.0	20.0
BAC Filter Pretreatment	-	13.2	13.2
Full-Scale NDN	-	25.0	25.0
Pumping and Conveyance to DLD Site	-	1.3	1.3
Final Effluent Chlorination	-	-	2.6
Treated Water Storage	-	-	36.7
OPCC	**14.9**	**95.7**	**135.0**
Construction Contingency, 10%	1.5	9.6	13.5
Total Estimated Construction Cost	**16.4**	**105.3**	**148.5**
Engineering, Legal and Administrative Costs, 35%[d]	5.2	33.5	47.3
Total Project Costs	**21.6**	**138.8**	**195.8**
Total Project Costs with Initial Project		**160.4**	**217.4**

[a] From Table 7-1
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation includes raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-19. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd FAT Facility with Ozonation Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[a]	9.0 mgd Treated Water Augmentation[b]
MF Membrane System	1.8	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	7.4	12.6	12.6
BAC Filter Pretreatment	-	13.2	13.2
Full-Scale NDN	-	25.0	25.0
Pumping and Conveyance to DLD Site	-	1.3	1.3
Final Effluent Chlorination	-	-	2.6
Treated Water Storage	-		36.7
OPCC	**20.3**	**88.3**	**127.6**
Construction Contingency, 10%	2.0	8.7	12.8
Total Estimated Construction Cost	**22.3**	**97.1**	**140.4**
Engineering, Legal and Administrative Costs, 35%[d]	7.1	30.9	44.7
Total Project Costs	**29.4**	**128.0**	**185.1**
Total Project Costs with Initial Project		**157.4**	**214.5**

[a] From Table 7-5
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-20. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd FAT Facility with Ozonation plus BAC Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[a]	9.0 mgd Treated Water Augmentation[b]
MF Membrane System	1.6	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	7.4	12.6	12.6
BAC Filter Pretreatment	4.4	8.8	8.8
Full-Scale NDN	-	25.0	25.0
Pumping and Conveyance to DLD Site	-	1.3	1.3
Final Effluent Chlorination	-	-	2.6
Treated Water Storage	-		36.7
OPCC	**24.5**	**83.9**	**123.2**
Construction Contingency, 10%	2.5	8.4	12.6
Total Estimated Construction Cost	**27.0**	**92.3**	**135.8**
Engineering, Legal and Administrative Costs, 35%[d]	8.6	29.4	43.1
Total Project Costs	**35.6**	**121.7**	**178.9**
Total Project Costs with Initial Project		**157.3**	**214.5**

[a] From Table 7-5
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-21. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd FAT Facility with Full-Scale NDN Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[a]	9.0 mgd Treated Water Augmentation[b]
MF Membrane System	1.8	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	-	20.0	20.0
BAC Filter Pretreatment	-	13.2	13.2
Full-Scale NDN	25.0	-	-
Pumping and Conveyance to DLD Site	-	1.3	1.3
Final Effluent Chlorination	-	-	2.6
Treated Water Storage	-	-	36.7
OPCC	**37.9**	**70.7**	**110.0**
Construction Contingency, 10%	3.8	7.1	11.0
Total Estimated Construction Cost	**41.7**	**77.8**	**121.0**
Engineering, Legal and Administrative Costs, 35%[d]	13.3	24.7	38.5
Total Project Costs	**55.0**	**102.5**	**159.5**
Total Project Costs with Initial Project		**157.5**	**214.5**

[a] From Table 7-5
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-22. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd FAT Facility with Parallel Stream MBR Pretreatment Progression

Project Component	Estimated Cost for FAT Facilities with Pretreatment[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[a]	9.0 mgd Treated Water Augmentation[b]
MF Membrane System	1.8	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	-	20.0	20.0
BAC Filter Pretreatment	-	13.2	13.2
Full-Scale NDN	-	8.6	8.6
MBR Treatment System	35.0	-	-
Pumping and Conveyance to DLD Site	-	1.3	1.3
Final Effluent Chlorination	-	-	2.6
Treated Water Storage	-	-	36.7
OPCC	**47.9**	**79.3**	**118.6**
Construction Contingency, 10%	4.8	7.9	11.9
Total Estimated Construction Cost	**52.7**	**87.2**	**130.5**
Engineering, Legal and Administrative Costs, 35%[d]	16.8	27.8	41.5
Total Project Costs	**69.5**	**115.0**	**172.0**
Total Project Costs with Initial Project		**184.5**	**241.5**

[a] From Table 7-5
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-23. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd Raw Water Augmentation Progression

Project Component	Estimated Cost for 3.0 mgd Raw Water Augmentation Facilities[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[a]	9.0 mgd Treated Water Augmentation[b]
MF Membrane System	1.8	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	7.4	12.6	12.6
BAC Filter Pretreatment	4.4	8.8	8.8
Full-Scale NDN	25.0	-	-
Pumping and Conveyance to DLD Site	-	1.3	1.3
Final Effluent Chlorination	-	-	2.6
Treated Water Storage	-	-	36.7
OPCC	**49.7**	**58.9**	**98.2**
Construction Contingency, 10%	5.0	5.9	9.8
Total Estimated Construction Cost	**54.7**	**64.8**	**108.0**
Engineering, Legal and Administrative Costs, 35%[c]	17.4	20.6	34.4
Total Project Costs	**72.1**	**85.4**	**142.4**
Total Project Costs with Initial Project		**157.5**	**214.5**

[a] From Table 7-12
[b] Raw water augmentation requires NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
[c] Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
[d] Calculated as a percentage of the OPCC

Table 7-24. Estimated Costs for 9.0 mgd Raw Water and Treated Water Augmentation with the 3.0 mgd Treated Water Augmentation Progression

Project Component	Estimated Cost for 3.0 mgd Treated Water Augmentation Facilities[a], million $	Estimated Additional Cost, million $	
		9.0 mgd Raw Water Augmentation[a]	9.0 mgd Treated Water Augmentation[b]
MF Membrane System	1.8	11.9	11.9
RO Membrane System	8.8	17.2	17.2
UV/AOP System	2.3	7.1	7.1
Ozone Pretreatment	7.4	12.6	12.6
BAC Filter Pretreatment	4.4	8.8	8.8
Full-Scale NDN	25.0	-	-
Pumping and Conveyance to DLD Site	1.3	-	-
Final Effluent Chlorination	0.7	-	2.6
Treated Water Storage	28.8	-	36.7
OPCC	**80.5**	**57.6**	**96.9**
Construction Contingency, 10%	8.1	5.8	9.7
Total Estimated Construction Cost	**88.6**	**63.4**	**106.6**
Engineering, Legal and Administrative Costs, 35%[c]	28.2	20.2	33.9
Total Project Costs	**116.8**	**83.6**	**140.5**
Total Project Costs with Initial Project		**200.4**	**257.3**

(a) From Table 7-12
(b) Raw water augmentation requires full NDN pretreatment, ozonation plus BAC, MF, RO, and UV/AOP processes.
(c) Treated water augmentation requires raw water augmentation processes plus chlorination and storage.
(d) Calculated as a percentage of the OPCC

Annual operation costs and NPV costs for the 9.0 mgd raw water and treated water augmentation alternatives discussed above are shown in Table 7-25 for the raw water/treated water augmentation alternatives. A more detailed breakdown of these operating costs are provided in **Appendix I-6**.

Table 7-25. Annual Operating Costs for 9.0 mgd Raw Water and Treated Water Augmentation Facilities, million $[a]

Component	Annual Cost by Pretreatment Alternative, million $			
	Full-Scale NDN and Ozonation plus BAC		Full-Scale NDN, Ozonation plus BAC, and Parallel Stream MBR	
	Raw Water Augmentation	Treated Water Augmentation	Raw Water Augmentation	Treated Water Augmentation
Chemical	0.8	0.9	0.9	0.9
Electrical	1.8	1.8	1.9	1.9
Labor	1.5	1.5	1.5	1.5
Sampling and Analytical	0.5	0.5	0.5	0.5
Replacements (e.g., UV lamps, membrane modules)	0.2	0.2	0.2	0.2
Total Annual Cost	**4.8**	**4.9**	**5.0**	**5.0**

[a] Operating costs assume a split 3.0/6.0 MGD facility.

Estimated lifecycle costs are summarized in Table 7-26 for the raw water/treated water augmentation alternatives. As shown, the lowest cost long-term option would involve starting with a 3.0 mgd full NDN pretreatment facility.

Table 7-26. Estimated Lifecycle Costs for 9.0 MGD Raw Water and Treated Water Augmentation Facilities, million $

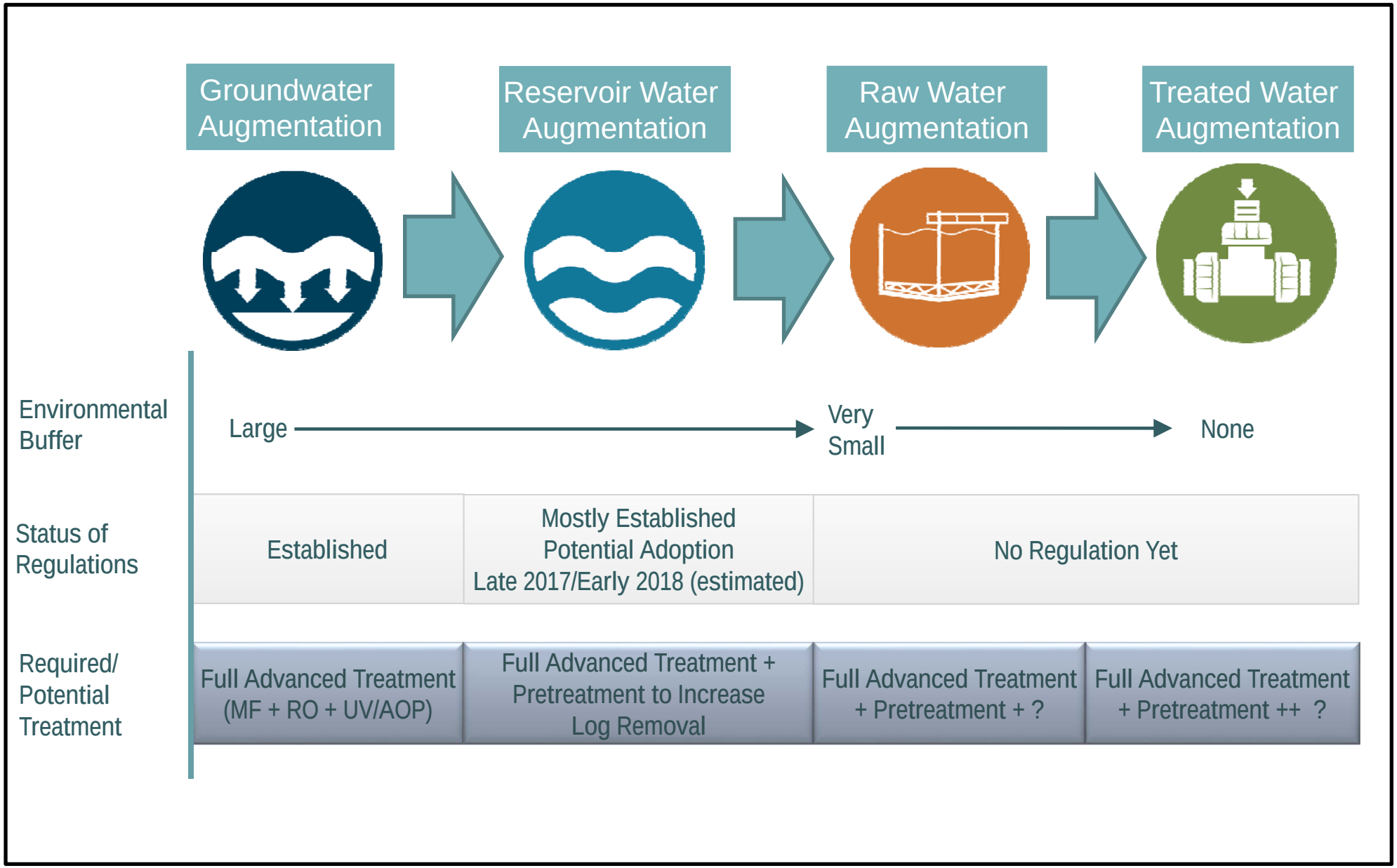
Type	Raw Water Augmentation	Treated Water Augmentation
Previously Installed: 3.0 mgd FAT Facility w/ No Pretreatment		
Total Project Costs	160.4	217.4
NPV of Annual Cost	103.2	104.1
Lifecycle Costs	**263.6**	**321.5**
Previously Installed: 3.0 mgd FAT Facility w/ Ozone Pretreatment		
Total Project Costs	157.4	214.5
NPV of Annual Cost	103.2	104.1
Lifecycle Costs	**260.6**	**318.6**
Previously Installed: 3.0 mgd FAT Facility w/ Ozone plus BAC Pretreatment		
Total Project Costs	157.3	214.5
NPV of Annual Cost	103.2	104.6
Lifecycle Costs	**260.5**	**318.3**
Previously Installed: 3.0 mgd FAT Facility w/ Full-Scale NDN		
Total Project Costs	157.5	214.5
NPV of Annual Cost	103.2	104.1
Lifecycle Costs	**260.7**	**318.6**
Previously Installed: 3.0 mgd FAT Facility w/ MBR Pretreatment		
Total Project Costs	184.5	241.5
NPV of Annual Cost	106.4	107.4
Lifecycle Costs	**290.9**	**318.6**
Previously Installed: 3.0 mgd Raw Water Augmentation Facility		
Total Project Costs	157.5	214.5
NPV of Annual Cost	103.2	104.1
Lifecycle Costs	**260.7**	**318.6**
Previously Installed: 3.0 mgd Treated Water Augmentation Facility		
Total Project Costs	-	257.3
NPV of Annual Cost	-	104.1
Lifecycle Costs	**-**	**361.4**

Table 7-27 shows the estimated cost per acre-foot of recycled water produced. As shown, the cost would be approximately $2,000 per acre-foot produced for raw water augmentation and approximately $2,400 per acre-foot produced for treated water augmentation.

Table 7-27. Estimated Costs per Acre-Foot of Raw Water and Treated Water Augmentation Production in a 9.0 mgd Capacity Facility, million $

Type	Raw Water Augmentation	Treated Water Augmentation
Range of Lifecycle Costs	261 - 291	318 - 361
Lifecycle Cost per Acre-Foot Delivered[(a)]	1,800 – 2,000	2,200 – 2,500

[(a)] Unit lifecycle costs are based on a 30-year present worth value, and an estimated production volume of 1,565 Mgal (or 4,800 acre-feet) per year for 30 years.



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Figure 7-1

Overview of Potable Reuse Regulations

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\CAD\Figures\Figure 7-2_Treatment System Schematic.dwg 8/23/2017 2:59 PM bgong

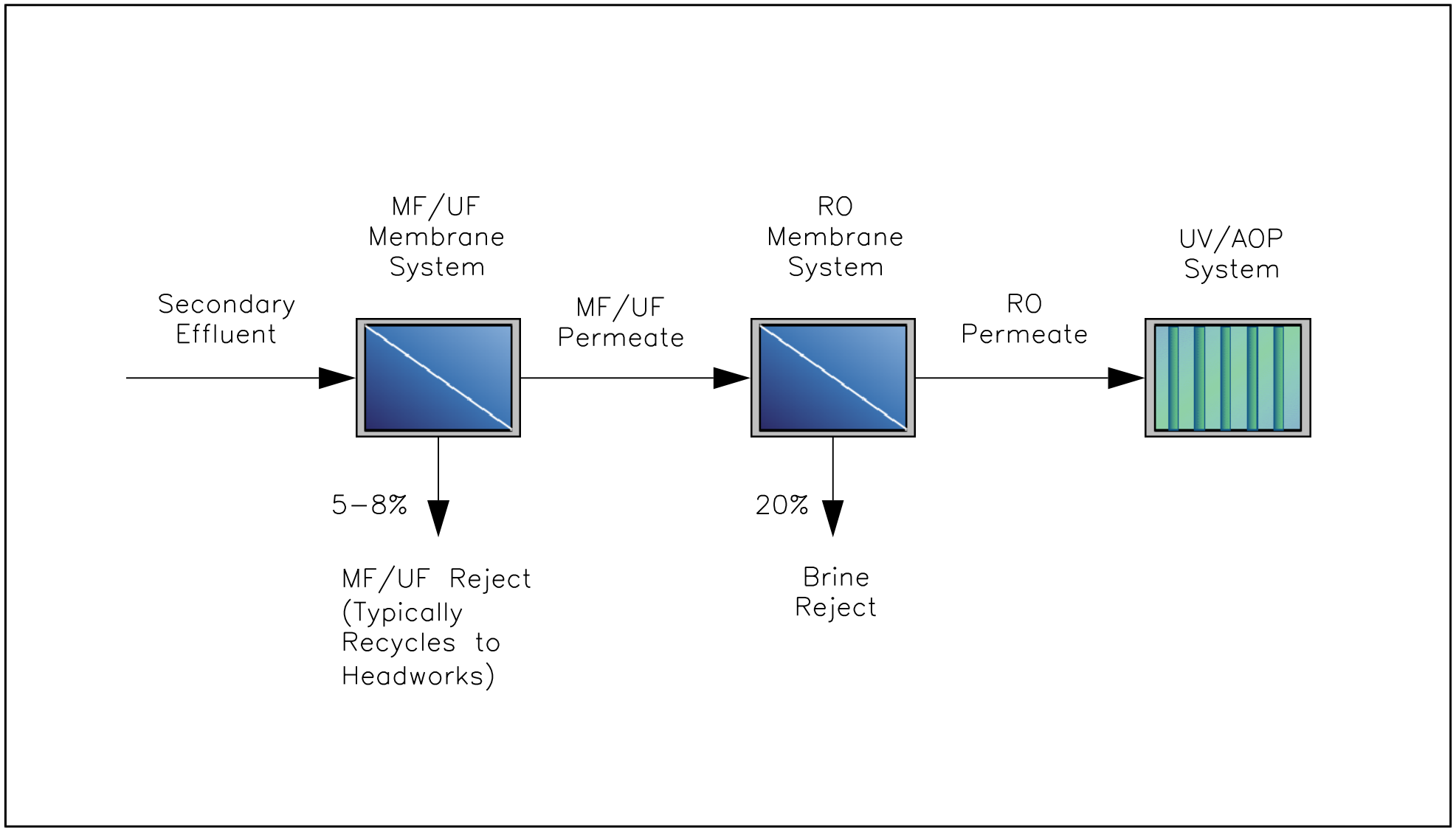


Figure 7-2

Schematic of Typical Full Advanced Treatment (FAT) System

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- FAT Facilities
- New Conveyances

0 70 140

Scale in Feet

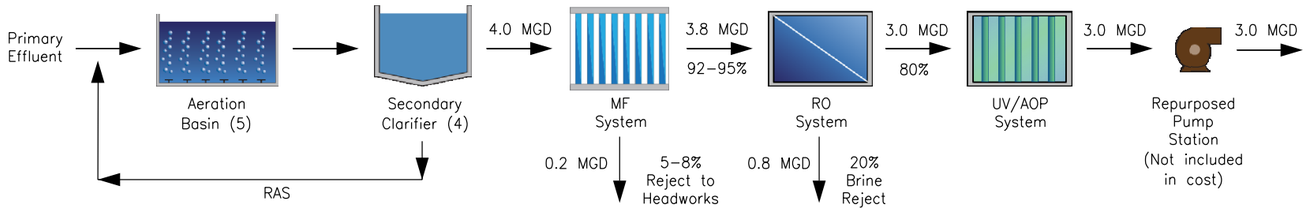
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Figure 7-3

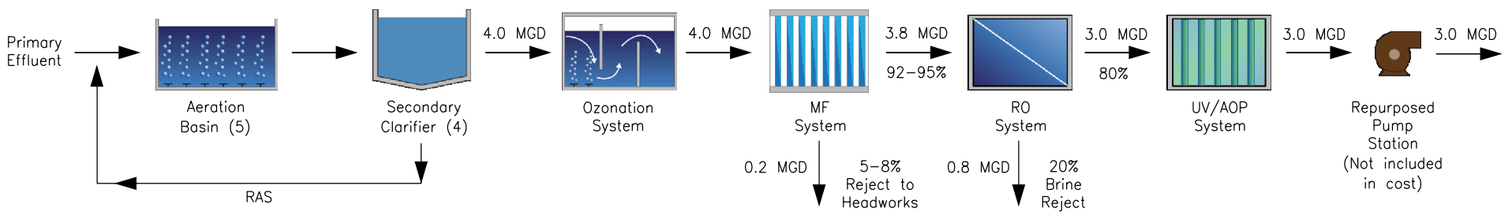
Layout of 3.0 MGD FAT Facilities

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

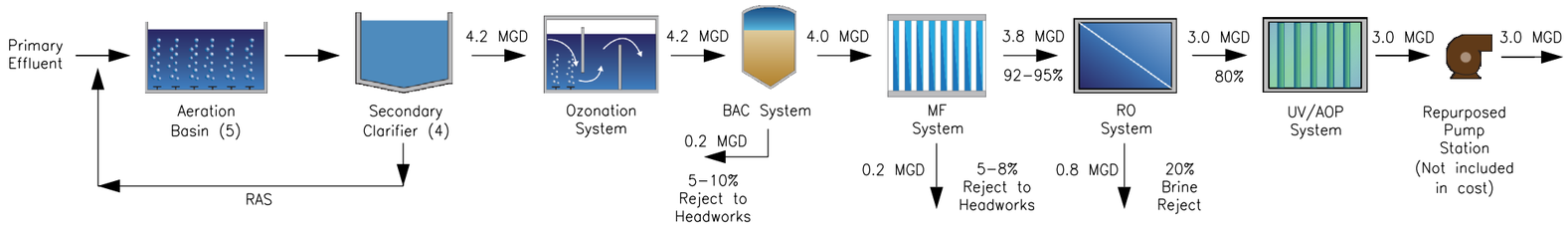
3.0 MGD FAT System with No Pretreatment



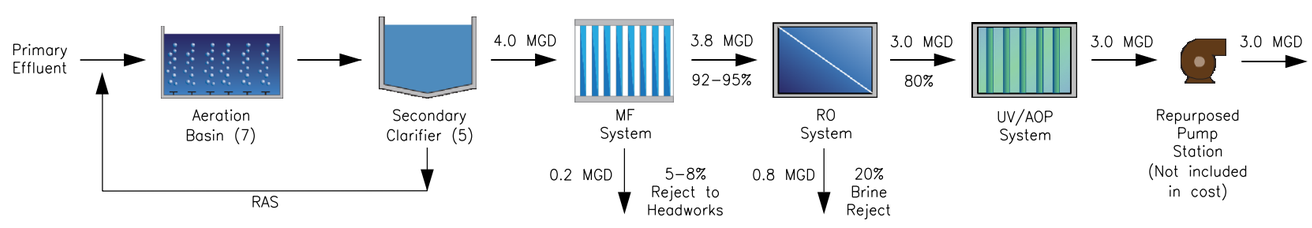
3.0 MGD FAT System with Ozonation Pretreatment



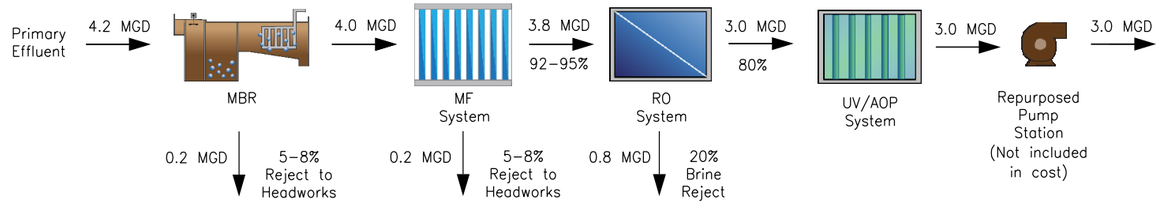
3.0 MGD FAT System with Ozonation plus BAC Pretreatment



3.0 MGD FAT System with Full-Scale NDN Pretreatment (New Aeration Basins and Secondary Clarifier)



3.0 MGD FAT System with Parallel-Stream MBR Pretreatment



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WEST YOST ASSOCIATES

Figure 7-4

Schematics of Pretreatment Alternatives for a 3.0 MGD FAT System

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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MF/RO Systems

Biologically Active Carbon (BAC) Filtration

Repurpose Existing Product Water Pump Station for Export (Not in Project Costs)

Divert from Existing Pipe

UV/AOP System

Existing 14" MF Feed Pipeline

Use Existing Pipe for Additional Ozone Contact Time

New Concrete Contactor Structure

Ozone Building

LOX Tank

Parallel Stream MBR Facility

Full-Scale NDN: Mixed Liquor Return Channel

Existing MF Transfer Pump Station

Full-Scale NDN: Aeration Basins

Full-Scale NDN: Secondary Clarifier

Symbology

- FAT Facilities
- Advanced Oxidation and Pretreatment Facilities: Ozonation and Ozonation plus BAC
- Advanced Oxidation and Nitrogen Removal Pretreatment Facilities: Full-Scale NDN or Parallel Stream MBR
- New Conveyances

N

0 70 140

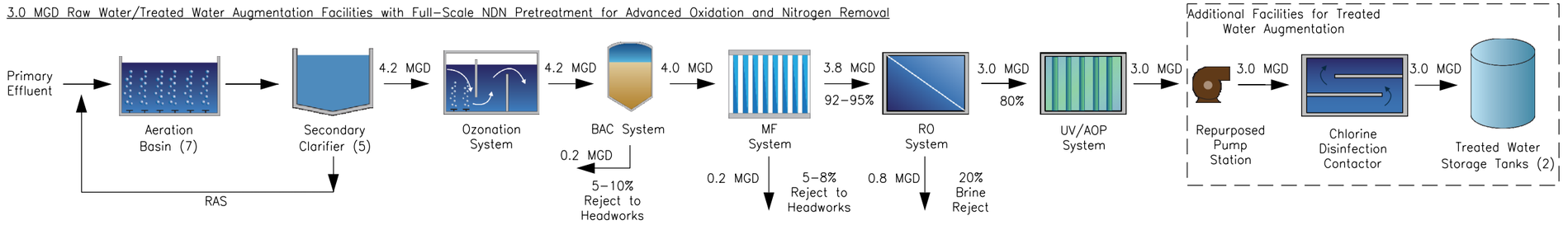
Scale in Feet

WEST YOST ASSOCIATES

Figure 7-5

Layout of Pretreatment Alternatives for 3.0 MGD FAT Facilities

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



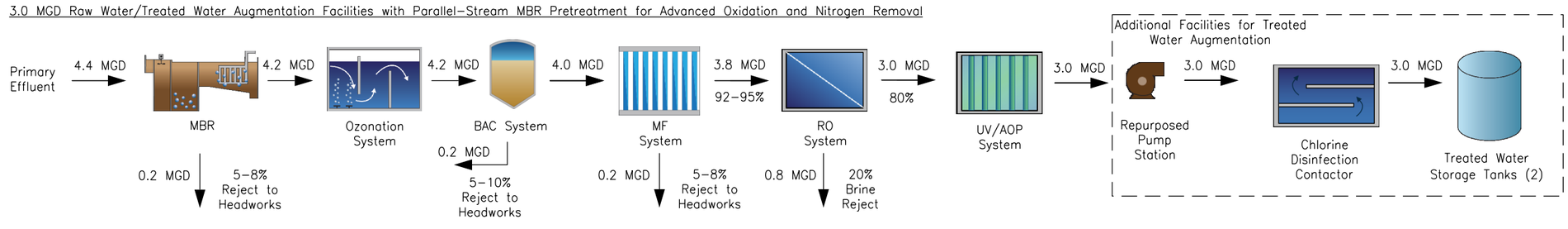


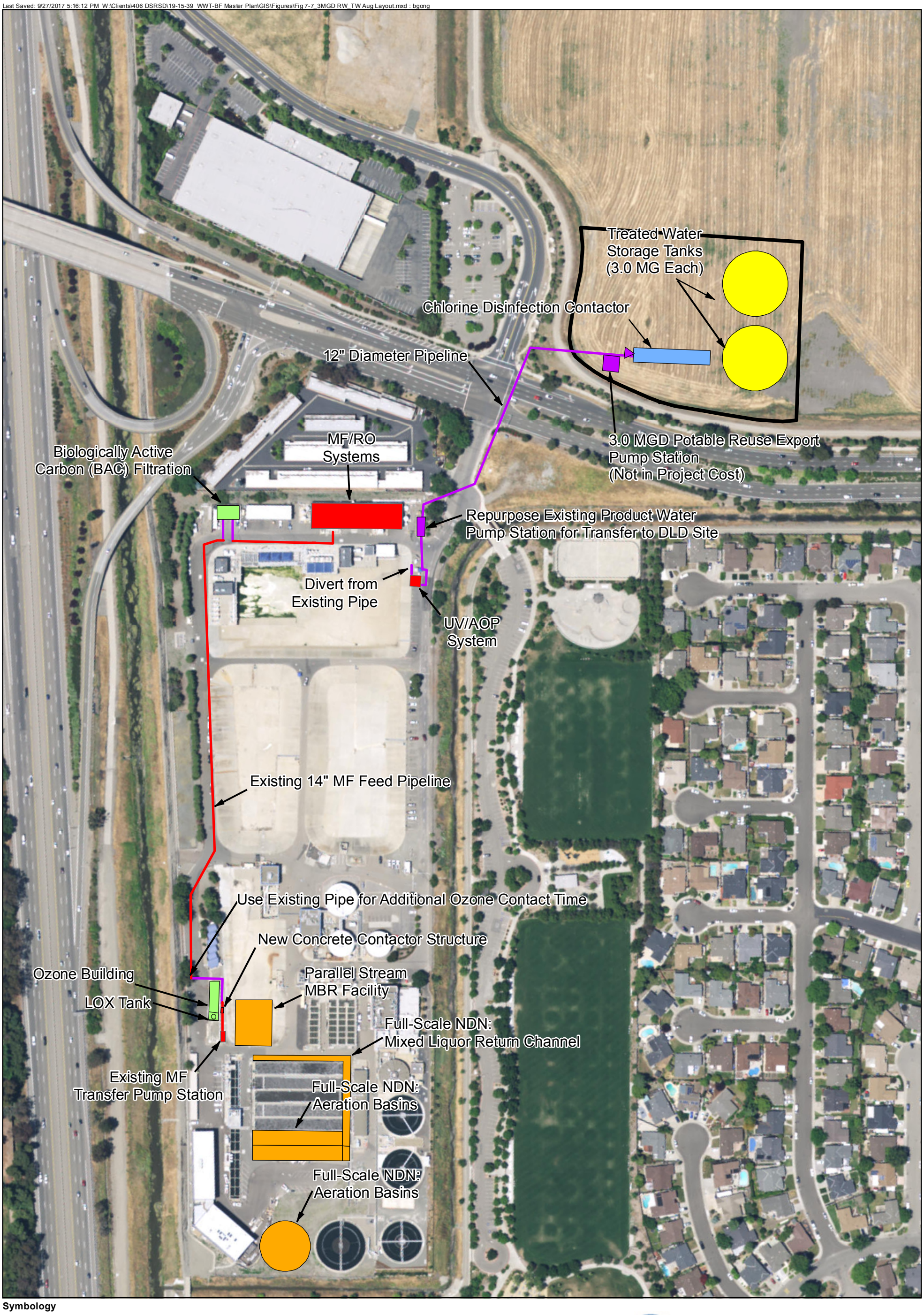
Figure 7-6

Schematics of 3.0 MGD Raw Water/Treated Water Augmentation Treatment Systems

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

Raw Water Augmentation Facilities

- FAT
- Advanced Oxidation Pretreatment: Ozonation plus BAC
- Advanced Oxidation and Nitrogen Removal Pretreatment: Full-Scale NDN or Parallel Stream MBR

Additional Treated Water Augmentation Facilities

- New Conveyances
- Chlorination
- Treated Water Storage

N

0 100 200

Scale in Feet

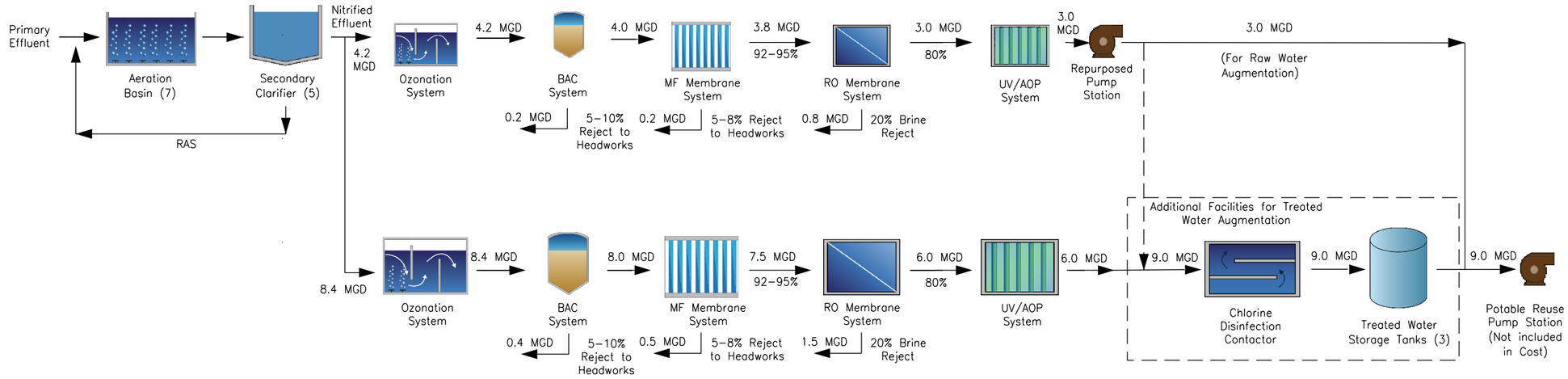
WEST YOST ASSOCIATES

Figure 7-7

Layout of 3.0 MGD Raw Water/Treated Water Augmentation Facilities

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

9.0 MGD Raw Water/Treated Water Augmentation Progression from 3.0 MGD IPR with Full-Scale NDN Pretreatment



9.0 MGD Treated Water Augmentation Progression with 3.0 MGD Parallel-Stream MBR Pretreatment

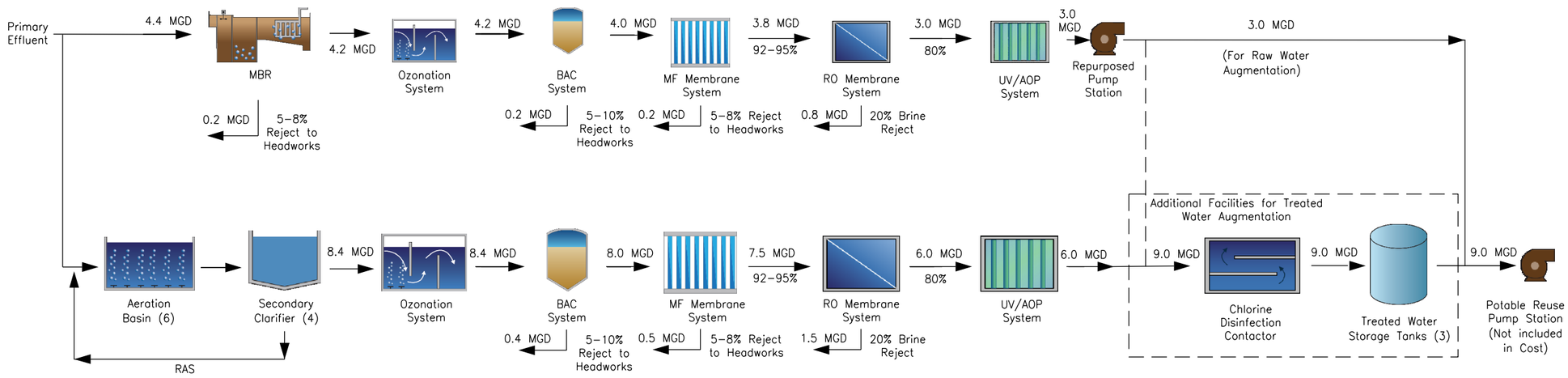


Figure 7-8

Schematics of 9.0 MGD Raw Water/Treated Water Augmentation Treatment Systems

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

WEST YOST ASSOCIATES

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Diversion from Existing Secondary Pipe for Ozone/BAC Pretreatment

Ozonation System (6.0 MGD Only)

Biologically Active Carbon Filtration (6.0 MGD Only)

MF System (6.0 MGD)

Treated Water Storage Tanks (3.0 MG Each)

Optional Discharge from New 30" Pipe

RO System (6.0 MGD)

UV/AOP System (6.0 MGD)

MF/RO Systems (3.0 MGD)

Biologically Active Carbon (BAC) Filtration (3.0 MGD)

Chlorine Disinfection Contactor (9.0 MGD)

9.0 MGD Potable Reuse Export Pump Station (Not in Project Cost)

12" Diameter Pipeline

Repurpose Existing Product Water Pump Station for Transfer to DLD Site

Divert from Existing Pipe

UV/AOP System (3.0 MGD)

Existing 14" MF Feed Pipeline

Use Existing Pipe for Additional Ozone Contact Time

New Concrete Contactor Structure

Ozonation Facilities (3.0 MGD)

MBR Facility (optional)

LOX Tank

Mixed Liquor Return Channel

Existing MF Transfer Pump Station

Aeration Basins (Only one needed if MBR facility is installed)

Secondary Clarifier (Not needed if MBR facility is installed)

Symbology

- FAT Facilities
- Additional Facilities for 9.0 mgd Raw Water Augmentation
- Additional Facilities for 9.0 mgd Treated Water Augmentation
- New Conveyances

N

0 100 200

Scale in Feet

WEST YOST ASSOCIATES

Figure 7-9

Layout of 9.0 MGD Raw Water/Treated Water Augmentation Facilities

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

CHAPTER 8
Solids Treatment Process Alternatives Evaluation

This Chapter presents an evaluation of the facilities required to meet the long-term solids treatment process and conveyance needs for the WWTP. Facilities that may be necessary in the future to address potential future regulatory requirements and emerging opportunities for beneficial solids reuse are also identified. To support the identification and development of these potential future facility needs, a series of Visioning Panel Workshops were conducted to gather input and direction from external experts regarding likelihood of various future opportunities and constraints. Meeting notes from these workshops are provided in Appendix E.

The topics addressed in this Chapter include:

- Evaluation Methodology
- Alternatives for Increasing Anaerobic Digestion Capacity
- Alternatives for Reducing DLD Site Hydraulic Loadings
- Potential Future Full Dewatering and Offsite Disposal Improvements
- Potential Future Addition of a Supercritical Oxidation Process

8.1 EVALUATION METHODOLOGY

This section presents an overview of the approach applied when evaluating the sizing and capacity of the required solids treatment process alternatives under this Master Plan. The following specific topics are addressed:

- Evaluation Objectives
- Process Sizing Approach

8.1.1 Evaluation Objectives

The primary objectives of this solids treatment process evaluation are as follows:

- **Identify Improvements Needed to Address Identified Capacity Deficiencies:** As discussed in Chapter 3 of this Master Plan, the anaerobic digesters are not adequately sized to process the solids anticipated at the 2035 design condition. In addition, the DLD site is currently operating at, or near, its hydraulic loading limitation, and the available data suggests that a significant volume of biosolids are accumulating in the FSLs each year. FSL harvesting data from 2009-2015 indicate that an average of 1,210 dt of solids are applied to the DLD before it reaches its hydraulic capacity. The plant mass balance for current conditions indicates that 2,200 dt of solids should be disposed of from the FSLs to avoid accumulating solids. This evaluation identifies the approximate cost, size, location, and layout of facility improvements needed to address these two capacity deficiencies.

- **Identify Treatment Facilities Potentially Needed to Eliminate Biosolids Disposal on the DLD Site:** Continued use of the DLD site as a disposal option for the District's biosolids may not be a viable long-term strategy. First, there is risk that either changing regulations or community nuisance complaints could drive the District to eliminate the practice. Second, biosolids are increasingly being considered a valuable product due to their ability to rehabilitate and add nutrients to soils. This evaluation identifies the approximate cost, size, location, and layout of facility improvements needed to allow for full dewatering of the biosolids from the FSLs in the event it becomes desired or necessary to eliminate the current practice of disposing biosolids on the DLD site.
- **Identify Treatment Facilities Potentially Needed to Eliminate Use of the FSL and DLD Site:** Continued use of the FSLs for biosolids stabilization and storage prior to DLD application may not be a viable long-term strategy. First, there is risk that either changing regulations or community nuisance complaints could drive the District to eliminate the use of the FSLs. Second, the FSLs provide approximately 82 million gallons of storage. As discussed in the Storage Needs Assessment TM provided in Appendix C, the District could potentially use the storage capacity provided by the FSLs for storage of treated recycled water or storage of secondary effluent that is destined for treatment and reuse. If the FSLs are eliminated, the District would need to dewater the digester solids year-round and it would no longer be possible to apply liquid biosolids to the DLD site. Therefore, elimination of the FSLs would also require either 1) a facility that allows for storage of dewatered solids for later application on the DLD site, or 2) the facilities needed to allow for offsite disposal. This evaluation identifies the approximate cost, size, location, and layout of facility improvements needed to allow for full dewatering of the District's biosolids from the digester and to dispose of the dewatered solids offsite year-round, eliminating both FSLs and DLD operations.
- **Evaluate the Relationships Between a Potential AquaCritox Solids Disposal Process Located at the WWTP Site and the Facility Improvements Needed to Address Long-Term Biosolids Management Goals:** The District is considering partnering with Supercritical Fluids International (SCFI) to construct an AquaCritox system at the WWTP site as part of the Bay Area Biosolids Coalition (BABC) program. This evaluation identifies what facilities would be necessary to accommodate the AquaCritox system and how this helps to address the District's long-term biosolids management goals.
- **Identify Potential Location for Future Biosolids Return Stream Treatment**: As discussed in Chapter 6, it is recommended that biosolids return stream treatment be provided in the event that the BACWA Level 3 treatment requirements are applied to the WWTP. The type of treatment possible for the biosolids return stream will depend on what strategies being applied for biosolids management. However, the preferred return stream treatment technology will also depend on what strategy is being employed for nutrient removed. The combined factors that should be considered over the next 20-year period are addressed in Chapter 10 (Recommended Plan). However,

Table 8-13. Summary of Additional Facility Needs to Achieve Full Dewatering

Precursor Alternative: Facility Type	Thicken All FSL solids for DLD Site Application in Summer		Dewater Portion of Digested Solids for Offsite Disposal Year-Round		Dewater a Portion of FSL Solids for Offsite Disposal in Winter		Dewater All FSLs Solids for DLD Site Application in Summer	
	From FSLs	From Digesters	From FSLs	From Digesters	From FSLs	From Digesters	From FSLs	From Digesters
Thickening/ Dewatering Equipment	Convert two thickeners to dewatering units	Convert three thickeners to dewatering units	None	Install one additional screw presses	None	Install two additional screw presses	None	
Pumping and Conveyance	None	Install pumping and conveyance to between digested sludge pipeline and dewatering building	New pipeline between FSL dredge piping and dewatering building	None	None	Install pumping and conveyance to connect digested sludge pipeline to dewatering building	None	Install pumping and conveyance to connect digested sludge pipeline to dewatering building
Dewatered Sludge Conveyance	Install dewatered sludge conveyor to connect dewatering equipment to truck loading area		None		None		None	
Additional Flow Equalization	20,000 gallons of FSL solids storage	80,000 gallons of digested solids storage, convert thickened sludge storage to filtrate storage	40,000 gallons of FSL solids storage	100,000 gallons of digested solids storage, 70,000 gallons filtrate storage	60,000 gallons of FSL solids storage	120,000 gallons of digested solids storage, 80,000 gallons filtrate storage	60,000 gallons of FSL solids storage	120,000 gallons of digested solids storage, 20,000 gallons filtrate storage
Building and Odor Control	None	Expand building to provide odor controlled truck loading bay and add biofilter,	None		None	Expand building to provide odor controlled truck loading bay and add biofilter,	None	Expand building to provide odor controlled truck loading bay and add biofilter,
Stranded Assets	Two thickening units; Thickened sludge pumping, conveyance and storage	Thickened sludge pumping to equalization storage	Expanded building and odor control biofilter	None	None	None	Solids spreader truck	
							Three screw press[(b)]	One screw presses

[(a)] Dewatering and hauling period could be decreased to 20 weeks given the number of available screw presses.

8.2 ALTERNATIVES FOR INCREASING ANAEROBIC DIGESTION CAPACITY

As documented in Chapter 3, the volatile solids and hydraulic loadings to the anaerobic digesters exceed the available capacity if the largest unit is out of service. This section presents a discussion of the evaluation completed to assess the options for addressing the identified anaerobic digester capacity deficiency. The topics addressed are as follows:

- Alternatives Overview
- Alternatives Comparison

8.2.1 Alternatives Overview

The anaerobic digestion capacity could be increased by either constructing a fourth digester or installing recuperative thickening equipment at the existing digesters. Both alternatives could provide a sufficient increase in volatile solids and hydraulic loading capacity to reliably treat the 2035 maximum 30-day conditions with the largest digester out of service.

8.2.1.1 Fourth Digester

Construction of a fourth anaerobic digester would increase the solids and hydraulic loading capacities of the anaerobic digesters. For this analysis, the new digester is assumed to be a 70-foot diameter digester like the existing Digester No. 3. In addition to the digester itself, new digester heating equipment and mixing equipment would need to be installed.

8.2.1.2 High Solids Digestion

A high solids digestion process increases the solids content and solids residence time within a digester allowing for increased solids loading to the digester and decreased hydraulic residence time, eliminating the need for a fourth digester to meet the 2035 planning condition required capacity. This is achieved through the use of recuperative thickeners which draw solids from the digesters, remove water, and return the thickened solids to the digesters. Empirical results from other treatment plants that utilize the high solids digestion process demonstrate that the volatile solids loading to the digester can be safely increased from 0.15 lb/ft^3 to 0.45 lb/ft^3. This improved performance is assumed to result from a higher concentration of microorganisms and improved digester mixing.

A high solids digestion thickening and mixing system called Omnivore is manufactured by Anaergia and was evaluated for this alternative. Two Omnivore systems, which include recuperative thickeners and a digester mixing system, installed on the two smaller existing digesters would be needed to adequately increase the anaerobic digestion capacity. One of the Omnivore systems provides redundancy. Digester No. 3 would continue to be operated as a conventional anaerobic digester.

The following additional facilities would be needed with the Omnivore thickening and mixing systems:

- A 40,000-gallon thickening filtrate storage tank and the associated pumps to equalize the filtrate return stream before it is directed to the liquid treatment process
- New digested sludge pumps which transfer digested sludge to the FSLs to handle the higher solids content digested sludge
- Two new digester heating systems are required to meet the increased heat demand of the digesters which have the Omnivore systems installed

8.2.2 Alternatives Comparison

This section presents a comparison of the two alternatives discussed above with respect to treatment capacity, layout, lifecycle costs, and the non-economic factors. These alternatives also provide advantages and disadvantages with respect to potential long-term solids handling projects that might occur in the future. A recommended strategy for meeting the digester capacity needs must consider all of digester facilities plus the potential future downstream solids handling facilities. Thus, a recommended approach to addressing the digester capacity limitations is not addressed below, but is instead provided in Chapter 10 of this Master Plan.

8.2.2.1 Treatment Capacity

A comparison of the expected firm and total digestion capacities with the fourth digester and with recuperative thickening is provided in Table 8-2. Also shown for reference is the required digestion capacity. As shown, the recuperative thickening alternative provides for more volatile solids loading capacity than the fourth digester alternative, and the fourth digester provides for more hydraulic loading capacity. Both alternatives result in excess digestion capacity, which could allow for the receipt of other hauled wastes (such as fats, oil and grease, or FOG, and organic food waste) at the WWTP.

Table 8-2. Anaerobic Digestion Capacity with Fourth Digester

Digester Number	VSS Loading Capacity, lb/day		Hydraulic Loading Capacity, gpd	
	Add a Fourth Digester	High Solids Digestion	Add a Fourth Digester	High Solids Digestion
No.1 (existing)	11,600	34,800	38,600	72,400
No. 2 (existing)	11,600	34,900	38,700	72,600
No. 3 (existing)	20,700	20,700	68,900	68,900
No. 4 (future)	20,700	-	68,900	-
New Firm Capacity	43,900	55,500	146,200	141,300
New Total Capacity	64,600	90,400	215,100	213,900
Anticipated Maximum 30-Day Loading	28,200		127,900	

8.2.2.2 Facilities Layout

The potential layouts of the two digestion alternatives are shown on Figure 8-3. The following information is shown on this figure:

- Both the digestion and recuperative thickening processes would be constructed within the District's existing employee parking areas. Discussions with District staff indicate that employee parking would need to be accommodated in the spaces that surround Holding Basin 3. This may require relocating the residential fill station that currently occupies this space.
- Recommended locations for a potential future FOG and/or food waste receiving facilities are identified. These locations allow for reasonable truck access, and in the case of the food waste receiving facility, provide for the facilities to be located in an odor-controlled building. Additional discussion regarding these potential facilities is provided in Chapter 9 (Energy Management Plan).
- A recommended location for a potential future return stream treatment process is identified for the recuperative thickening alternative. As discussed previously, it is recommended that the District provide for treatment of high-strength return streams under the BACWA Level 3 nutrient removal scenario. Recuperative thickening results in the generation of a high-strength return stream. Therefore, this alternative would require that a return stream treatment system be located at the main treatment plant site.

8.2.2.3 Lifecycle Costs

A comparison of the capital costs for the two alternatives is provided in Table 8-3. These costs were developed following the procedures detailed in Chapter 5, and cost estimate details are provided in Appendix H-10. Annual operation costs and estimated net present value costs are shown in Tables 8-4 and 8-5, respectively. A more detailed breakdown of these operating costs is provided in Appendix I-7.

Table 8-3. Summary of Capital Cost Estimates for Digester Capacity Alternatives

Operating Cost Component	Estimated Costs, million $	
	Fourth Digester	High Solids Digestion
Estimated Facilities Cost	5.3	6.6
Construction Contingency, 10%	0.5	0.7
Total Estimated Construction Cost	**5.8**	**7.3**
Engineering, Legal and Administrative Costs, 35%	1.9	2.3
Total Project Costs	**7.7**	**9.6**

Table 8-4. Summary of Operating Cost Estimates for Digester Capacity Alternatives

Operating Cost Component	Estimated Costs, million $	
	Fourth Digester	High Solids Digestion
Chemicals	-	0.2
Electrical	0.2	0.1
Labor and Maintenance	0.1	0.2
Total	**0.3**	**0.5**

Table 8-5. Summary of Net Present Value for Digester Capacity Alternatives

Operating Cost Component	Estimated Costs, million $	
	Fourth Digester	High Solids Digestion
Total Capital Costs	7.7	9.6
Total Annual Costs as NPV[(a)]	6.4	10.7
Total Present Worth Value (Lifecycle Cost)	**14.1**	**20.3**

As indicated in the tables above, the costs associated with the installation of a fourth digester are lower than the costs associated with recuperative thickening. However, as discussed later in this Chapter, there could be economic benefits associated with recuperative thickening under a solids thickening/dewatering scenario. In addition, recuperative thickening would provide more overall volatile solids digestion capacity, which increases the District's potential for bringing in hauled waste to generate energy at the WWTP from biogas. These benefits should therefore be considered as part of the economic evaluation of these alternatives.

8.2.2.4 Non-Economic Comparison

Non-economic factors were considered as part of the alternatives analysis. Factors considered include reliability, flexibility, potential for resource recovery, ancillary impacts, and others. A summary of these non-economic factors is presented in Table 8-6. Additional details regarding this subjective comparison are provided in Appendix K-3.

Table 8-6. Non-Economic Comparison of Alternatives for Addressing Anaerobic Digestion Capacity Limitations

Criteria	Weighted Subjective Criteria	
	Fourth Digester	High Solids Digestion
Reliability	9.7	4.0
Flexibility	5.3	5.3
Odor generation potential	4.5	1.5
Maximize Available Water Storage Volume	(a)	(a)
Energy Management	4.0	5.0
Chemical use	3.0	2.0
Resource Recovery	2.3	3.0
Ancillary Impacts	(a)	(a)
Total Subjective Ranking	**27.8**	**21.6**

(a) Category is not applicable to these alternatives.

As shown in Table 8-6, the fourth digester alternative ranked significantly higher than the high solids digestion alternative. Key factors that impacted this analysis are as follows:

- The most significant rankings difference is in the reliability category. Anaerobic digestion is a known, reliable technology and does not introduce a new process at the WWTP. Whereas high solids digestion is a relatively new technology and introduces a new process. Therefore, high solids digestion ranked considerably lower than the fourth digester in this category.
- High solids digestion and the fourth digester alternative ranked the same for flexibility. The fourth digester provides for future innovations with respect to high solids digestion (an emerging field), but the fourth digester takes up more space and has more of an impact on long-term space planning for the WWTP.
- The high solids digestion process has a separate thickening unit, which could be a source of odors. Therefore, this option ranked lower for this criterion.
- High solids digestion provides for higher volatile solids loadings, which would provide opportunities for increased digester gas production over the fourth digester approach.
- The high solids digestion process would allow for recovery of additional water from the biosolids that would otherwise be lost in the FSL process. In addition, the high solids digestion would provide for a higher-concentrations of nutrients in the biosolids. Therefore, recovery of nutrients could be more economical.

8.3 ALTERNATIVES FOR REDUCING DLD SITE HYDRAULIC LOADINGS

This section presents an evaluation of the alternatives that have been identified for reducing hydraulic loadings to the DLD site. The topics addressed include:

- Strategy for Evaluating Facilities to Reduce DLD Hydraulic Capacity Loadings
- Alternatives Overview
- Alternatives Comparison

8.3.1 Strategy for Evaluating Facilities to Reduce DLD Hydraulic Loadings

The District's plans to continue to rely on the DLD to the extent it provides a reliable, cost-effective biosolids disposal method. As noted previously, the DLD site is currently limited by hydraulic loading. Specifically, biosolids are currently applied from the FSLs to the DLD site at approximately three percent solids content. At this solids content, the soils onsite become saturated by the end of the application season, preventing further application of biosolids until the excess water has evaporated or percolated. This saturation indicates that the current application rate represents the hydraulic capacity of the DLD site. From 2009 to 2015, the average solids applied to the DLD site was approximately 1,200 dt/year. Thus, if solids are applied at a 3 percent solids content, the capacity of the DLD site is 1,200 dt/year.

There are two basic approaches for addressing the DLD hydraulic loading issues. One approach would be to thicken/dewater the FSL solids prior to disposal on the DLD site. The second approach involves dewatering all or a portion of the treated biosolids and disposing of them in a different manner. A range of alternatives have been identified that could achieve these objectives.

For all of the alternatives identified, it has been assumed that the thickening or dewatering facilities could be readily expanded to allow for full dewatering of all solids. This strategy will allow for a quick transition away from the DLD operation should it be necessary or desired either due to new regulations, if concerns emerge over odors generated from the DLD site, or if a market opens up for reuse of the biosolids.

8.3.2 Alternatives Overview

The following alternatives have been identified for reducing hydraulic loadings to the DLD site:

- Thicken all FSL solids for DLD site application in summer
- Dewater a portion of digested solids for offsite disposal year-round
- Dewater a portion of FSL solids for offsite disposal in winter
- Dewater all FSL solids for DLD site application in summer

Schematics for each of these biosolids alternatives are presented on Figure 8-4.

Note that the schematics depicted in Figure 8-4 all assume the District selects the fourth digester alternative to address the digestion capacity issues. The only difference between the approach shown and the recuperative thinking approach would be the thickness of the solids leaving the digestion process. However, under most of the alternatives, all of these thicker solids would be directed to the FSLs where they would then become diluted to three percent solids. The only exception would be the alternative that involves dewatering a portion of the digested solids for offsite disposal. However, for this alternative, the dry tons of solids that would be digested would not change even if the percent solids leaving the digester did increase.

8.3.2.1 Thicken All FSL Solids for DLD Site Application in Summer

One option considered for increasing the solids capacity of the DLD site is to thicken the solids prior to injection on the DLD site. If the solids content of the applied solids in the summer is increased, a larger mass of solids can be applied to the site with the same volume of water. Assuming a 12-week operation period at the 2035 design condition, solids will need to be thickened to approximately 5.1 percent for all of the FSL solids to be applied to the DLD.

Three rotary screen thickeners sized to process 270 gpm of sludge at three percent solids would be required for this alternative. The thickeners are assumed to be operated over a minimum period of 12 weeks, five days per week, and six hours per day. A redundant unit would not be provided because the FSLs provide adequate flexibility to store solids and the hours or duration of operation could be increased in the event one of the units is temporarily out of service. In addition, one full time employee is assumed to be needed to operate the thickening equipment, and two full time employees are assumed to be needed to apply the thickened solids to the DLD site.

The following additional facilities would also be needed:

- A dredge with sufficient capacity to supply 810 gpm of solids from the FSLs to the thickening equipment. A dredge equipped with a sludge density meter may be advantageous operationally, as it enables better control over the solids content pumped from the lagoon.
- 100,000 gallons of FSL solids storage to equalize solids flows to the thickening equipment – equivalent to two hours of storage.
- 60,000 gallon thickened solids storage tank to hold thickened solids prior to DLD application
- 120,000-gallon filtrate storage for equalization of the return flows before treatment and return to the headworks
- A 4,400-square foot building to house the thickening equipment. It is assumed that an odor controlled building will not be needed, as the volatile fraction of the FSL stabilized solids will have been significantly reduced. This assumption regarding odor control should be assessed prior to implementation of this alternative, should it be selected as the preferred approach.

8.3.2.2 Dewater a Portion of Digested Solids for Offsite Disposal Year-Round

Reducing the amount of biosolids that are directed to the FSLs will, in turn, reduce the amount of biosolids and water that needs to be applied to the DLD site. This reduction can be achieved by diverting a portion of the digested solids upstream of the FSLs, dewatering them, and then hauling the dewatered solids offsite for disposal or beneficial reuse year-round. This alternative would allow the remaining portion of the solids to be harvested from the FSLs and applied to the DLD site consistent with current operations.

Two screw presses that are sized to process 80 gpm of biosolids at 1.6 percent solids content would be required for this alternative. This capacity is based on an assumption that dewatering would occur seven days a week, 52 weeks per year, for six hours per day. A redundant unit would not be provided, as the hours of operation could be increased or more solids could be sent to the FSLs in the event one of the units is temporarily out of service. All solids not diverted to the dewatering equipment would be discharged to the FSLs. One full time employee is assumed to be needed to operate the dewatering equipment, and two full time employees are assumed to be needed 12 weeks per year to apply the remaining FSL solids to the DLD site.

The following additional facilities would also be needed:

- 60,000 gallons of filtrate storage to equalize the filtrate flows before treatment and return to the headworks
- 80,000 gallons of digested solids storage to equalize solids flows to the dewatering equipment
- A 5,900 square foot building to house the dewatering equipment and control odors from the equipment
- An odor control biofilter facility to mitigate odors from the dewatering equipment

In addition, because fewer solids would be sent to the FSLs, only four of the six existing FSLs would be needed, allowing for two of the FSLs to be decommissioned (or potentially repurposed to provide treated effluent or recycled water storage).

8.3.2.3 Seasonally Dewater a Portion of FSL Solids for Offsite Disposal in Winter

A third alternative is to continue to send all digested solids to the FSLs and maximize solids application to the DLD site at three percent solids during the summer, as is currently practiced. However, during the winter months, excess solids could be drawn from the FSLs, dewatered, and hauled offsite for disposal or beneficial reuse.

Two screw presses, sized to dewater 80 gpm at three percent solids, would be required for this alternative, assuming dewatering would occur a minimum of 20 weeks per year, seven days per week, and six hours per day. A redundant unit would not be provided because the FSLs provide adequate flexibility to store solids and the hours or duration of operation could be increased in the event one of the units is temporarily out of service. In addition, one full time employee is assumed to be needed to operate the dewatering equipment and two full time employees are assumed to be needed 12 weeks per year to apply the remaining FSL solids to the DLD site.

The following additional facilities would also be needed:

- 50,000 gallons of filtrate storage
- 60,000 gallons of FSL solids storage to equalize solids flows to the dewatering equipment
- A 4,400-square foot building to house the dewatering equipment. (It is assumed that an odor controlled building will not be needed, as the volatile fraction of the FSL stabilized solids will have been significantly reduced.)

8.3.2.4 Dewater All FSLs Solids for DLD Site Application in Summer

The final alternative evaluated for addressing the hydraulic capacity limitations of the DLD site is to continue to send all digested solids to the FSLs and subsequently dewater all FSL solids prior to application to the DLD site in the summer months.

Five screw presses sized for a flow of 70 gpm are needed to dewater all of the solids from the FSLs during the summer months. For the purpose of sizing the equipment, it was assumed that the screw presses would be operated 20 weeks per year, six hours per day, and seven days per week. One full time employee is assumed to be needed to operate the dewatering equipment. Similar to the previous alternatives, no redundant press is needed and it is assumed that odor control will not be a concern for this alternative.

The following additional facilities would be needed:

- 110,000 gallons of filtrate storage
- 60,000 gallons of FSL solids storage to equalize solids flows to the dewatering equipment
- A 4,400-square foot building to house the dewatering equipment.
- A solids spreader truck would need to be purchased to apply the dewatered solids to the site

8.3.2.1 Summary of Facility Requirements

A summary of the facilities needed under each of the four alternatives listed above is provided in Table 8-7.

Table 8-7. Summary of Facilities Needed to Address DLD Hydraulic Capacity Limitations

Facility Type	Thicken All FSL solids for DLD Site Application in Summer	Dewater Portion of Digested Solids for Offsite Disposal Year-Round	Dewater a Portion of FSL Solids for Offsite Disposal in Winter	Dewater All FSLs Solids for DLD Site Application in Summer
Period of Operation	12 weeks/year	52 weeks/year	20 weeks/year	20 weeks/year
Conveyances	Connect existing dredge piping system to new thickening equipment	Connect digested sludge pipeline to new dewatering equipment/building	Connect existing dredge piping system to new dewatering equipment/building	Connect existing dredge piping system to new dewatering equipment/building
Thickening/ Dewatering Equipment Processing Rate	Three thickening screw presses, sized for 3 percent solids, 270 gpm	Two screw presses, sized for 1.6 percent solids, 80 gpm[(a)]	Two screw presses, sized for 3 percent solids, 70 gpm	Five screw presses, sized for 3 percent solids, 70 gpm
Flow Equalization	100,000 gallon FSL solids storage, 60,000 gallon thickened solids storage tank, 120,000-gallon filtrate storage	80,000 gallon digested solids storage[(b)], 60,000-gallon filtrate storage[(c)]	60,000 gallon FSL solids storage, 50,000-gallon filtrate storage	60,000 gallon FSL solids storage, 110,000-gallon filtrate storage
FSLs	6 FSLs	4 FSLs	6 FSLs	6 FSLs
Building and Odor Control	Building without odor control	Building with odor control and biofilter	Building without odor control	Building without odor control
Other	New dredge to pump 810 gpm from FSLs to thickening equipment	Dewatered sludge conveyance	Dewatered sludge conveyance	Dewatered sludge conveyance Solids spreader truck

(a) One screw press, sized for 6 percent solids and 45 gpm would be required if recuperative thickening were provided on the two small digesters to meet the digestion treatment capacity requirements.

(b) The size of this storage facility would be reduced by approximately 55,000 gallons if recuperative thickening were provided on the two small digesters to meet the digestion treatment capacity requirements.

(c) The size of this storage facility would be reduced by approximately 40,000 gallons if recuperative thickening were provided on the two small digesters to meet the digestion treatment capacity requirements.

8.3.3 Alternatives Comparison

This section presents a comparison of the four alternatives discussed above with respect to capacity, layout, lifecycle costs, and the non-economic factors. However, the alternatives evaluated herein also provide advantages and disadvantages with respect to potential long-term solids handling projects that might occur in the future. Therefore, a recommended strategy for reducing the hydraulic loadings to the DLD site must consider all of these potential future facilities. Thus, a recommended approach is not addressed below, but is instead provided in Chapter 10 of this Master Plan.

8.3.3.1 Disposal Capacity

Table 8-8 presents a summary of the solids handling capacities for each of the alternatives discussed above. As noted previously, the mass of solids to be processed is the same whether anaerobic digester capacity is increased through the construction of a fourth digester or the installation of the recuperative thickening system.

Table 8-8. Solids Handling Capacity for the DLD Hydraulic Limitations

Alternative	Solids from Digester, dt/yr	Digested Solids to Dewatering, dt/yr	Solids From FSL, dt/yr	FSL Solids to Thickening, dt/yr	FSL Solids to Dewatering, dt/yr	FSL Solids to DLD		Digested Solids Offsite, dt/yr
						dt/yr	%	
Thicken All FSL Solids to Apply to DLD	3,210	N/A	2,150	2,150	N/A	2,150	5.1	N/A
Dewater Portion of Digested Solids for Offsite Disposal		1,400	1,210	N/A	N/A	1,210	3.0	1,400
Dewater a Portion of FSL Solids for Offsite Disposal in Winter		N/A	2,150	N/A	940	1,210	3.0	940
Dewater All FSL Solids for DLD Application in Summer		N/A	2,150	N/A	2,150	2,150	18	N/A

8.3.3.2 Facilities Layout

Proposed layouts for the required facilities under all of the biosolids alternatives are shown on Figure 8-5. As shown, the recommended location for these facilities would be in the open space west of FSL 6. The following information is shown on this figure:

- Locations of the existing digested biosolids conveyance to the FSLs, the existing FSL supernatant return line, and the existing harvested solids pipeline.
- Recommended location for a thickening/dewatering building and odor control biofilter. The identified locations allow for truck access along the existing access road and preserve space in the open space for future uses.
- Recommended locations for alum and potential return stream treatment units.
- Suggested approximate location of new pipelines.

8.3.3.3 Lifecycle Costs

A comparison of the capital costs for the four alternatives listed above is provided in Table 8-9. These costs were developed following the procedures detailed in Chapter 5, and cost estimating details are provided in Appendix H-11. As shown, the alternative that involves dewatering a portion of the FSL solids for offsite disposal in the winter is the lowest cost approach. However, one of the key factors in this cost is the assumptions regarding odor control. If a larger building with odor control were required, this alternative would not be as cost competitive.

Table 8-9. Summary of Capital Cost Estimates for DLD Site Capacity Alternatives

Project Component	Estimated Costs by DLD Site Capacity Alternative, million $			
	Thicken All FSL solids for DLD Site Application in Summer	Dewater Portion of Digested Solids for Offsite Disposal Year-Round	Dewater a Portion of FSL Solids for Offsite Disposal in Winter	Dewater All FSLs Solids for DLD Site Application in Summer
Pipeline and Pumps	1.8	0.8	0.9	0.9
Thickening Facilities	4.6	-	-	-
Dewatering Facilities	-	2.7	2.7	6.1
Thickener/ Dewatering Building	5.0	6.6	5.0	5.0
Flow Equalization	3.8	1.9	1.5	2.3
Odor Control	-	1.1	-	-
Solids Spreader Truck	-	-	-	0.5
OPCC	**15.2**	**13.1**	**10.1**	**14.8**
Construction Contingency, 10%	1.5	1.3	1.0	1.5
Total Estimated Construction Cost	**16.7**	**14.4**	**11.1**	**16.3**
Engineering, Legal and Administrative Costs, 35%[(a)]	**5.3**	**4.6**	**3.5**	**5.2**
Total Project Costs	**22.0**	**19.0**	**14.6**	**21.5**

[(a)] Calculated as a percentage of the OPCC

Annual operation costs for the four alternatives are shown in Table 8-10 and the lifecycle costs are presented in Table 8-11. A more detailed breakdown of these operating costs is provided in Appendix I-8. As shown in these tables, the operating costs associated with the dewatering a portion of digested solids for offsite disposal is almost double that of the other alternatives. This is because dewatering of the solids occurs over the entire year, necessitating that significantly more staff time be devoted to the operating the dewatering equipment than for other alternatives.

Table 8-10. Annual Operating Cost Estimates for DLD Site Capacity Alternatives

Project Component	Estimated Costs by DLD Site Capacity Alternative, million $			
	Thicken All FSL solids for DLD Site Application in Summer	Dewater Portion of Digested Solids for Offsite Disposal Year-Round	Dewater a Portion of FSL Solids for Offsite Disposal in Winter	Dewater All FSLs Solids for DLD Site Application in Summer
Chemicals	0.4	0.2	0.2	0.4
Electrical	0.01	0.02	0.01	0.01
Operation and Maintenance	0.7	0.7	0.7	0.7
Hauling and Disposal	0	0.4	0.3	0
Total	**1.1**	**1.3**	**1.2**	**1.1**

Table 8-11. Estimating Lifecycle Costs of Alternatives for Addressing DLD Hydraulic Loading Capacity Limitations

Project Component	Estimated Costs by DLD Site Capacity Alternative, million $			
	Thicken All FSL solids for DLD Site Application in Summer	Dewater Portion of Digested Solids for Offsite Disposal Year-Round	Dewater a Portion of FSL Solids for Offsite Disposal in Winter	Dewater All FSLs Solids for DLD Site Application in Summer
Total Capital Costs	22.0	19.0	14.6	21.5
Total Annual Costs (NPV)	23.8	28.3	25.9	23.8
Total Present Worth Value (Lifecycle Cost)	**45.8**	**47.3**	**40.5**	**45.3**

[(a)] Calculated as a percentage of the OPCC

8.3.3.4 Non-Economic Comparison

Non-economic factors were considered as part of the alternatives analysis. Factors considered include reliability, flexibility, potential for resource recovery, ancillary impacts, and others. A summary of these non-economic factors is presented in Table 8-12. Additional details regarding this subjective comparison are provided in Appendix K-4.

Table 8-12. Non-Economic Comparison of Alternatives for Reducing DLD Site Hydraulic Loadings

Criteria	Weighted Subjective Criteria			
	Thicken All FSL solids for DLD Site Application in Summer	Dewater Portion of Digested Solids for Offsite Disposal Year-Round	Dewater a Portion of FSL Solids for Offsite Disposal in Winter	Dewater All FSLs Solids for DLD Site Application in Summer
Reliability	3.0	7.5	9.0	6.0
Flexibility	3.5	6.1	6.1	5.3
Odor generation potential	3.8	1.5	3.8	3.0
Maximize Available Water Storage Volume	3.8	5.0	3.8	3.8
Energy Management	3.0	4.0	3.0	3.0
Chemical use	1.0	2.0	2.0	1.0
Resource Recovery	1.9	2.9	2.5	2.5
Ancillary Impacts	1.0	1.0	1.3	1.3
Total Subjective Ranking	**20.9**	**30.0**	**31.4**	**25.8**

As shown in Table 8-12, dewatering a portion of the digested solids for offsite disposal and dewatering a portion of the FSL solids for offsite disposal both ranked higher than the other alternatives. Key factors that impacted the analysis are as follows:

- The thickening FSL solids for DLD application ranked the lowest with respect to reliability because it is unclear how effective this alternative will be with respect to operation of the DLD site.
- The alternatives that include dewatering ranked higher for flexibility because dewatering lends itself to more options for long-term solids management. The alternatives that also include offsite disposal ranked the highest.
- All the alternatives have the potential to increase odor generating potential. The thickening and DLD application alternative ranked the highest because solids will be disposed beneath the soil. The dewatering of digested solids ranked the lowest because odor producing potential will be decreased through the digestion that occurs in the FSLs.

- Dewatering of digested solids versus FSL solids was given a higher ranking for maximizing storage volume because it reduces the number of FSLs needed to manage the solids – freeing up that resource for other potential uses.
- Approaches that involve dewatering ranked more favorably for resource recovery because they provide an opportunity to recover bound water and nutrients from the biosolids. Similarly, alternatives that involve thickening or dewatering of all solids ranked higher than the partial dewatering alternatives.
- Alternatives that include offsite disposal provide an opportunity for the solids to be beneficially used (e.g., composted, land applied), while biosolids applied to the DLD will not have that opportunity.
- The added truck traffic from hauling to and from the site could have an adverse ancillary impact at the site, which is reflected in the ancillary impacts scoring.
- The alternatives that involve increases discharge of solids to the DLD site have a higher potential for long-term groundwater impacts, and therefore ranked lower.

8.4 POTENTIAL FUTURE FULL DEWATERING AND OFFSITE DISPOSAL IMPROVEMENTS

In the future, storage of liquid biosolids in the FSLs and/or application of biosolids to the DLD site may no longer be permitted or desired. This change could be driven by new regulations, concerns about odor control at the FSL/DLD site, a need/desire to use the FSLs or DLD area (or a portion thereof) for some other purpose, or increased value of dewatered solids such that DLD disposal is no longer economical. If such restrictions or opportunities were to develop, the District would need to provide full dewatering of the WWTP biosolids. This section presents an overview of the additional facilities that would be needed to provide for full dewatering, depending on which alternative is selected for reducing hydraulic loadings on the DLD site. The topics addressed include:

- Potential Drivers for Full Dewatering and Offsite Disposal
- Strategy for Evaluating Potential Future Full Dewatering and Offsite Disposal Facilities
- Facilities Overview
- Facilities Layout
- Lifecycle Costs

8.4.1 Potential Drivers for Full Dewatering and Offsite Disposal

The Visioning Panel process was used to identify constraints with respect to ongoing operation of the FSLs and DLD site, as well as opportunities with respect to long-term beneficial reuse of the District's biosolids. The following conclusions were made with respect to these factors:

- There are not currently any major regulations anticipated that would preclude the use of the FSLs or DLD site over the long term. Therefore, barring a condition where these facilities are determined to be causing groundwater quality impacts (which is not expected given the dense, clayey soils at the site), the District should be able to continue using the FSLs and DLD site a disposal location for the foreseeable future.
- It is possible that the DLD site could come under increased scrutiny for greenhouse gas emissions. However, at this time only about two percent of the state's biosolids are disposed of in a DLD operation. Thus, regulating this source of greenhouse gasses is not likely to be a priority.
- The District did experience a major odor event associated with the FSLs several years ago. There is a concern that a second event could drive the need to eliminate the FSLs and/or DLD site.
- Agronomic reuse of biosolids continues to be a viable strategy, and farmers are starting to recognize the benefits of this resource as a fertilizer and soil amendment. Although at this time there is no market for biosolids as an agricultural soil amendment, such a market could emerge in the future.

- There may be an emerging market for biosolids as amendment for damaged soils. The Governor's Healthy Soil Initiative had identified benefits of biosolids applications to fire ravaged lands, superfund sites and overgrazed rangelands. However, the timing or viability of this market as a long-term biosolids management approach is uncertain.
- Technologies for converting biosolids to energy or to a fertilizer product are emerging rapidly, and there are a number of projects being implemented throughout the Bay Area that would allow for biosolids reuse. (This includes the supercritical oxidation process being considered for the WWTP site.) Even if these emerging opportunities do not result in a market for the biosolids, they would involve development of a marketable product, which would offset disposal costs.
- The District would likely need to go to a full dewatering from the digesters approach to take advantage of technologies that provide for phosphorus recovery from biosolids because phosphorus levels in the FSLs return flows are not high enough for this process. However, at the current state of the technology, applying phosphorus recovery at the WWTP would not likely be cost effective because of the size of the facility. Nevertheless, the District should remain appraised of advances in phosphorus recovery technologies and look for opportunities to implement a phosphorus recovery strategy, should conditions become more favorable in the future.

8.4.2 Strategy for Evaluating Potential Future Full Dewatering and Offsite Disposal Facilities

The analysis of full dewatering is assumed to build off of the alternatives discussed previously for reducing DLD hydraulic loadings. Thus, the analysis only considers the additional facilities that would need to be constructed to achieve full dewatering, assuming the starting point is one of the previously listed DLD hydraulic loading reduction alternatives.

If the DLD site were eliminated, the District would need to provide full dewatering of all of the FSL solids so that an alternative (likely offsite) disposal option could be pursued. If the FSLs were eliminated, the District would need to provide full dewatering of the digested solids, because there would no longer be a storage location for liquid biosolids. Thus, this analysis considers full dewatering from the FSLs and full dewatering from the digesters, where dewatering from the digesters results in more solids that require dewatering because the District would lose the advantage associated with volatile solids reduction that occurs in the FSLs.

As noted previously, this analysis also assumes that a building sized to accommodate full dewatering would be constructed with the improvements installed to address the DLD hydraulic loading concerns. However, for an FSL solids dewatering approach, it has been assumed that odor control is less of a concern because the solids receive additional digestion in the FSLs, resulting in a Class A product.

Finally, this analysis does not identify a preferred location for offsite disposal of the District's biosolids. As noted above, there are a wide range of options potentially available for disposal/reuse of the District's biosolids. If discontinuation of DLD application becomes necessary, the District will need to complete a more detailed analysis of the options available at the time to make an informed decision on the preferred strategy. Additional discussion regarding potential offsite disposal options that should be considered is presented in Chapter 10 (Recommended Plan).

8.4.3 Facilities Overview

This section presents an overview of the additional facilities needed to convert each of the alternatives previously presented for reducing hydraulic loadings to the DLD site to a full dewatering facility that operates 52 weeks per year, seven days a week, six hours per day. For full dewatering from the FSLs, two screw presses (no redundant unit needed because FSLs can store solids if dewatering equipment fails), sized for 3 percent solids at 70 gpm would be required. For full dewatering from the digesters, four screw presses (one is redundant), sized for 1.6 percent solids at 125 gpm would be required. Four screw presses (one is redundant) would also be required if recuperative thickening was implemented at the digesters. However, the screw presses would be sized for 6.0 percent solids at 35 gpm.

The changes to the dewatering equipment provided under various alternatives previously identified for addressing DLD hydraulic capacity limitations are as follows:

- The alternative based on thickening all FSL solids for DLD site application in summer discussed in Section 8.3.2.1 would be modified by converting the FSL solids thickening equipment to dewatering equipment.
- The alternatives that involve dewatering of a portion of the biosolids discussed in Section 8.3.2.2 and Section 8.3.2.3 would be expanded to provide the additional dewatering equipment needed for year-round solids dewatering.
- The dewatering of all of the FSL solids for DLD application in the summer alternative discussed in Section 8.3.2.4 would not require additional dewatering equipment. However, if switching from dewatering from the FSLs to dewatering directly from the digesters, additional odor control will be required.

Additional discussion of the specific improvements associated with modifying each of the four alternatives previously defined for addressing DLD site hydraulic limitations are described below.

8.4.3.1 Modify the Alternative for Thickening all FSL Solids by Converting FSL Solids Thickening Equipment to Allow for Year-Round FSL or Digested Solids Dewatering

Assuming a screw press technology were used for thickening prior to DLD application, these presses could be converted to dewatering units by installing new screens and adjusting operation and polymer dosing. Other thickening equipment options are also available that would allow for converting to dewatering equipment at a later date. Because there is potential that the District would go to full dewatering in the future, it is recommended that any equipment installed for thickening prior to DLD application be convertible to allow for dewatering if thickening solids prior to DLD application is identified as a preferred strategy for addressing the existing DLD hydraulic loading issue.

For dewatering from the FSLs, only two modified dewatering units, sized for 70 gpm at 3 percent solids, would be required and it is assumed that the District would abandon the other two units. If dewatering from the digesters, all three thickening units would need to be modified and one additional dewatering unit would need to be purchased, a new connection between the digested sludge pipeline and the dewatering building would be needed, and the thickening/dewatering building would need to be modified to provide additional odor control for the dewatering area and truck loading areas. An additional 80,000 gallons of equalization storage would be required between the digesters and the dewatering equipment, for a total of 180,000 gallons, to hold the solids discharged from the digester while the dewatering equipment is not operating. This would provide storage of approximately one day of digested solids flow. Finally, under both full dewatering approaches, a sludge conveyance system would need to be installed to move the dewatered solids from the dewatering equipment to a hauling truck.

8.4.3.2 Expand Seasonal Digested Solids Dewatering Equipment to Provide for Year-Round FSL or Digested Solids Dewatering

No additional dewatering units would be needed to modify the system from partial digested sludge dewatering to full FSL dewatering. However, a new connection between the FSL dredge piping and the dewatering building would be needed, as well as an additional 40,000 gallons of harvested FSL solids equalization storage between the FSLs and the dewatering equipment to hold the FSL solids prior to dewatering (equivalent to two hours of storage).

To expand the partial digested solids dewatering system to accommodate all of the digested solids, two additional screw presses sized to process 125 gpm at 1.6 percent solids would be required. In addition, capacity of the sludge filtrate storage would also need to be increased to provide an additional 70,000 gallons of storage and 100,000 gallons of additional equalization solids storage between the digesters and the dewatering equipment to provide 180,000 gallons of storage.

8.4.3.3 Expand Seasonal FSL Dewatering Equipment to Provide for Year-Round FSL or Digested Solids Dewatering

No additional equipment would be needed to modify the system from partial FSL dewatering to year-round FSL dewatering because the partial dewatering equipment would be sized for a 20-week operation and this same equipment could be used year-round to dewater all of the FSL solids. An additional 60,000 gallons of solids storage between the FSLs and the dewatering equipment would be needed to provide approximately two hours of equalization.

To expand the seasonal FSL solids dewatering system to accommodate all of the digested solids, two additional screw presses sized to process 125 gpm at 1.6 percent solids would be required. In addition, a new connection between the digested sludge pipeline and the dewatering building would be needed, the capacity of the sludge filtrate storage would also need to be increased to provide an additional 80,000 gallons of storage, and 120,000 gallons additional of equalization solids storage between the digesters and the dewatering equipment must be provided so that 180,000 gallons of storage is available. Finally, the dewatering building would need to be modified to provide additional odor control for the dewatering area and truck loading areas.

8.4.3.4 Convert Application of Dewatered Solids on DLD Site to Offsite Disposal

No additional dewatering equipment would be needed to modify the system from full FSL dewatering in the summer to year-round FSL dewatering, because the full dewatering equipment for summer operations would be more than adequate for year-round operation. Indeed, the only difference between these two approaches is whether the solids are applied to the DLD site or hauled offsite. However, an additional 60,000 gallons of dredged FSL solids storage will be needed to hold the dredge solids until they can be dewatered. Additional storage is needed because fewer dewatering units will be operated, increasing the amount of dredged solids to be stored.

The equipment installed for full dewatering of FSL solids in the summer would also be more than adequate to provide full dewatering of all of the digested solids. However, a new connection between the digested sludge pipeline and the dewatering building would be needed, the capacity of the sludge filtrate storage would need to be increased to provide an additional 20,000 gallons of storage, 120,000 gallons of additional digested sludge equalization storage must be provided (for a total of 180,000 gallons of storage), and the dewatering building would need to be modified to provide additional odor control for the dewatering area and truck loading areas.

8.4.3.5 Summary of Additional Facility Needs

Table 8-13 summarizes the additional facilities which would be required for the facility to dewater all solids from either the FSLs or digesters, assuming that one of the intermediate solids handling alternatives discussed in previous sections has been implemented.

Table 8-13. Summary of Additional Facility Needs to Achieve Full Dewatering

Precursor Alternative: Facility Type	Thicken All FSL solids for DLD Site Application in Summer		Dewater Portion of Digested Solids for Offsite Disposal Year-Round		Dewater a Portion of FSL Solids for Offsite Disposal in Winter		Dewater All FSLs Solids for DLD Site Application in Summer	
	From FSLs	From Digesters	From FSLs	From Digesters	From FSLs	From Digesters	From FSLs	From Digesters
Thickening/ Dewatering Equipment	Convert two thickeners to dewatering units	Convert three thickeners to dewatering units	None	Install one additional screw presses	None	Install two additional screw presses	None	
Pumping and Conveyance	None	Install pumping and conveyance to between digested sludge pipeline and dewatering building	New pipeline between FSL dredge piping and dewatering building	None	None	Install pumping and conveyance to connect digested sludge pipeline to dewatering building	None	Install pumping and conveyance to connect digested sludge pipeline to dewatering building
Dewatered Sludge Conveyance	Install dewatered sludge conveyor to connect dewatering equipment to truck loading area		None		None		None	
Additional Flow Equalization	20,000 gallons of FSL solids storage	80,000 gallons of digested solids storage, convert thickened sludge storage to filtrate storage	40,000 gallons of FSL solids storage	100,000 gallons of digested solids storage, 70,000 gallons filtrate storage	60,000 gallons of FSL solids storage	120,000 gallons of digested solids storage, 80,000 gallons filtrate storage	60,000 gallons of FSL solids storage	120,000 gallons of digested solids storage, 20,000 gallons filtrate storage
Building and Odor Control	None	Expand building to provide odor controlled truck loading bay and add biofilter,	None		None	Expand building to provide odor controlled truck loading bay and add biofilter,	None	Expand building to provide odor controlled truck loading bay and add biofilter,
Stranded Assets	Two thickening units; Thickened sludge pumping, conveyance and storage	Thickened sludge pumping to equalization storage	Expanded building and odor control biofilter	None	None	None	Solids spreader truck	
							Three screw press[(b)]	One screw presses

[(a)] Dewatering and hauling period could be decreased to 20 weeks given the number of available screw presses.

8.4.4 Facilities Layout

The proposed facility layouts for the two full dewatering approaches is shown on Figure 8-6. The following information is shown on this figure:

- Recommended location of dewatering building, return stream treatment, and odor control biofilter. The identified locations allow for truck access along the existing access road and preserve space in the open space for additional uses.
- Dewatering all FSL solids would allow for the abandonment of the DLD site, potentially freeing space for other beneficial uses.
- Dewatering all digester solids would enable both the FSLs and the DLD to be put to other uses.

8.4.5 Lifecycle Costs

The costs associated with installing the additional facilities required for full dewatering of all solids will depend on what facilities are installed for addressing the DLD hydraulic capacity limitations. Thus, four different sets of capital cost estimates were developed for the full dewatering upgrades, as follows:

- Table 8-14 provides the estimated costs to provide full, year-round dewatering assuming the equipment has previously been installed for thickening of the FSL solids prior to summer-only DLD injection.
- Table 8-15 provides the estimated costs to provide full, year-round dewatering assuming the equipment has previously been installed for partial year-round dewatering of the digester solids for offsite disposal.
- Table 8-16 provides the estimated costs to provide full, year-round dewatering assuming the equipment has previously been installed for winter-season partial dewatering of the FSL solids for offsite disposal.
- Table 8-17 provides the estimated costs to provide full, year-round dewatering assuming the equipment has previously been installed for full, summer-only dewatering of the FSL solids for application on the DLD site.

These costs were developed following the procedures detailed in Chapter 5, and additional details regarding these capital cost estimates are provided in Appendix H-11.

Table 8-14. Estimated Costs for Full Biosolids Dewatering for the Thicken all FSL Solids for Summer-Only DLD Application Progression

Project Component	Estimated Cost for Thickening All FSL Solids for DLD Site Application[a], million $	Estimated Additional Costs, million $	
		Full Dewatering from FSLs	Full Dewatering from Digesters
Pumping and Conveyance	1.8	0.1	0.1
Thickening Equipment	4.6	0.2	1.4
Thickening/Dewatering Building	5.0	-	1.7
Dewatered Solids Conveyor and Hopper	-	0.3	0.5
Flow Equalization	3.8	0.3	1.1
Odor Control	-	-	1.1
Total Estimated Component Cost	15.2	0.9	5.9
Construction Contingency	1.5	0.1	0.6
Total Estimated Construction Cost	**16.7**	**1.0**	**6.5**
Engineering, Legal and Administrative Costs, 35%[b]	5.3	0.3	2.1
Total Project Costs	**22.0**	**1.3**	**8.6**
Total Project Costs with Initial Project		**23.3**	**30.6**

[a] From Table 8-9.
[b] Calculated as a percentage of the OPCC

Table 8-15. Estimated Costs for Full Biosolids Dewatering for the Partial Dewatering of Digester Solids for Year-Round Offsite Disposal Progression

Project Component	Estimated Cost for Partial Dewatering of Digester Solids[a], million $	Estimated Additional Costs, million $	
		Full Dewatering from FSLs	Full Dewatering from Digesters
Pumping and Conveyance	0.8	0.02	-
Dewatering Equipment	2.7	-	1.5
Thickening/Dewatering Building	6.6	-	-
Flow Equalization	1.9	0.6	2.3
Odor Control	1.1	-	-
Sludge Storage Pumps	0.1	0.1	0.1
Total Estimated Component Cost	13.1	0.7	3.9
Construction Contingency	1.3	0.07	0.4
Total Estimated Construction Cost	**14.4**	**0.8**	**4.3**
Engineering, Legal and Administrative Costs, 35%[b]	4.6	0.2	1.4
Total Project Costs	**19.0**	**1.0**	**5.7**
Total Project Costs with Initial Project		**20.0**	**24.7**

[a] From Table 8-9.
[b] Calculated as a percentage of the OPCC

Table 8-16. Estimated Costs for Full Biosolids Dewatering for the Partial Dewatering of FSL Solids for Winter Offsite Disposal Progression

Project Component	Estimated Cost for Partial Dewatering of FSL Solids[a], million $	Estimated Additional Costs, million $	
		Full Dewatering from FSLs	Full Dewatering from Digesters
Pumping and Conveyance	0.9	0.1	0.1
Dewatering Equipment	2.7	-	2.6
Thickening/Dewatering Building	5.0	-	1.7
Flow Equalization	1.5	0.9	2.7
Odor Control	-	-	1.1
Total Estimated Component Cost	10.1	1.0	8.2
Construction Contingency	1.0	0.1	0.8
Total Estimated Construction Cost	**11.1**	**1.1**	**9.0**
Engineering, Legal and Administrative Costs, 35%[b]	3.5	0.4	2.9
Total Project Costs	**14.6**	**1.5**	**11.9**
Total Project Costs with Initial Project		**16.1**	**26.5**

[a] From Table 8-9.
[b] Calculated as a percentage of the OPCC

Table 8-17. Estimated Costs for Full Biosolids Dewatering for the Full Dewatering of FSL Solids for Summer-Only DLD Application Progression

Project Component	Estimated Cost for Full Dewatering of FSL Solids for DLD Site Application in Summer[a], million $	Estimated Additional Costs, million $	
		Full Dewatering from FSLs	Full Dewatering from Digesters
Pumping and Conveyance	0.9	0.1	0.1
Dewatering Equipment	6.1	-	-
Thickening/Dewatering Building	5.0	-	1.7
Flow Equalization	2.3	0.9	1.9
Odor Control	-	-	1.1
Solids Spreader Truck	0.5	-	-
Total Estimated Component Cost	14.8	1.0	4.8
Construction Contingency	1.5	0.1	0.5
Total Estimated Construction Cost	**16.3**	**1.1**	**5.3**
Engineering, Legal and Administrative Costs, 35%[b]	5.2	0.4	1.7
Total Project Costs	**21.5**	**1.5**	**7.0**
Total Project Costs with Initial Project		**23.0**	**28.5**

[a] From Table 8-9.
[b] Calculated as a percentage of the OPCC

As shown in Tables 8-14 through 8-17, the added cost to expand to a full dewatering option is relatively minor compared to the initial cost for implementing an initial project to mitigate the DLD hydraulic loading limitations. This is primarily attributed to the fact that the initial project will require an investment in the dewatering facility infrastructure, including the construction of a dewatering building. Whereas the full dewatering project will primarily involve installing additional equipment (if necessary).

The operating cost for each alternative is presented in Table 8-18. Additional details regarding these operating cost estimates are provided in Appendix H-11. The major difference between the operating costs for the initial project to mitigate the DLD hydraulic loading limitation and the full dewatering is the hauling and tipping fee costs associated with biosolids disposal. Comparing the operating costs shown in Table 8-18 to the operating costs shown for the initial alternatives in Table 8-10, it is shown that a project that involves full dewatering from the FSLs for offsite disposal would increase the District's biosolids management operating costs by approximately $300,000 per year. Comparing the operating costs for the full dewatering from the FSLs and full dewatering from the digester in Table 8-18, also demonstrates that the added operating costs associated with managing the digested solids (vs the FSL solids) is approximately $700,000 per year.

Table 8-18. Summary of Operating Costs for Full Solids Dewatering, million $

Project Component	Full Dewatering from FSLs	Full Dewatering from Digesters
Chemical	0.4	0.6
Electrical	0.01	0.02
Labor and Maintenance	0.3	0.4
Hauling and Disposal	0.6	0.9
Total Cost	**1.3**	**1.9**

Table 8-19 presents the estimated lifecycle costs for the different alternatives. As shown, even when considering the costs to go to full dewatering, the initial alternative that involves dewatering a portion of the FSL solids for offsite disposal in the winter still is the least expensive approach.

Table 8-19. Estimating Lifecycle Costs to Dewater All Solids, million $

Cost Component	Full Dewatering from FSLs	Full Dewatering from Digesters
Thickening FSL Solids for DLD Application in Summer Progression		
Total Project Costs	23.3	30.6
NPV of Annual Cost	28.1	41.2
Lifecycle Costs	**51.4**	**71.8**
Dewater a Portion of Digested Solids for Offsite Disposal Year-Round Progression		
Total Project Costs	20.0	24.7
NPV of Annual Cost	28.1	41.2
Lifecycle Costs	**48.1**	**65.9**
Dewater Portion of the FSL Solids for Offsite Disposal in Winter Progression		
Total Project Costs	16.1	26.5
NPV of Annual Cost	28.1	41.2
Lifecycle Costs	**44.2**	**67.7**
Dewater All FSL Solids for DLD Site Application in Summer Progression		
Total Project Costs	23.0	28.5
NPV of Annual Cost	28.1	41.2
Lifecycle Costs	**51.1**	**69.7**

8.5 POTENTIAL FUTURE ADDITION OF A SUPERCRITICAL OXIDATION PROCESS

The District is considering partnering with a SCFI to construct an AquaCritox supercritical oxidation solids processing facility at the WWTP site as part of the BABC program. This section presents an overview of the additional facilities that would be necessary to allow the District to provide biosolids to the AquaCritox system and how this system could be compatible with the District's long-term biosolids management goals. Recommendations regarding the potential implementation of a AquaCritox process are provided in Chapter 10.

The topics addressed include:

- Overview of the AquaCritox Process
- Strategy for Evaluating a Potential AquaCritox Process
- Alternatives Overview
- Alternatives Comparison

8.5.1 Overview of the AquaCritox Process

The AquaCritox process involves injecting the biosolids and oxygen into a high pressure (approximately 3,400 psi), high temperature (approximately 1,100 degrees Fahrenheit) environment where the biosolids are oxidized. Once the process is initially started, the oxidation reaction creates enough heat to continue the reaction as long as biosolids are fed into the unit. Byproducts are steam/hot water, CO_2, and an ash of non-organics. The waste heat can also be used to generate electricity or other beneficial uses. The CO_2 can be captured and sold as commercial compressed CO_2 or dry ice. Some constituents of the ash can be refined to pure elements such as phosphorus and other useful materials.

The intent with the new AquaCritox process is that it would serve multiple agencies throughout the Bay Area. The proposed AquaCritox can process up to approximately 25,340 pounds of biosolids per day (expressed as dry weight). The WWTP is expected to produce approximately 15,800 dry pounds of biosolids per day by 2035, and if additional digestion in the FSLs continues, the WWTP will produce approximately 10,600 dry pounds per day (see Table 8-1). Therefore, additional solids, hauled from other facilities in the Bay Area, would be required.

The preferred solids content of the feed solids to the AquaCritox process is 15 to 18 percent. It is expected that the hauled solids from other treatment plants in the Bay Area would have solids contents from 20 to 25 percent (or higher). Thus, one of the key features of using the WWTP site for the AquaCritox process is that the District could contribute solids from the digesters at approximately 1.5 percent solids to mix with the trucked biosolids to achieve the desired solids content without using any additional water.

8.5.2 Strategy for Evaluating a Potential AquaCritox Process

As noted above, SCFI has indicated to the District that the supercritical oxidation process would rely on dilution flow of liquid solids from the digesters to blend with the other biosolids being trucked to the AquaCritox system. Given the AquaCritox processing capacity of 25,340 pounds of biosolids per day (dry weight), the AquaCritox process will require approximately 880 dry tons per year of digested solids at approximately 1.6 percent solids. That represents an approximate 6 percent reduction in solids to the FSLs. However, to address the DLD site hydraulic loading concerns identified previously, a 56 percent reduction in solids flow to the FSL is required. Therefore, sending digested solids to the AquaCritox system alone will not achieve the District's goals with respect to the DLD loading capacity.

For the District, a preferred AquaCritox strategy would provide the most cost-effective strategy for addressing the WWTP's solids handling needs. Thus, a range of options are considered for sending liquid solids with varying solids contents to the AquaCritox system. These options include sending enough thickened solids to AquaCritox to address the DLD capacity limitations, where it is assumed that the thickened solids will provide adequate moisture for mixing with higher solids content biosolids that have been hauled from other agencies. However, it is assumed that FSL Solids would not be sent to AquaCritox, as these solids do not have adequate volatile content to support the process.

For each alternative evaluated, the solids delivered to the AquaCritox process would reduce the volume of solids that require thickening or dewatering under the alternatives previously identified in the Chapter for reducing DLD hydraulic loadings. Thus, this analysis considers potential savings associated with the reduced dewatering volume.

8.5.3 Alternatives Overview

This section presents an overview of the following alternatives that are potentially available for delivering biosolids and dilution flow to the AquaCritox system:

- Undigested Solids to AquaCritox
- Digested Solids to AquaCritox
- Digested and Thickened Solids to AquaCritox

Schematics for each of these alternatives are presented on Figure 8-7 and 8-8.

8.5.3.1 Undigested Solids to AquaCritox

Undigested solids are sent to anaerobic digesters from the DAFT at approximately 3.5 percent solids. It is thus likely that these solids could serve as dilution flow for the hauled biosolids being processed by the AquaCritox system. This approach would reduce the loading to the anaerobic digesters and the FSLs. Moreover, the higher volatile content of the undigested sludge may provide a greater benefit to the AquaCritox process. To facilitate this approach, a dedicated pump station and pipeline from the main WWTP site to the AquaCritox process would be required.

It is assumed that the undigested solids could be delivered continuously. However, if storage and metering the solids into the process is required, the cost and operation of these facilities are assumed to be the responsibility of SCFI.

8.5.3.2 Digested Solids to AquaCritox

Solids leaving the digesters typically have a solids content of approximately 1.6 percent. If a recuperative thickening process were installed on two of the smaller digesters to meet the required digestion capacity needs, the solids content of the digested sludge would increase to 4.6 percent. Thus, digested solids, either with or without recuperative thickening, could also be acceptable as a dilution source for the AquaCritox process. Both of these approaches would reduce the loading to the FSLs and DLD site. To facilitate this approach, a pump station and pipeline would be needed to convey the digested solids from the existing digested solids pipeline to the AquaCritox process.

The digested solids could generally be delivered continuously over a 24-hour period, and like the other alternatives, SCFI would be responsible for storing and metering the digested biosolids to the AquaCritox process.

8.5.3.3 Digested and Thickened Solids to AquaCritox

The mass of biosolids that can be delivered to the AquaCritox process under the scenarios listed above is limited by 1) the amount of dilution water required to lower the solids content of the trucked solids to the 15 to 18 percent target of the AquaCritox process and 2) the solids content of the liquid biosolids provided by the District. It may be possible that the AquaCritox system could accept solids from the District that are at a solids content that is low enough to mix with the trucked biosolids and still achieve a target 15 to 18 percent solids content, but high enough that the District could use this method to dispose of all of the WWTP biosolids that exceed the disposal capacity of the DLD site. Assuming this is a viable option, this approach would eliminate the need to implement one of the other alternatives identified herein for reducing DLD site hydraulic loadings.

For this alternative, a pump station and pipeline to convey the thickened solids from the new thickening building to the AquaCritox process would be needed in addition to the thickening facilities. However, it is assumed that SCFI would be responsible for storing and metering the digested biosolids to the AquaCritox process.

As shown in Table 8-8, the District would need to dispose of 1,400 dt/yr of biosolids from the digester to accommodate the DLD loading restrictions. At a 1.6 percent solids content, this translates to approximately 58,000 gallons per day of digested solids flow. The District would pump this flow from the digested solids pipeline at rate of approximately 200 gpm over a six-hour period each day to the thickening units. Solids not sent to the thickening units would be discharged to the FSLs for eventual application on the DLD site.

It is also assumed for this alternative that SCFI would be responsible for storing the thickened solids until needed for blending with hauled solids.

8.5.4 Alternatives Comparison

This section presents a comparison of the four alternatives discussed above with respect to disposal capacity, relationships to other evaluations presented in the Chapter, facilities layout and lifecycle costs. A recommended strategy for incorporating a potential AquaCritox into the overall long-term solid handling strategy at the WWTP is provided in Chapter 10 of this Master Plan.

8.5.4.1 Disposal Capacity

Table 8-20 presents a summary of the solids that would be sent to the AquaCritox process and the remaining solids that would require disposal for each of the four alternatives discussed above, assuming the offsite biosolids that are hauled to the AquaCritox system have a 25 percent solids content. This analysis also shows the amount of solids that require alternative disposal both with and without the FSLs in operation. As shown, if the AquaCritox system can use thickened solids at 8 to 9 percent solids as a dilution source for the offsite solids, this alternative would eliminate the need to dispose of biosolids in any other manner.

Table 8-20. AquaCritox Alternatives Solids Disposal Capacities

Alternative	Solids to AquaCritox to Achieve 15 Percent Solids in Feed[a]			Digested Solids to FSL	Digested Solids from FSL	Unthickened Solids to DLD	Remaining FSL Solids Requiring Alternative Disposal[b]	Remaining Digested Solids Requiring Alternative Disposal[c]
	dt/yr	Percent Solids	Percent Volatile Solids	dt/yr	dt/yr	dt/yr	dt/yr	dt/yr
Undigested Solids	410	3.4	77	3,020	2,020	1,210	810	1,810
Digested Solids (Fourth Digester)	180	1.6	53	3,030	2,080		870	1,820
Digested Solids (Recuperative Thickening)	590	4.6	53	2,620	1,790		580	1,410
Digested and Thickened Solids	1,400	8.4	53	1,810	1,210		0	0

[a] Assumes other solids fed to the AquaCritox process would be at a 25 percent solids content.
[b] If all solids not directed to the AquaCritox process would be further stabilized in the FSLs
[c] If FSLs are decommissioned.

8.5.4.2 Beneficial Impacts on Downstream Process Capacity Requirements

The information presented in Table 8-20 can also be used to estimate the potential benefits of each of the alternatives with respect to loading and capacity requirements for the downstream solids handling facilities. The following specific conclusions can be drawn:

- Sending undigested solids to the AquaCritox system will reduce the solids loading to the digesters by approximately 410 dt/yr, or 2,200 pounds per day. This is an estimated 1,800 pound per day reduction in volatile solids loading to the digesters, and an estimated reduction in hydraulic loading of 8,000 gallons per day. Comparing these values to the digestion capacity information presented in Table 8-2 demonstrates that the reduced loadings to the digesters would not eliminate the need for a fourth digester or recuperative thickening. However, this approach would free up digestion capacity to allow for increased handling of a higher volatile content hauled waste, like FOG.

- Sending undigested solids to the AquaCritox system will reduce the solids loading to the FSLs by approximately 190 dt/yr, 1,000 pounds per day. This is an estimated 500 ppd reduction in volatile solids loading to the FSLs. As shown in Table 8-1, it is anticipated that the volatile solids loading to the FSLs in 2035 will be approximately 9,300 ppd. Thus, the undigested solids to AquaCritox alternative would reduce the volatile solid loading to the FSLs to 8,800 ppd. Reviewing the FSL capacities provided in Chapter 3, Table 3-17 reveals that without FSL 6 (the smallest FSL) the firm capacity of the FSLs for volatile solids would be 8,800 pounds per day. Thus, this alternative would allow the District to decommission FSL 6 (or rehabilitate it for another purpose).
- Sending digested solids to the AquaCritox system from a fourth digester will have a similar impact on FSL volatile solids loadings as sending undigested solids. Thus, this alternative would also allow the District to decommission one of the FSL ponds (or rehabilitate it for another purpose).
- Sending digested solids to the AquaCritox system from a recuperative thickening process will reduce the solids loading to the FSLs by approximately 590 dt/yr, 3,200 pounds per day. This is an estimated 1,700 pounds per day reduction in volatile solids loading to the FSLs. As shown in Table 8-1, it is anticipated that the volatile solids loading to the FSLs in 2035 will be approximately 9,300 pounds per day. Thus, the undigested solids to AquaCritox alternative would reduce the volatile solid loading to the FSLs to approximately 7,600 pounds per day. Reviewing the FSL capacities provided in Chapter 3, Table 3-17 reveals that the District could decommission one FSL pond under this alternative (or rehabilitate it for another purpose).
- Sending digested and thickened solids to the AquaCritox system will reduce the solids loading to the FSLs by approximately 1,400 dt/yr, or 7,700 pounds per day. This is an estimated 4,100 pound per day reduction in volatile solids loading to the FSLs. Comparing this reduction to Table 8-1, it is anticipated that the volatile solids loading to the FSLs in 2035 would be reduced to approximately 5,200 pounds per day. Reviewing the FSL capacities provided in Chapter 3, Table 3-17 reveals that two FSL ponds could be decommissioned (or rehabilitated for another purpose).

Table 8-21 provides a summary of the beneficial impacts on the downstream process capacity requirements.

Table 8-21. Benefits of AquaCritox Alternatives on Downstream Process Solids Capacities

Type of Solids Sent to AquaCritox	Potential for Additional VSS Loading Capacity in Digesters pounds of VSS per day	Requirements for FSL Operations
Undigested Solids	1,800	Opportunity to Decommission/Repurpose One FSL
Digested Solids (Fourth Digester)	0	Opportunity to Decommission/Repurpose One FSL
Digested Solids (Recuperative Thickening)	0	Opportunity to Decommission/Repurpose One FSL
Digested and Thickened Solids	0	Opportunity to Decommission/Repurpose Two FSLs

8.5.4.3 Beneficial Impacts on Facilities Required for Addressing DLD Site Hydraulic Loading Limitations

For the three AquaCritox alternatives that involve sending liquid (non-thickened) solids to the AquaCritox process, there would be a reduced amount of solids that would require processing and disposal in a manner alternative to the DLD. Thus, there would be savings associated with the alternatives previously presented for reducing DLD site hydraulic loadings. Table 8-22 provides a summary of these potential benefits in terms of reduction in facilities and potential project cost savings.

As shown in Table 8-22, the "Dewater All Digested Solids for DLD Application in Summer" alternative would be impacted the most in terms of reduced facilities and potential savings. However, even with the savings, the two alternatives that involve dewatering a portion of digested solids would still less expensive. Therefore, the SCFI alternatives do not impact the conclusion that the partial dewatering options are the lowest cost approach to addressing the DLD capacity limitations.

Table 8-22 also shows that sending digested solids to AquaCritox from a recuperative thickening process (at 4.6 percent solids) has the biggest impact in terms of savings, ranging from $2.4 million to $4.8 million in savings over the other approaches. However, even with these savings, the project lifecycle cost for the recuperative thickening alternative is still higher than the lifecycle cost associated with the fourth digester alternative (see Table 8-5).

Table 8-22. Facility and Total Project Cost Savings of AquaCritox Options for Alternatives Previously Identified Addressing DLD Hydraulic Capacity Limitations

SCFI Project Alterantive		Thicken FSL Solids and Apply to DLDs		Dewater Portion of Digested Solids for Offsite Disposal		Dewater a Portion of FSL Solids for Offsite Disposal in Winter		Dewater All FSL Solids for DLD Application in Summer	
		Facilities	Costs	Facilities	Costs	Facilities	Costs	Facilities	Costs
No Solids to AquaCritox™	Required Facilities	• Four thickening screw presses (sized for 3 percent solids, 270 gpm) • 60,000 gallons FSL solids storage[(a)] • 120,000 gallon of filtrate storage	$22.0	• Three screw presses (sized for 1.6 percent solids, 80 gpm) • 80,000 gallons digested solids storage[(a)] • 60,000 gallon of filtrate storage	$19.0	• Two screw presses (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage • 50,000 gallon of filtrate storage	$14.6	• Five screw presses (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage[(a)] • 110,000 gallon of filtrate storage	$21.5
Undigested Solids to AquaCritox™	Required Facilities	• Three thickening screw presses (sized for 3 percent solids, 250 gpm) • 60,000 gallons FSL solids storage[(a)] • 110,000 gallons of filtrate storage	$20.4 ($1.9 in savings)	• Three screw presses (sized for 1.6 percent solids, 65 gpm) • 65,000 gallons digested solids storage[(a)] • 45,000 gallons of filtrate storage	$18.6 ($0.4 in savings)	• Two screw presses (sized for 3 percent solids, 65 gpm) • 60,000 gallons FSL solids storage[(a)] • 40,000 gallons of filtrate storage	$14.5 ($0.1 in savings)	• Three screw presses (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage[(a)] • 100,000 gallons of filtrate storage	$17.8 ($3.7 in savings)
	Facilities Eliminated	• One less screw press • 10,000 gallons less filtrate storage		• 15,000 gallons less digested solids storage • 15,000 gallons less filtrate storage		• 10,000 gallons less filtrate storage		• Two less screw presses • 10,000 gallons less filtrate storage	
Digested Solids to AquaCritox™ (Fourth Digester)	Required Facilities	• Three thickening screw presses (sized for 3 percent solids, 260 gpm) • 60,000 gallons FSL solids storage[(a)] • 115,000 gallons of filtrate storage	$20.4 ($1.9 in savings)	• Three screw presses (sized for 1.6 percent solids, 70 gpm) • 70,000 gallons digested solids storage[(a)] • 50,000 gallons of filtrate storage	$18.6 ($0.4 in savings)	• Two screw presses (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage[(a)] • 40,000 gallons of filtrate storage	$14.5 ($0.1 in savings)	• Three screw presses (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage[(a)] • 100,000 gallons of filtrate storage	$17.8 ($3.7 in savings)
	Facilities Eliminated	• One less screw press • 5,000 gallons less filtrate storage		• 10,000 gallons less digested solids storage • 10,000 gallons less filtrate storage		• 10,000 gallons less filtrate storage		• Two less screw presses • 10,000 gallons less filtrate storage	
Digested Solids to AquaCritox™ (Recuperative Thickening)	Required Facilities	• Two thickening screw presses (sized for 3 percent solids, 300 gpm) • 60,000 gallons FSL solids storage[(a)] • 60,000 gallons of filtrate storage	$17.2 ($4.8 in savings)	• Two screw press (sized for 4.6 percent solids, 25 gpm) • 15,000 gallons of digested solids storage[(a)] • 10,000 gallons of filtrate storage	$14.9 ($4.1 in savings)	• One screw press (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage[(a)] • 25,000 gallons of filtrate storage	$12.2 ($2.4 in savings)	• Three screw presses (sized for 3 percent solids, 70 gpm) • 60,000 gallons FSL solids storage[(a)] • 80,000 gallons of filtrate storage	$17.7 ($3.9 in savings)
	Facilities Eliminated	• Two less thickening screw press • 60,000 gallons less filtrate storage		• One less screw presses • 65,000 gallons less digested solids storage[(a)] • 50,000 gallons less filtrate storage		• One less screw press • 25,000 gallons less filtrate storage		• Two less screw presses • 20,000 gallons less filtrate storage	

[(a)] Storage dredged FSL solids or digester solids is assumed to be needed to equalize flows prior to processing solids in thickening/dewatering equipment.

8.5.4.4 Facilities Layout

A potential size and layout of the AquaCritox facility was provided by SCFI. The potential configuration of the AquaCritox facilities in concert with the location of additional thickening and dewatering facilities, and what existing facilities may be abandoned are shown on the following figures:

- Figures 8-9 presents the layout for the undigested solids to AquaCritox alternative. As shown, this alternative would require separate thickening/dewatering facilities to handle the excess solids that cannot be sent to AquaCritox and/or applied to the DLD site.
- Figure 8-10 presents the layouts for the alternative involving supplying digested solids to AquaCritox either from recuperative thickening for from a fourth digester. As shown, separate thickening/dewatering facilities are required to handle the solids not sent to AquaCritox and/or applied to the DLD site. For this analysis, it has been assumed that the footprint of the dewatering building is the same regardless of whether the solids have been digested with a recuperative thinking process or conventionally. However, it is possible that the building size could be reduced of recuperative thickening is applied for solids digestion.
- Figure 8-11 presents the layout for sending the thickened solids to AquaCritox from the digesters. As shown, this alternative allows for two FSLs to be decommissioned.

8.5.4.5 Facilities Costs

Costs associated with the conveyance of liquid biosolids to the AquaCritox process (either undigested solids or solids from the digesters) or are shown in Table 8-23. These costs were developed following the procedures detailed in Chapter 5, and additional details regarding these capital cost estimates are provided in Appendix H-12-i.

Table 8-23. Estimated Cost for Facilities Required to Deliver Liquid Biosolids to AquaCritox

Project Component	Estimated Costs by Alternative, million $	
	Undigested Solids	Digested Solids (Fourth Digester or Recuperative Thickening)
Pipeline	1.4	0.01
Pumping	0.1	0.1
OPCC Total	**1.5**	**0.1**
Construction Contingency, 10%	0.2	0.01
Total Estimated Construction Cost	**1.7**	**0.1**
Engineering, Legal and Administrative Costs, 35%[(a)]	0.5	0.04
Total Project Costs	**2.2**	**0.2**

[(a)] Calculated as a percentage of the OPCC

Comparing Table 8-22 to Table 8-23 shows that, for the two lowest cost DLD capacity options (i.e., partial dewatering), the savings associated with the AquaCritox options generally offset by the added cost of facilities for conveying liquid biosolids to the AquaCritox process.

Table 8-24 shows the estimated costs for thickening all of the solids that cannot be applied to the DLD site and sending them to AquaCritox. These costs were developed following the procedures detailed in Chapter 5, and additional details regarding these capital cost estimates are provided in Appendix H-13.

Table 8-24. Summary of Capital Cost Estimates for Supplying Thickened Solids to AquaCritox

Project Component	Estimated Costs to Thicken a Portion of Digested Solids to Send to AquaCritox, million $
Pipeline	0.04
Pumps	0.2
Thickening Equipment	2.5
Thickener Building	5.0
Return Flow EQ	0.7
Odor Control	1.1
OPCC	**9.5**
Construction Contingency, 10%	1.0
Total Estimated Construction Cost	**10.5**
Engineering, Legal and Administrative Costs, 35%[(a)]	3.3
Total Project Costs	**13.8**

[(a)] Calculated as a percentage of the OPCC

Comparing Tables 8-24 and Table 8-9 shows that the costs associated with thickening and sending solids to SCFI are competitive with dewatering a portion of FSL solids for offsite disposal in the winter months (i.e., the lowest costs alternative for addressing the DLD hydraulic loading limitations).

8.5.4.6 Potential Solids Handling Process Cost Savings

The final factor to consider with respect to the AquaCritox options is the potential for savings related to offsite disposal of dewatered biosolids. As noted previously, the only AquaCritox options that would eliminate the need for offsite disposal are the alternatives that also involve thickening of solids that are sent to the AquaCritox process. Table 8-25 presents a summary of the anticipated savings over a 30-year lifecycle, assuming SCFI does not charge the District for accepting the liquid solid waste. As shown, sending digested solids from a recuperative thickening process has the highest potential benefits with respect to long-term savings. Combined with the capital savings shown in Table 8-22, the savings realized from this approach are similar to the added total project costs for recuperative thickening over the fourth digester.

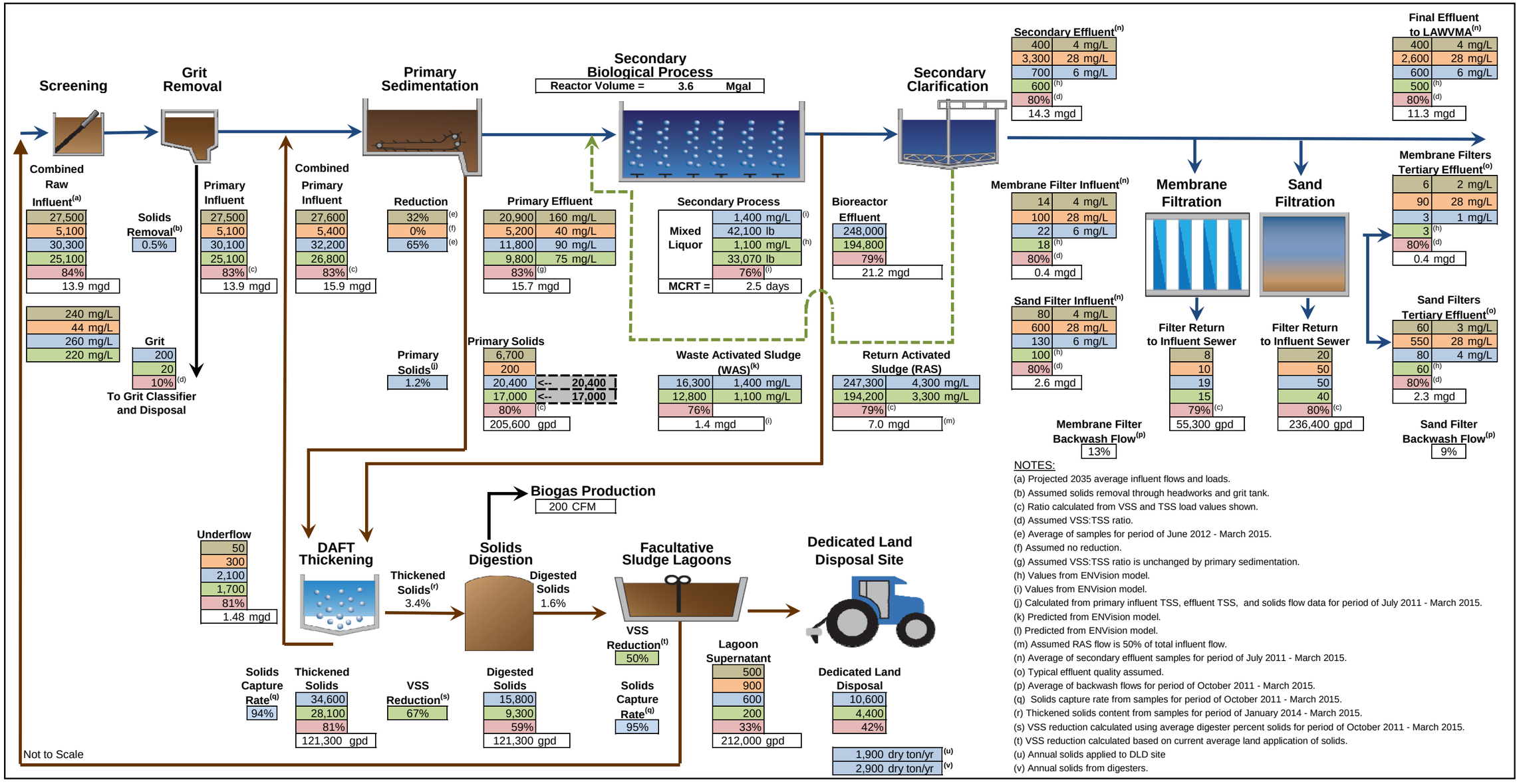
If it is determined that the AquaCritox operates best by mixing very dilute solids with a dewatered sludge, it may be necessary for the District to deliver both liquid solids and dewatered solids to the AquaCritox process. The liquid solids would be delivered at a rate that meets the needs of the AquaCritox system. The dewatered solids would be sent at a rate that addresses the DLD site limitations, in accordance with the selected alternatives for reducing the DLD hydraulic loadings. Under this scenario, the savings over offsite disposal would depend on difference between the cost of sending the thickened solids to AquaCritox and the cost of offsite disposal.

Table 8-25. Estimated 30-Year Lifecycle Costs Savings Associated with Sending Solids to AquaCritox[a]

Project Component	Estimated Costs by Alternative, million $			
	Undigested Solids	Digested Solids with Fourth Digester	Digested Solids with Recuperative Thickening on Digesters	Thickened Solids
Estimated Annual Savings in Offsite Solids Disposal Cost[b]	0.04	0.02	0.1	[c]
NPV of Annual Disposal Cost	0.9	0.4	2.6	

[a] There would be no offsite disposal cost savings for the thicken solids prior to DLD application alternative. However, some District labor savings may be realized.
[b] Assumes a $50 per wet ton hauling and tipping fee.
[c] All solids would go to AquaCritiox, so savings would be based on whatever tipping fee was charged.

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NOTES:

(a) Projected 2035 average influent flows and loads.
(b) Assumed solids removal through headworks and grit tank.
(c) Ratio calculated from VSS and TSS load values shown.
(d) Assumed VSS:TSS ratio.
(e) Average of samples for period of June 2012 - March 2015.
(f) Assumed no reduction.
(g) Assumed VSS:TSS ratio is unchanged by primary sedimentation.
(h) Values from ENVision model.
(i) Values from ENVision model.
(j) Calculated from primary influent TSS, effluent TSS, and solids flow data for period of July 2011 - March 2015.
(k) Predicted from ENVision model.
(l) Predicted from ENVision model.
(m) Assumed RAS flow is 50% of total influent flow.
(n) Average of secondary effluent samples for period of July 2011 - March 2015.
(o) Typical effluent quality assumed.
(p) Average of backwash flows for period of October 2011 - March 2015.
(q) Solids capture rate from samples for period of October 2011 - March 2015.
(r) Thickened solids content from samples for period of January 2014 - March 2015.
(s) VSS reduction calculated using average digester percent solids for period of October 2011 - March 2015.
(t) VSS reduction calculated based on current average land application of solids.
(u) Annual solids applied to DLD site
(v) Annual solids from digesters.

LEGEND:

1,000	BOD Value (lb/day, unless noted otherwise)
1,000	NH_4-N Value (lb/day, unless noted otherwise)
1,000	TSS Value (lb/day, unless noted otherwise)
1,000	VSS Value (lb/day, unless noted otherwise)
1,000	Assumed VSS percentage of TSS
1,000	Flow (units as indicated)
1,000	Values used for iterative process

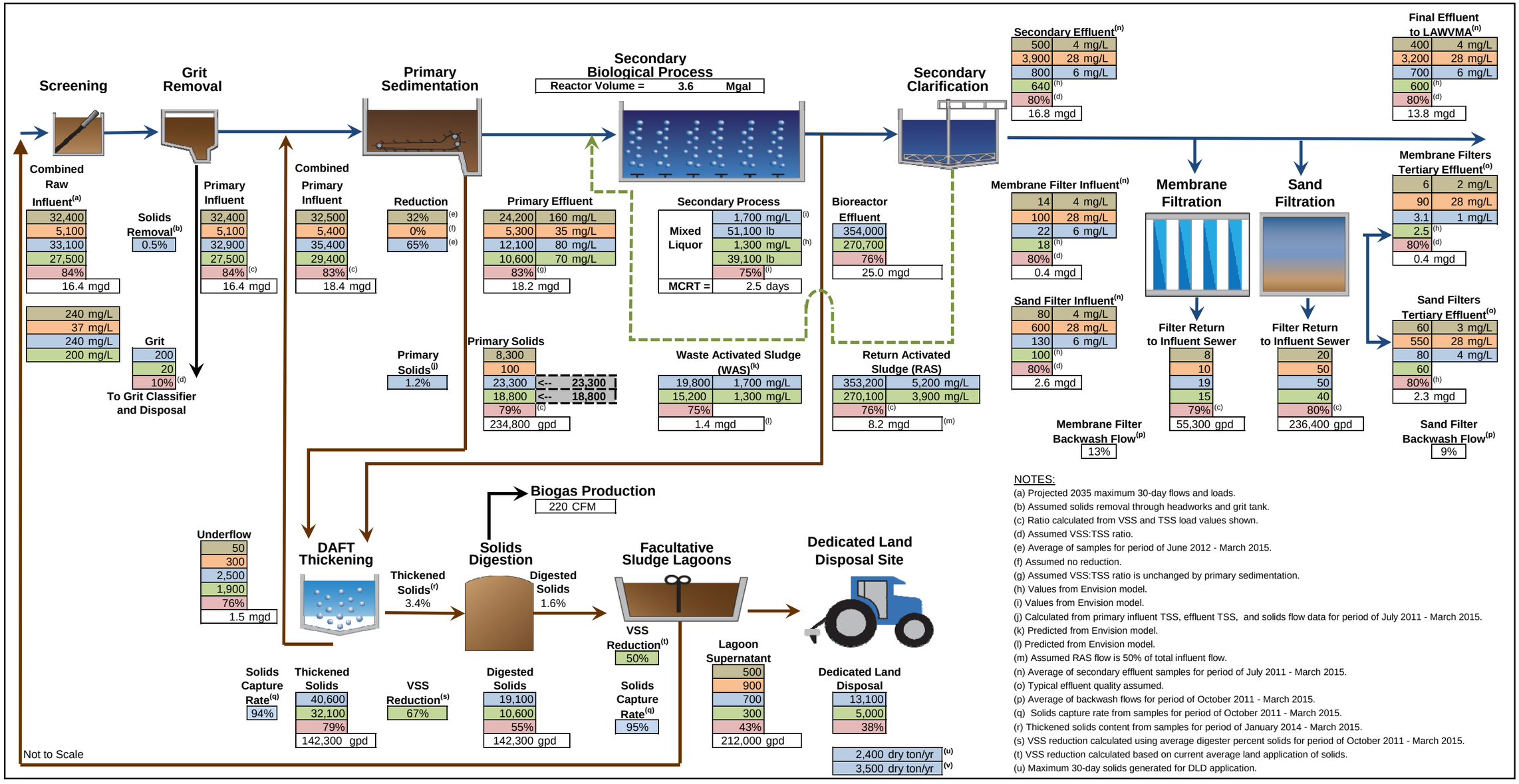
Solids flow through liquid treatment
Solids flow through solids treatment
Solids flow recycle streams
Solids flow external to mass balance (grit disposal)

WEST YOST ASSOCIATES
Consulting Engineers

Figure 8-1

2035 Average Solids Balance – 65% Capture in Primary Sedimentation Basins

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



NOTES:

(a) Projected 2035 maximum 30-day flows and loads.
(b) Assumed solids removal through headworks and grit tank.
(c) Ratio calculated from VSS and TSS load values shown.
(d) Assumed VSS:TSS ratio.
(e) Average of samples for period of June 2012 - March 2015.
(f) Assumed no reduction.
(g) Assumed VSS:TSS ratio is unchanged by primary sedimentation.
(h) Values from Envision model.
(i) Values from Envision model.
(j) Calculated from primary influent TSS, effluent TSS, and solids flow data for period of July 2011 - March 2015.
(k) Predicted from Envision model.
(l) Predicted from Envision model.
(m) Assumed RAS flow is 50% of total influent flow.
(n) Average of secondary effluent samples for period of July 2011 - March 2015.
(o) Typical effluent quality assumed.
(p) Average of backwash flows for period of October 2011 - March 2015.
(q) Solids capture rate from samples for period of October 2011 - March 2015.
(r) Thickened solids content from samples for period of January 2014 - March 2015.
(s) VSS reduction calculated using average digester percent solids for period of October 2011 - March 2015.
(t) VSS reduction calculated based on current average land application of solids.
(u) Maximum 30-day solids generated for DLD application.

LEGEND:

1,000 BOD Value (lb/day, unless noted otherwise)
1,000 NH_4-N Value (lb/day, unless noted otherwise)
1,000 TSS Value (lb/day, unless noted otherwise)
1,000 VSS Value (lb/day, unless noted otherwise)
1,000 Assumed VSS percentage of TSS
1,000 Flow (units as indicated)
1,000 Values used for iterative process

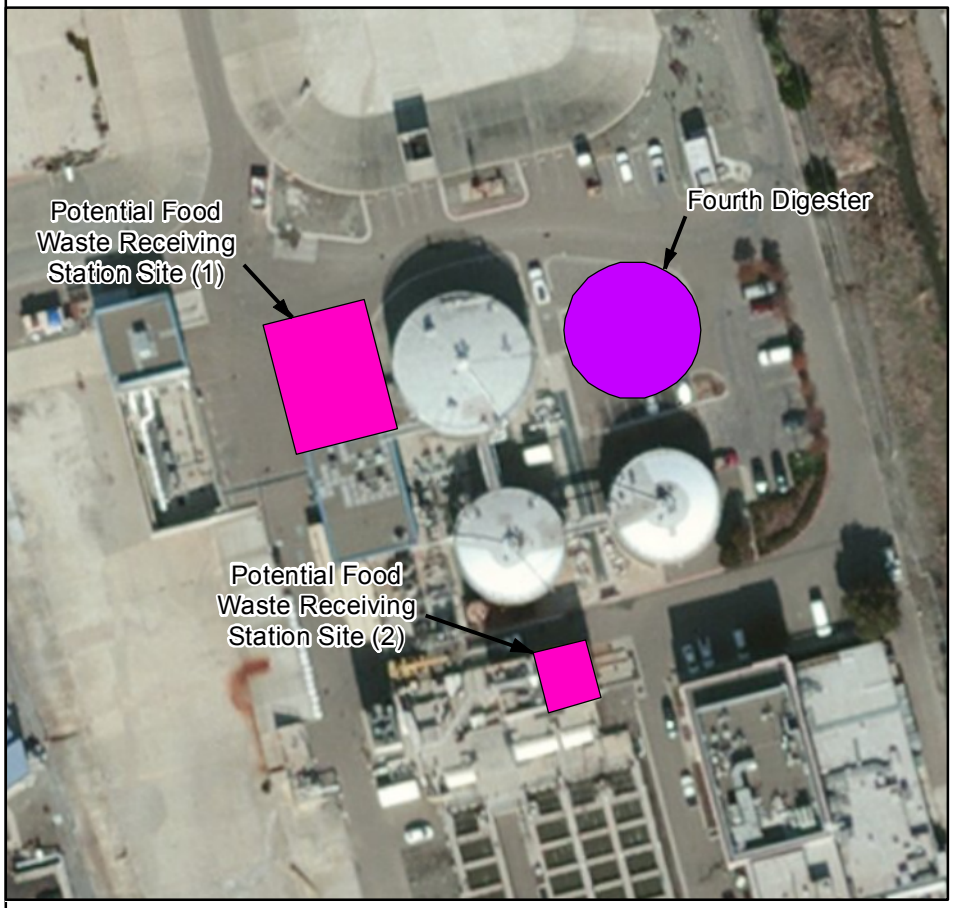
Solids flow through liquid treatment
Solids flow through solids treatment
Solids flow recycle streams
Solids flow external to mass balance (grit disposal)

WEST YOST ASSOCIATES
Consulting Engineers

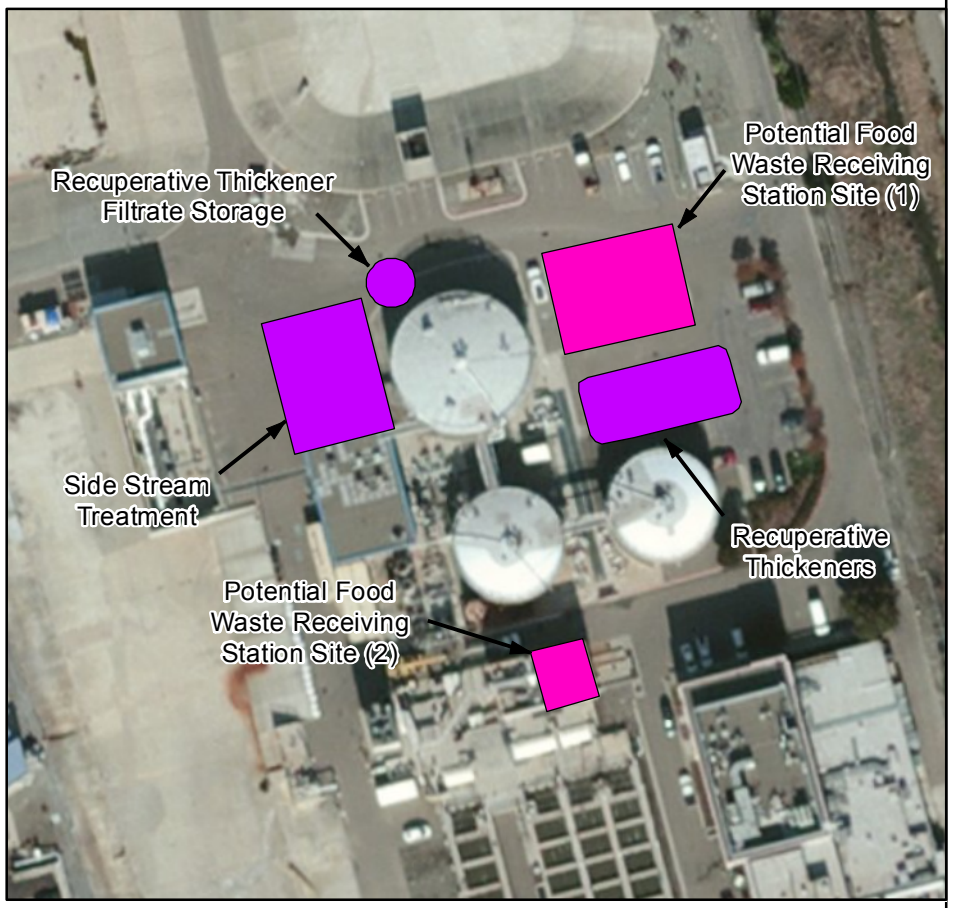
Figure 8-2
2035 Maximum 30-Day Solids Balance – 65% Capture in Primary Sedimentation Basins
Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Fourth Digester and
Waste Receiving Station



Omnivore Thickening and
Waste Receiving Station



Symbology

- Potential Solids Treatment Facilities
- Potential Facilities

Note: Side Stream Treatment only required for BACWA Level 3 Nutrient Removal

N

0 50 100

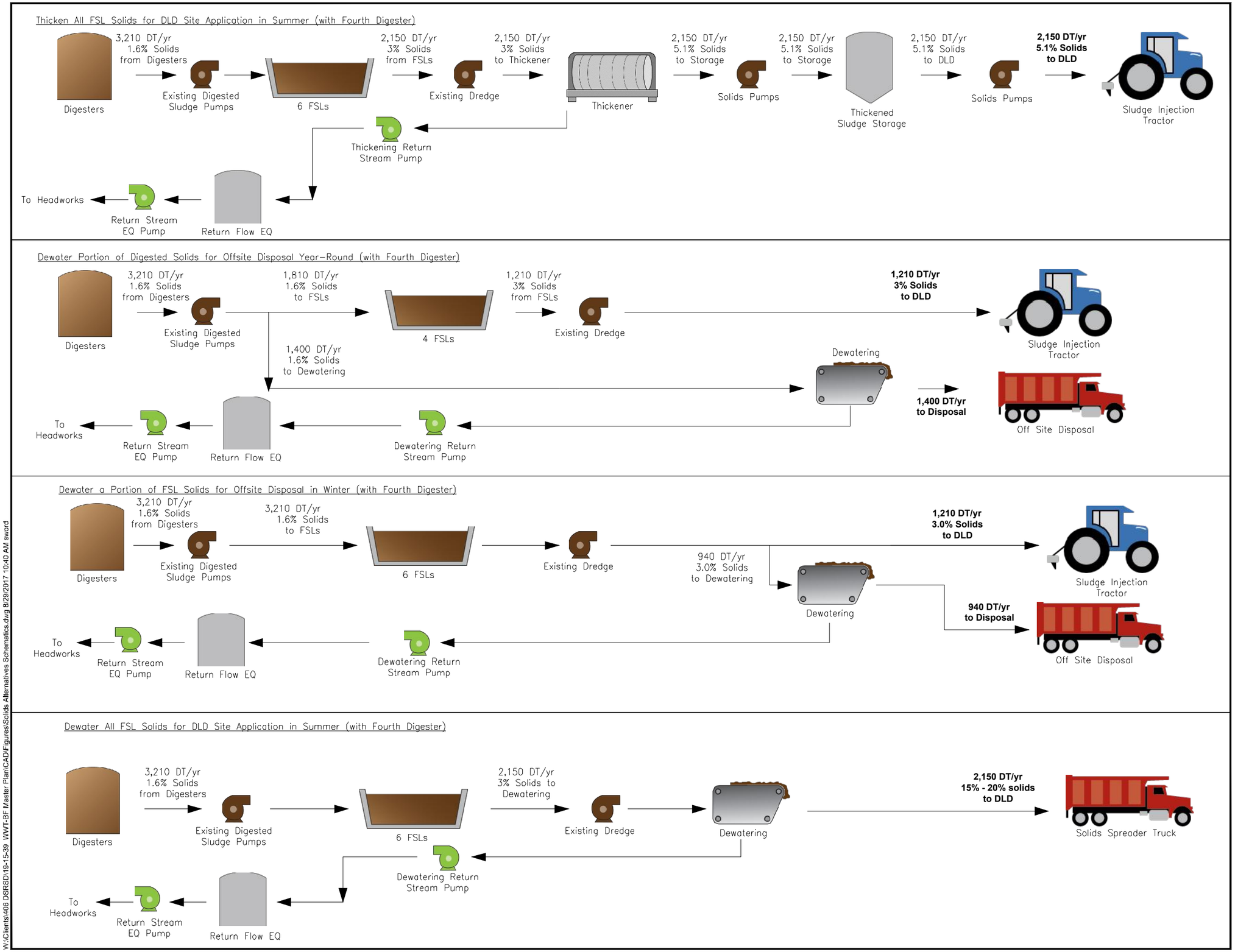
Scale in Feet

WEST YOST ASSOCIATES

Figure 8-3

Alternatives for Increasing Anaerobic Digestion Capacity

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



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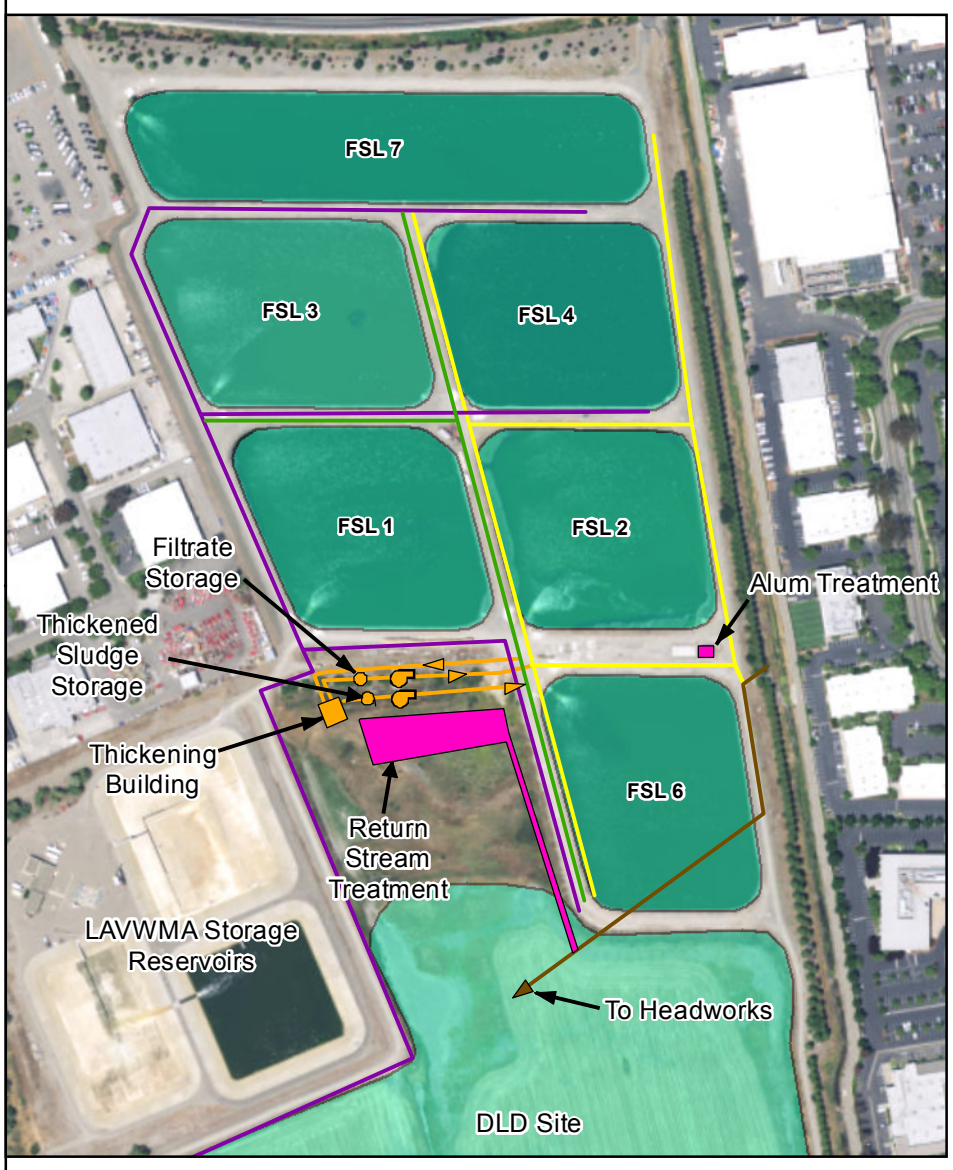
Figure 8-4

Schematics of Alternatives for Reducing DLD Site Hydraulic Loadings

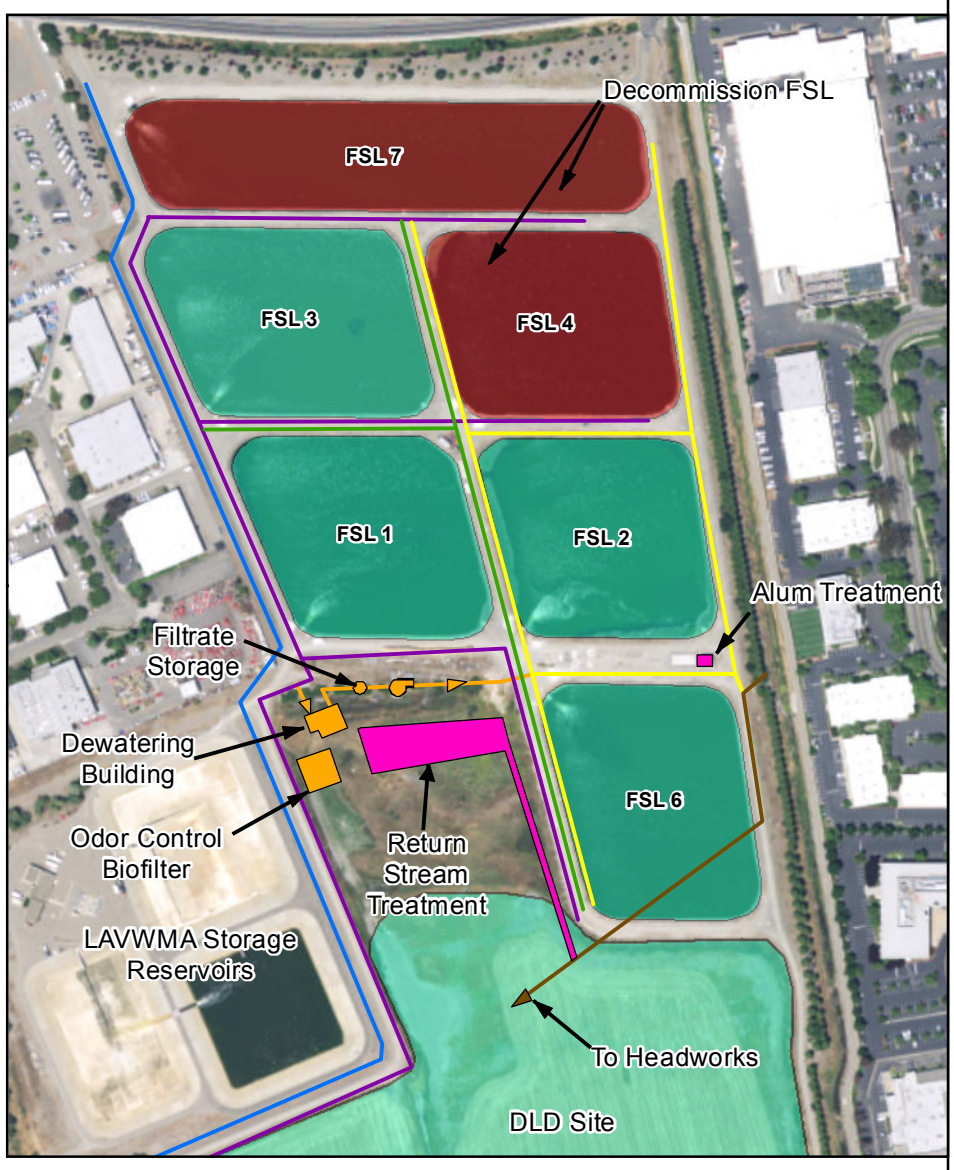
Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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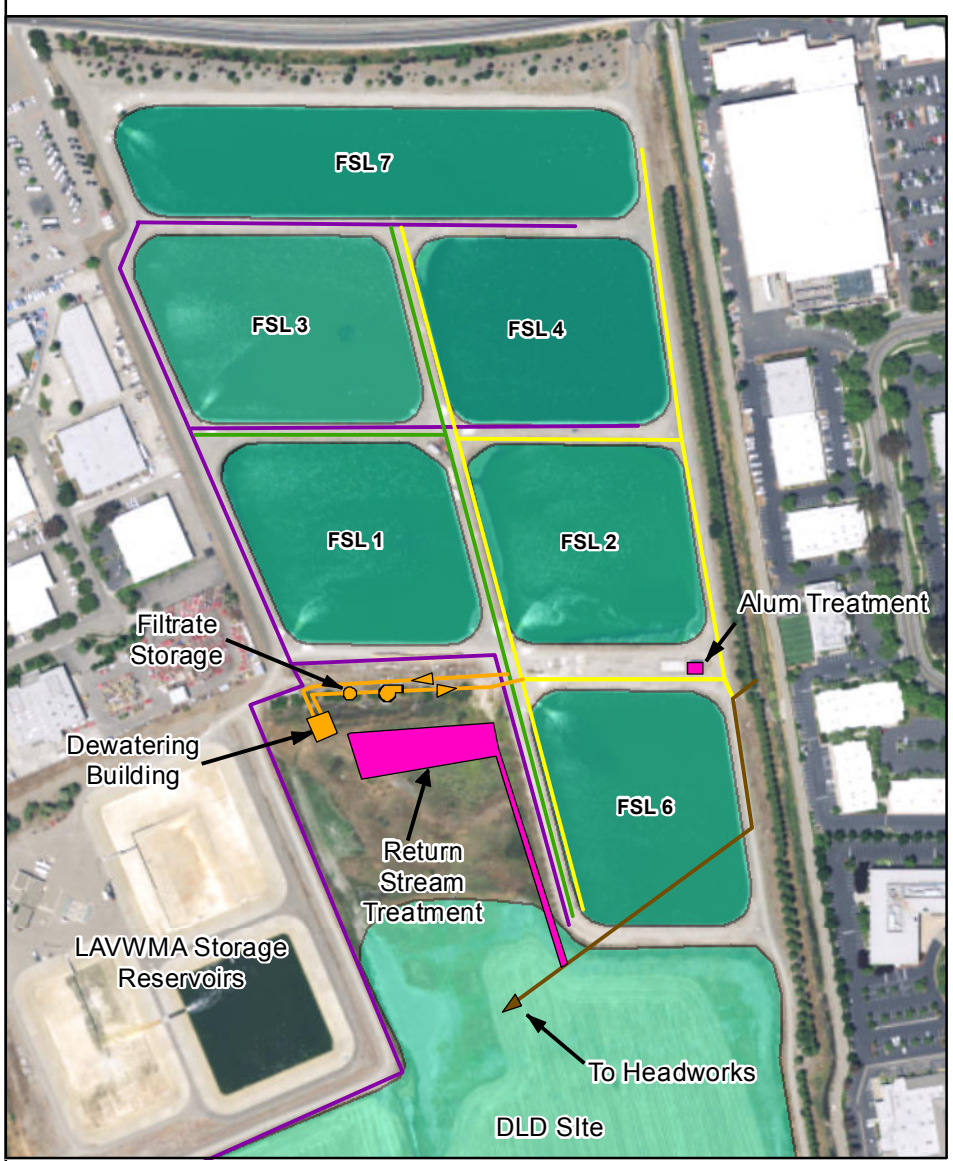
Thicken All FSL Solids for DLD Site Application in Summer



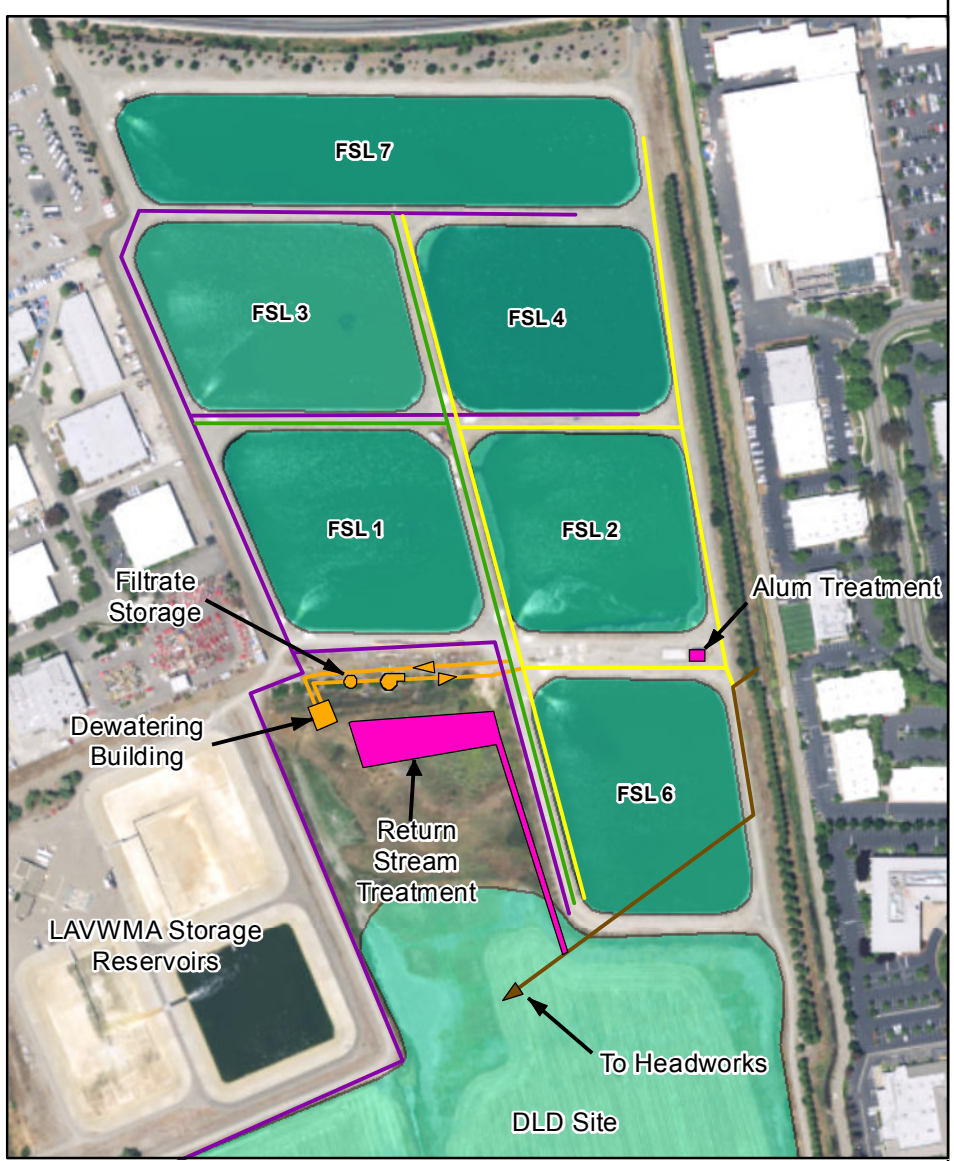
Dewater a Portion of Digested Solids for Offsite Disposal Year-Round



Dewater a Portion of FSL Solids for Offsite Disposal in Winter



Dewater all FSL Solids for DLD Site Application in Summer



Symbology

- Potential Thickening/Dewatering Facilities
- Existing Facilities
- Potential Long-Term Facilities
- Abandoned Facilities-Legend
- Harvested Biosolids Pipeline
- Existing FSL Supernatant Pipeline
- Digested Biosolids Feed Pipeline
- Truck Route
- Existing Sanitary Sewer

N

0 200 400

Scale in Feet

WEST YOST ASSOCIATES

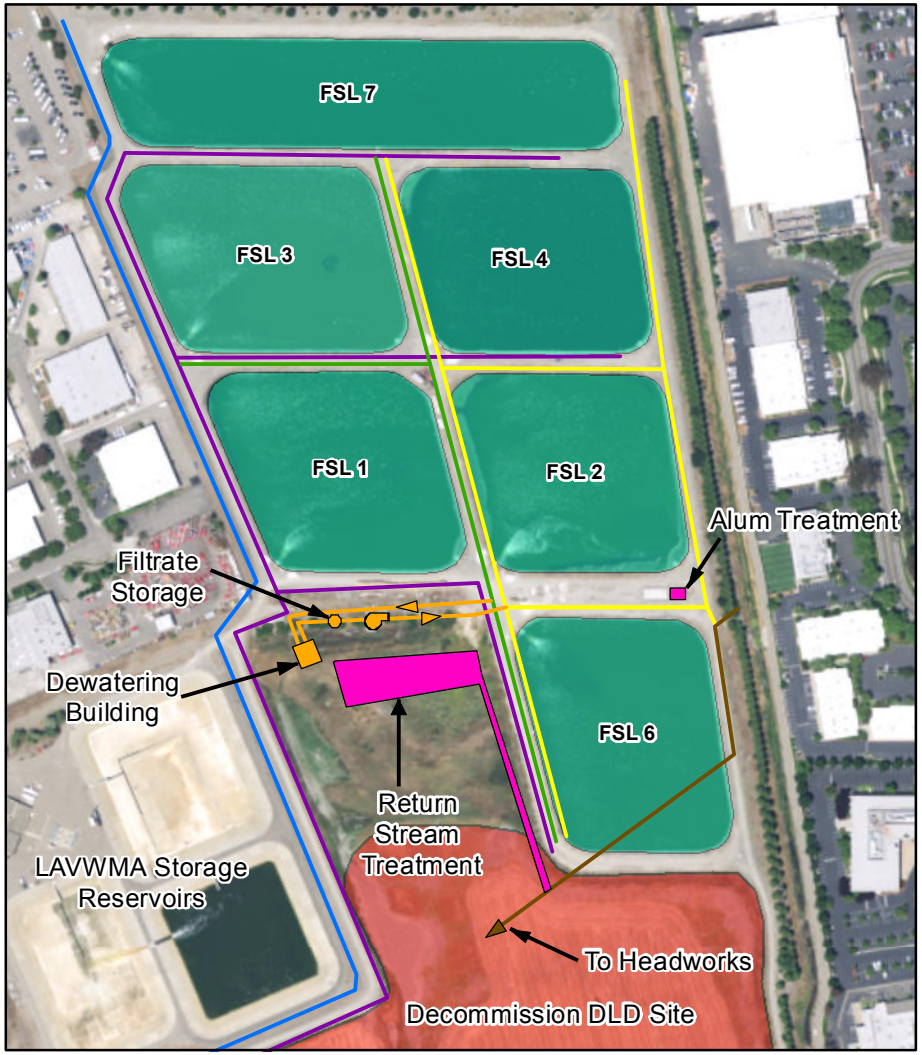
Figure 8-5

Alternatives for Biosolids Disposal

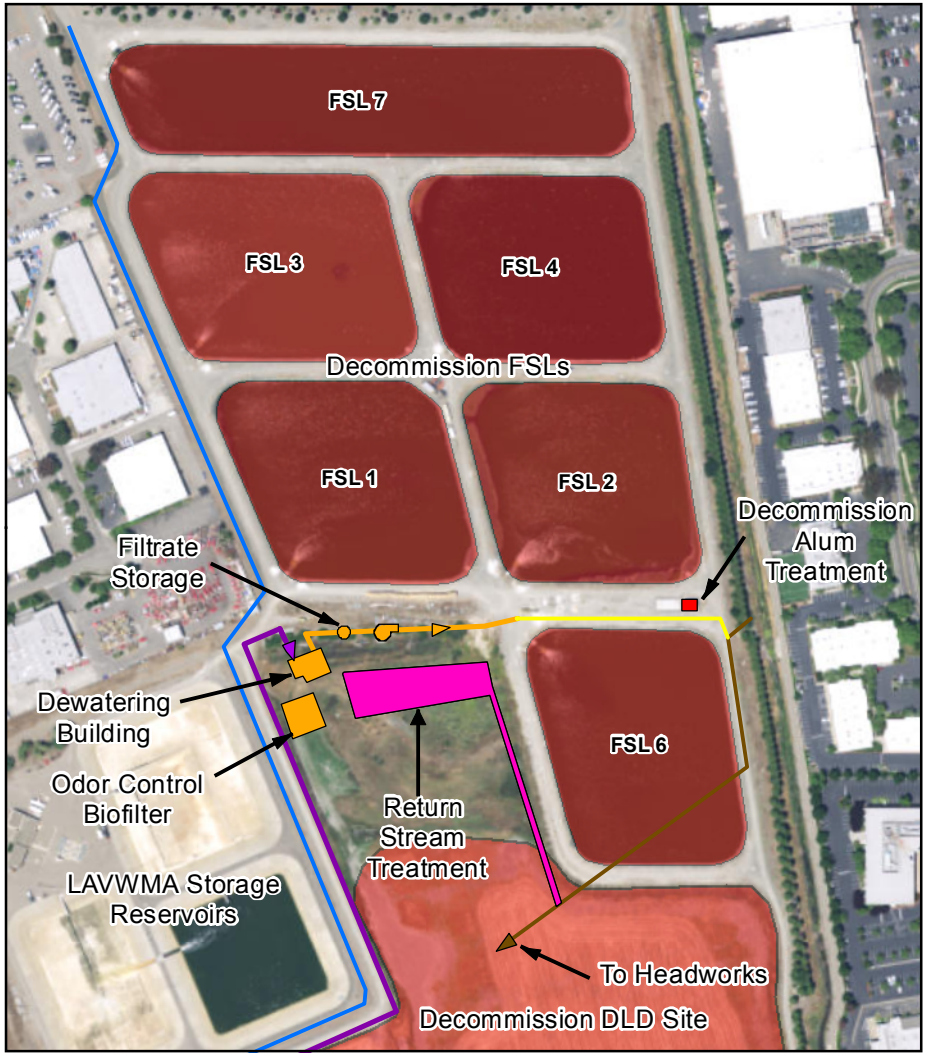
Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Dewater all FSL Solids and
Send Offsite Year-Round



Dewater all Digested Solids and
Send Offsite Year-Round



Symbology

- Potential Thickening/Dewatering Facilities
- Existing Facilities
- Potential Facilities
- Abandoned Facilities
- Harvested Biosolids Pipeline
- Existing FSL Supernatant Pipeline
- Digested Biosolids Feed Pipeline
- Truck Route
- Existing Sanitary Sewer

N

0 200 400

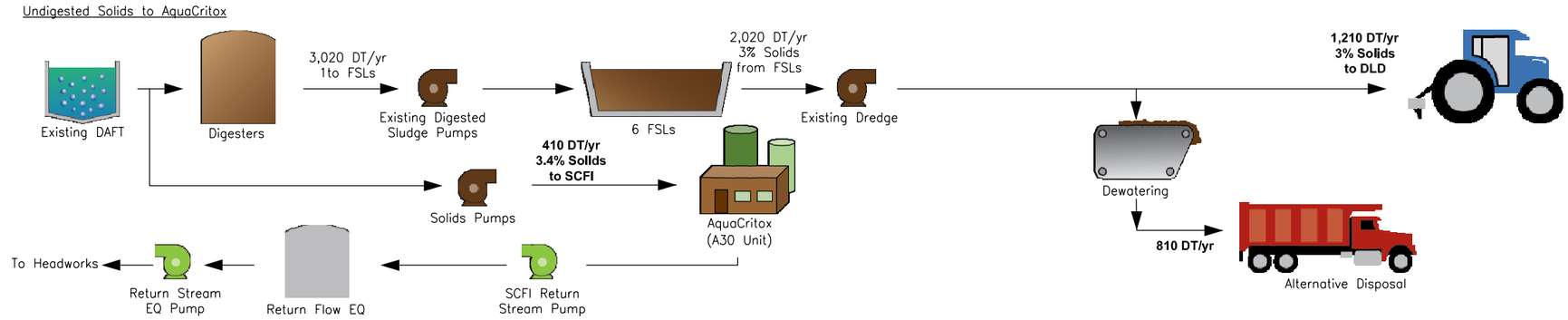
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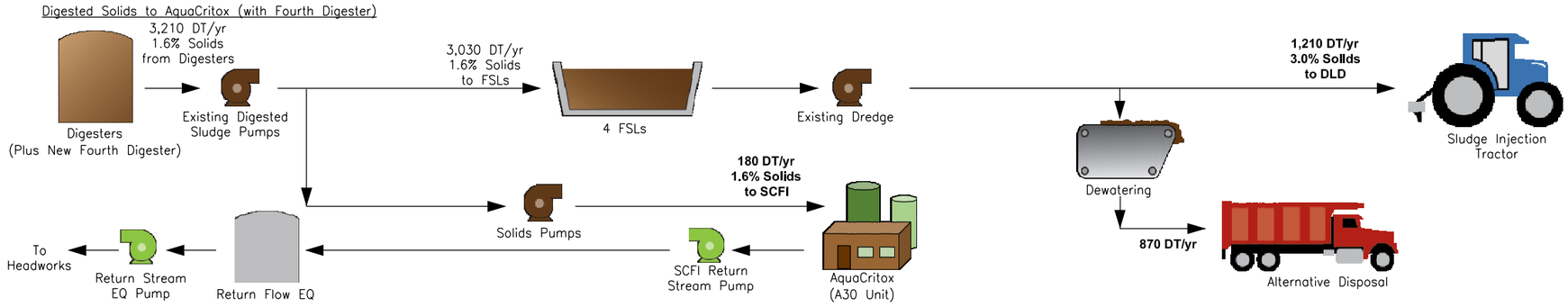
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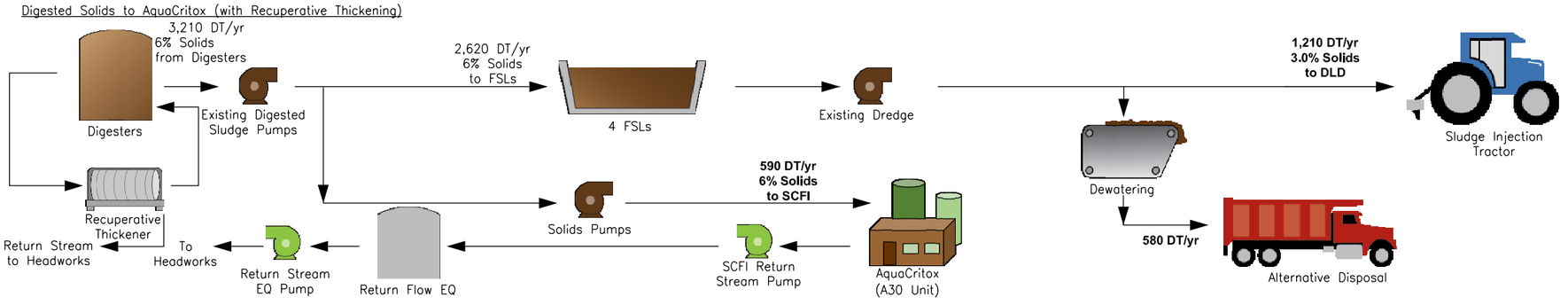
Figure 8-6

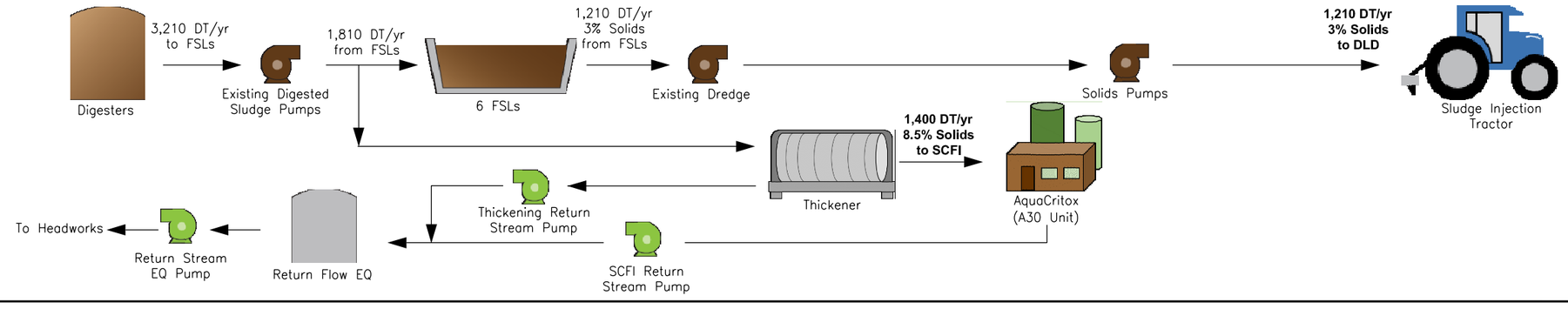
Potential Future Full Dewatering and Offsite Disposal Facilities

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan









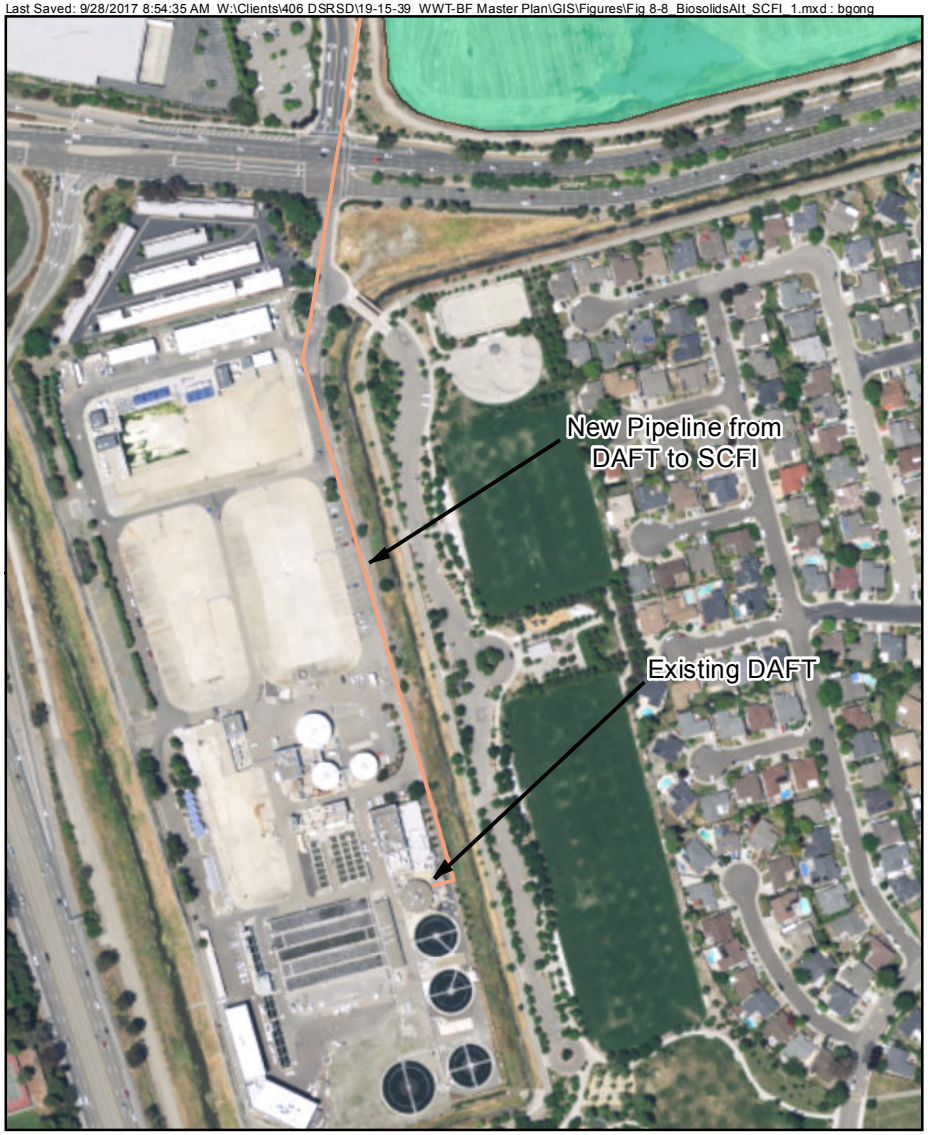
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Figure 8-7

AquaCritox™ Alternatives Schematics

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- Space Reserved for Potential Thickening/Dewatering Facilities
- Potential SCFI Facilities
- Existing Facilities
- Abandoned Facilities
- Harvested Biosolids Pipeline
- Existing FSL Supernatant Pipeline
- Digested Biosolids Feed Pipeline
- Truck Route
- Undigested Biosolids Pipeline
- Existing Sanitary Sewer

N

0 200 400

Scale in Feet

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Figure 8-8

Facilities to Send Undigested Solids to AquaCritox™

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



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Symbology

- Space Reserved for Potential Thickening/ Dewatering Facilities
- Potential SCFI Facilities
- Existing Facilities
- Abandoned Facilities
- Harvested Biosolids Pipeline
- Existing FSL Supernatant Pipeline
- Digested Biosolids Feed Pipeline
- Truck Route
- Existing Sanitary Sewer

N

0 150 300

Scale in Feet

WEST YOST ASSOCIATES

Figure 8-9

Facilities to Send Digested Solids to AquaCritox™

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



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Symbology

- Space Reserved for Potential Thickening/ Dewatering Facilities
- Potential SCFI Facilities
- Existing Facilities
- Abandoned Facilities
- Harvested Biosolids Pipeline
- Existing FSL Supernatant Pipeline
- Digested Biosolids Feed Pipeline
- Existing Sanitary Sewer

N

0 150 300

Scale in Feet

WEST YOST ASSOCIATES

Figure 8-10

Facilities to Send Digested and Thickened Solids to AquaCritox™

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

CHAPTER 9
Energy Management Plan

This Chapter presents an overview of the energy use and management strategies for the WWTP. The major topics discussed are as follows:

- WWTP Energy Demands and Sources
- Potential Energy-Saving Improvements
- Strategies for Achieving Net Zero Energy Demand
- Potential Net Zero Energy Project Costs and Revenues

9.1 WWTP ENERGY DEMANDS AND SOURCES

This section presents an energy balance evaluation that was developed for the WWTP Master Plan to characterize the current and future WWTP energy demands and sources. Topics discussed in this section include:

- Energy Balance Overview
- Energy Balance for Current WWTP Operations
- Energy Balance for 2035 WWTP Operations

9.1.1 Energy Balance Overview

The energy balance developed for the WWTP Master Plan shows how the energy supplied to the WWTP from Pacific Gas and Electric (PG&E) or through onsite engine generation is used to meet the energy demands of the different WWTP process areas. Because the WWTP operations vary significantly between the summer and winter conditions (primarily due to recycled water production), an energy balance was developed for each season separately. The summer season includes the months of May through October. The winter season includes the months of November through April.

Energy balances were created for current conditions by calculating the energy supply and demands using data from several sources, as discussed in more detail in Section 9.1.2. The difference between the total energy supplied to the WWTP and the calculated process demands reflects the energy demands associated with the non-process use energy demand of the plant (e.g., lighting, computer use, heating/cooling of buildings not on the heat loop, etc.).

Energy demands for each of the existing process areas were projected into 2035 by applying a multiplier to the current energy demands. Power demands from additional facilities that may be constructed by 2035 were also calculated. Based on direction from District staff, two scenarios were evaluated for these 2035 conditions: 2035 Planned and 2035 Potential. The 2035 Planned conditions include all facilities that the District will have online by the year 2035, regardless of changing regulations or new reuse opportunities. The 2035 Potential conditions include facilities that the District may construct by the year 2035 resulting from changing regulations or the development of new opportunities. Additional discussion regarding the 2035 facilities under each of these scenarios is provided in Section 9.1.3.2.2.

9.1.2 Energy Balance for Current WWTP Operations

This section presents the analyses completed to develop the energy balances for current WWTP operations. The following specific topics are addressed:

- Current WWTP Energy Sources
- Current WWTP Energy Demands
- Current WWTP Energy Balance Summary

9.1.2.1 Current WWTP Energy Sources

The WWTP receives its energy from three cogeneration engines and a direct connection to the PG&E power grid. Data for the period of September 2013 to September 2015 was used to assess power generation rates for the three cogeneration engines and power withdrawal from PG&E. A summary of this data is presented in Table 9-1. As shown, the average energy supplied was 1,710 kW in the summer and 1,440 kW in the winter. Approximately 65 to 70 percent of the energy is supplied from the onsite cogeneration engines. A portion of the gas used to run the cogeneration engines is biogas generated in the digesters. The remaining gas supply is natural gas received from PG&E.

Table 9-1. Power Supplied from Cogeneration and PG&E September 2013 to September 2015

Facility	Summer, kW	Winter, kW
Cogeneration Engine 1	370	330
Cogeneration Engine 2	390	330
Cogeneration Engine 3	370	360
From PG&E	580	420
Total	**1,710**	**1,440**

9.1.2.2 Current WWTP Energy Demands

Table 9-2 lists the existing WWTP processes/facilities that draw a significant energy demand and were thus evaluated as part of this effort. Also shown is the name of the power distribution panel associated with the equipment used in the process/facility. For this analysis, the current average summer and winter power demands were calculated for each of these WWTP processes/facilities.

Table 9-2. Major WWTP Process Areas and Associated Distribution Panels

Process Area	Distribution Panel
East Amador Lift Station	DPD
Influent Screenings	
Headworks Biofilter	
Aerated Grit Tanks	
Primary Sedimentation Basins	
Anaerobic Digesters	
Gas Treatment System	DPD (not able to be verified)
Influent Pumps	DPD and DPA
Primary Sludge Pumping	
Odor Reduction Tower	DPE, DPD, and DPA
DAFT	DPA
DAFT and Clarifier Biofilter	DPA (not able to be verified)
Aeration Tanks	DPE
Secondary Clarifiers	
Mixed Liquor/WAS Pumps	
RAS Pumps	
3W Pumps	DPG
EPS 1	
FSL Cap Water Pumps	
MF Supply Pumps	
Chillers, Heat Loop, Maintenance Building AC[(a)]	
EPS 2[(a)]	DPM
Tertiary Influent Pumps	
Sand Filtration	
UV System 2 (SF)[(a)]	DPN
Recycled Water Pump Station	
MF Building HVAC System[(b)]	SG-1
MF System	
UV System 1 (MF)[(a)]	
FSL Aeration System	Separate PG&E Meter

(a) Process area energy demand was back calculated from distribution panel power demand data.
(b) MF Building HVAC System power consumption taken as the average power consumption on days when the MF System was not running.

For most of the process/facilities listed in Table 9-2, the equipment-specific information collected from the District's Asset Database and available design drawings was used along with the available data to calculate energy demands using the following procedures:

- For most of the mechanical equipment assets, power demand was assumed to be 70 percent of the motor horsepower. This horsepower was then multiplied by the average daily hours of operation to determine the average daily demand.
- Energy demand of most major pump systems was calculated from an assumed pump efficiency (based on pump type), average daily flow through the pump based on data for the period of September 2013 to September 2015, and the estimated total dynamic head at the typical operating condition.
- Minor pieces of equipment that contribute a negligible power demand because they run infrequently and have a low demand were not included in the calculations.

Power monitors associated with the distribution panels collect usage data for the entire panel via data collection system called Powernet. In some cases, the distribution panel Powernet data could be used to estimate power demand for a given system by subtracting the power demands for the other major processes supplied by the distribution panel that were already estimated using the procedures listed above. For this analysis, Powernet data for distribution panels DPG, DPM, DPN, and SG-1 collected over the period of September 2013 to September 2015[1] was used. This approach was used for the following specific processes:

- Chillers and heat loop system at the Maintenance Building energy use was determined using data from distribution panel DPG;
- Effluent Pumping Station No. 2 energy use was determined using data from distribution panel DPM;
- UV system associated with the sand filter facility (UV system 2) energy use was determined using data from distribution panel DPN;
- MF Building HVAC system energy use was determined using data from distribution panel SG-1, but only from the days when the MF and associated UV system were not in operation; and

[1] The District also provided Powernet data for distribution panels DPD and DPE. However, this data could not be used to assess power demands because the power flowing through the Powernet sensors is recorded as both a positive and negative number, depending on the direction of power flow. This phenomenon occurs because the distribution panels (along with distribution panel DPA) are located on the same circuit as the three cogeneration engines, and this circuit is also connected to the WWTP power grid. Because there are no controls on this circuit to direct the flow of power only in one direction, the direction of power flow can change depending on where the demands and power sources are coming from at any given time. As such, the data being collected by the DPD and DPE Powernet sensors is not an accurate representation of the power used by the equipment connected to these distribution panels.

- UV system associated with the MF facility (UV system 1) energy use was determined using the data from distribution panel SG-1 during period when the system was receiving flow and subtracting out the calculated energy demand from the MF Building HVAC system (and the MF system).

For the aeration blowers and FSL aeration system, other information was used to determine the energy demands. For the blower system, Motor Control Center B5 and B6 power use data for the period of September 2013 to September 2015 was used to directly determine the energy demand. For the FSL aeration system, the PG&E power use data for the meter located at the FSL site for the period of June 2012 to March 2015 was used to directly determine the energy use.

The calculated energy demands for each process/facility are presented graphically for the summer and winter conditions in Figure 9-1 and Figure 9-2, respectively. As shown in these figures, the largest energy demanding process areas are the aeration tanks and recycled water pumps in the summer, and the aeration tanks and DAFT in the winter. The specific energy demand calculations for each process area are described in detail in **Appendix M-1.**

9.1.2.3 Current WWTP Energy Balance Summary

The energy source and energy demand information discussed above was combined to create energy balances for the WWTP that represent the current average conditions. These energy balances are presented in Figure 9-3 and Figure 9-4 for the summer and winter conditions, respectively.

The energy balance analysis shows that the non-process energy consumption represents about 11 percent and 18 percent of the total WWTP energy demand in the summer and winter, respectively. In the summer months, the non-process energy demand is the third highest demand (with the aeration tanks and recycle water pumps having higher demands). In the winter months, the non-process energy is the second highest demand, second only to the aeration tanks.

As noted previously, approximately one third of the plant energy demands are met by electrical power supplied from PG&E. The remaining energy used by the WWTP is supplied from the cogeneration system. It is estimated that between 30 and 35 percent of the gas used in the cogeneration system is biogas generated in the digesters. Thus, approximately 21 to 23 percent of the total WWTP energy demand is met by this onsite renewable energy source (i.e., biogas).

9.1.3 Energy Balance for 2035 Conditions

This section presents the analyses completed to develop the energy balances for the 2035 WWTP operating conditions. The following topics are discussed:

- Anticipated 2035 Energy Supplies
- Anticipated 2035 Energy Demands
- Anticipated 2035 Energy Balance Summary

9.1.3.1 Anticipated 2035 Energy Supplies

The District estimates that each cogeneration system engine generator is available approximately 80 percent of the time, allowing for maintenance and repairs. Therefore, the total cogeneration potential at the WWTP is assumed to average about 1,120 kW, which is about 20 kW less than the average energy demand that occurred between September 2013 and September 2015. If no additional engine generators are installed, electricity supplied from the PG&E grid or some other power source will be needed to meet the additional 2035 demands.

9.1.3.2 Anticipated 2035 Energy Demands

Energy demands in 2035 will include both the existing facilities and new facilities that will (or may) be in operation by 2035. The procedures applied in calculating these anticipated demands are provided below, followed by a summary of the calculated values.

9.1.3.2.1 Anticipated Energy Demands for Existing Facilities

For the existing facilities, anticipated 2035 energy demands are calculated by applying process-specific multipliers to the calculated current energy demands. These multipliers were developed based on the anticipated changes between present conditions and the 2035 conditions. The following specific assumptions were applied in developing the multipliers:

- The equipment that is not expected to have a significant change in energy demand between now and 2035 were assigned a multiplier of 1.0. Equipment assigned this multiplier will not be expanded and will generally operate for the same amount of time each year regardless of the anticipated increase in flows and loads. This multiplier was also applied to the non-process use energy demands.
- All pumping systems that convey all or a set portion of the wastewater flow were assigned a multiplier of 1.29. This multiplier is the 2035 average influent flow divided by the current average influent flow.
- Equipment that will be expanded due to planned improvements was assigned multipliers based on the assumed expansion increment. For example, there are three existing digesters, and one more is planned. Therefore, the digestion facility equipment was assigned a multiplier of 1.47.

- Equipment that will operate more frequently or at a higher horsepower due to increasing flows and/or loads entering the WWTP, but where the increases in use are not proportional to the expected increase in average influent flow, were assigned multipliers based on the anticipated changes in flows or loads that must be handled by the process they support. For example, the multiplier for the filter influent pump station was developed based on the anticipated future recycled water flow divided by the current average influent flow.
- Finally, the MF/UV system is assumed to not be used for recycled water production in 2035. Thus, a multiplier of 0 was applied to these facilities. (Note that a separate energy demand calculation was developed for the 3.0 mgd FAT system under the 2035 Potential scenario.)

The specific multipliers applied to each process/facility, along with the reasoning behind their development, and the resulting anticipated 2035 energy demands of the existing facilities are provided in **Appendix M-2**.

9.1.3.2.2 Planned and Potential Facilities

For the new facilities that may be online by 2035, the District has identified two scenarios of interest. The 2035 Planned scenario includes all facilities that the District will have online by the year 2035, regardless of changing regulations or new reuse opportunities. The 2035 Potential scenario also includes facilities that could reasonably be constructed by the year 2035 due to changing regulations or the development of new opportunities.

The specific facilities included in the 2035 Planned scenario include:

- A fourth digester and associated equipment
- Three additional primary sedimentation basins
- Additional gas treatment to handle the increased digester gas production (from increased solids loading and improved solids capture)
- A ballasted flocculation system installed upstream of the Sand Filter system
- Alum addition to the FSL supernatant
- A partial biosolids dewatering system (the "Dewatering of FSL Solids for Offsite Disposal in the Winter" alternative was assumed)

The additional facilities that are included under the 2035 Potential scenario are as follows:

- BACWA Level 2 improvements:
  - Three additional aeration tanks
  - One additional secondary clarifier
- 3 mgd FAT system:
  - MF/UF system
  - RO system
  - UV/AOP system
- A full biosolids dewatering system (the "Full Dewatering from FSLs" alternative was assumed)

A figure showing the facilities listed above for the 2035 Planned and 2035 Potential scenarios is provided as Figure 9-5.

9.1.3.2.3 Anticipated Energy Demands for Planned and Potential Facilities

Energy demands for the planned and potential facilities were calculated using the following procedures:

- The alum addition facilities at the FSLs were based on the operating costs presented in Chapter 6. Energy demand was calculated by converting the annual power costs of chemical feed equipment to an average power demand. For the energy balance, the alum addition facilities were added into the FSL process group energy demands.
- The ballasted flocculation system energy demand was calculated based on the design equipment list and using the same approach as described in Section 9.1.2.2. For the energy balance, the ballasted flocculation energy demand was added into the Sand Filter process group energy demands.
- BACWA Level 2 Improvements energy demand was calculated based on the operating costs presented in Chapter 6, where the energy demand was calculated by converting the annual power costs to an average energy demand. For the energy balance, the calculated BACWA Level 2 improvements energy demand replaces the energy demand that was calculated for the existing aeration tanks and secondary clarifiers process areas.
- 3.0 mgd FAT Systems energy demand was calculated from the operating costs developed in Chapter 7, where the annual power costs were converted into an average energy demand. For the energy balance, these costs replace the energy consumption currently associated with the MF/UV system.
- Partial and full biosolids dewatering system energy demands were calculated directly from the electrical operating costs developed in Chapter 8, where the annual power costs were converted into an average energy demand.

Details of energy calculations can be found in **Appendix M-3** and **Appendix M-4** for 2035 Planned and 2035 Potential scenarios, respectively.

9.1.3.2.4 Energy Demand Summary

The calculated energy demands for each major process area anticipated under the 2035 Planned Scenario are presented graphically on Figure 9-6 and Figure 9-7 for the summer and winter conditions, respectively. Compared to current conditions, the aeration tank process area is expected to use less energy in 2035 Planned scenario due to the increased solids removal in the primaries. Thus, in the summer months, the recycled water pumps become the largest energy user. Moreover, the recycled water system will have the largest increases in power demand. In the winter months, the aeration tanks power demand is reduced from current demands, however, it remains the process area with the largest overall power demand.

The calculated energy demands for each major process area under the 2035 Potential scenario are presented graphically on Figure 9-8 and Figure 9-9 for the summer and winter conditions, respectively. For the 2035 Potential scenario, the aeration system power demand increases by a factor of two, due to the addition of BACWA Level 2 improvements, making the aeration tanks process area the largest power demand in winter and the second largest energy demand in summer. The largest power demand in the summer is still expected to be the recycled water pumps. The second largest energy demand in the winter is the 3.0 mgd FAT system process area.

9.1.3.3 Anticipated 2035 Energy Balance Summary

The summer and winter season energy balances developed for 2035 Planned scenario are presented in Figure 9-10 and Figure 9-11, respectively. As shown, the overall demand from PG&E is expected to increase to 1,480 kW in the summer and to 650 kW in the winter. This represents an increase over current demands of approximately 900 kW in the summer and 200 kW in the winter. Given the cogeneration power production will remain the same at 1,120 kW, approximately 43 percent of the WWTP energy demand will be supplied by the cogeneration system in the summer, and approximately 63 percent of the WWTP energy demand will be supplied by the cogeneration system in the winter. In both seasons, biogas will make up approximately a third of the total gas supplied to the cogeneration system. Thus, onsite renewable energy is expected to make up approximately 14 percent of the plant energy needs in the summer and approximately 21 percent of the plant energy demands in the winter.

The summer and winter season energy balances developed for 2035 Potential scenario are presented in Figure 9-12 and Figure 9-13, respectively. As shown, the overall demand from PG&E is expected to increase to 1,930 kW in the summer and to 1,590 kW in the winter. This represents an increase over current demands of approximately 1,400 kW in the summer and 1,100 kW in the winter. Approximately 36 percent of the WWTP energy demand is expected to be supplied by the cogeneration system in the summer, and approximately 41 percent of the WWTP energy demand is expected to be supplied by the cogeneration system in the winter. Thus, it is estimated that between 12 and 14 percent of the plant energy demands would be from renewable sources.

9.2 POTENTIAL ENERGY-SAVING IMPROVEMENTS

This section presents an evaluation of potential improvements that could be made to reduce unnecessary power consumption at the WWTP. The information presented below is based, in part, on an independent evaluation that was completed by HDR for this WWTP Master Planning effort. A TM summarizing the findings from this independent energy analysis is provided in **Appendix N**. Key findings from these efforts are summarized below under the following topics:

- Barscreen Building Exhaust Fans
- Gas Treatment System
- DAFT System
- Operational Adjustments

9.2.1 Barscreen Building Exhaust Fans

The current exhaust fans for the barscreen building, which include the odor control system for the washer/compactor, are oversized. These exhaust fans, which are 75 HP, are estimated to provide approximately two times the amount of air flow required. Thus, a savings of roughly half of the current energy consumption (estimated at 39 kW) could be made by rebalancing the odor collection system and changing the fan to motor belt drive sheaves.

9.2.2 Gas Treatment System

Energy consumption of the gas treatment system is higher than required, and the system has several inefficiencies, as follows:

- Flow control is done by recycling gas from the high-pressure discharge back to the compressor suction. A variable frequency drive on the compressor motor would be a better way to accomplish this goal.
- System heat exchanger piping could be re-arranged to improve heat recovery and reduce refrigerant cooling loop energy.
- Piping size in between systems is too small.
- The heat exchangers, which are plate and frame type that allow for additional plates to be added, could be expanded to provide lower pressure drop and more heat transfer surface area. This would improve the energy efficiency of the system overall.
- The system condensate drains were provided with pumps. Gravity can be used to replace the pumped condensate drains.

As further detailed in Section 9.3.4, the existing gas treatment system is also undersized for current biogas production. Therefore, given the age of the existing equipment, the fact that it is undersized, and the efficiency issues listed above, a new gas treatment system is recommended.

9.2.3 DAFT System

The DAFT system is one of the major power consuming systems at the WWTP, and there are other technologies available that could achieve the same (or possibly better) thickening by using less energy. Given that the DAFT system was recently rehabilitated, it is likely that the existing system will reliably serve the WWTP solids thickening needs for the next 15- to 20-year period. It is therefore unlikely that the energy savings associated with changing the thickening process would justify replacement, because the existing system does not currently require rehabilitation. However, once the equipment associated with the DAFT facility has reached the end of its useful life, the District should evaluate whether replacing the DAFT with a less energy intensive thickening process is warranted. Such an evaluation would consider the difference between cost of rehabilitation versus cost of replacement and how that compares to potential energy savings associated with replacement, and potential increases in chemical use that may be required by an alternative system. A lifecycle cost analysis developed with this information will inform the District whether a system replacement makes sense.

9.2.4 Operational Adjustments

Potential operational strategies that have been identified to reduce WWTP energy demands are as follows:

- Optimizing aeration blower header pressure. Even a small reduction in pressure could reduce costs significantly.
- Cleaning the aeration diffusers regularly now that drought-related pressures are gone and the District has more flexibility to take basins offline.
- Reducing pressure of air compressor for utility and air scour air for the moving bed sand filters.
- Optimizing 3W system to reduce the amount of total flow pumped. The most potential savings would be realized if the amount of flow used to remove waste heat from the cogeneration system could be reduced.
- Optimizing mixing energy for anaerobic digesters by cycling the mixers to the extent feasible.
- Operating the aeration basins with an anaerobic selector year-round to improve air transfer efficiency in the basins. (This strategy is evaluated n Chapter 6 as an improvement to increase treatment capacity.)

9.3 STRATEGIES FOR ACHIEVING NET ZERO ENERGY DEMAND

The District is interested in increasing onsite renewable energy production. Achieving net zero energy for the WWTP would mean that the total amount of energy used at the WWTP on an annual average basis is roughly equal to the amount of renewable energy created onsite. This section presents a strategy developed for this Master Plan that would result in a net zero energy condition under the 2035 Planned scenario discussed previously in this Chapter. This strategy is described below under the following topics:

- Overall Strategy for Achieving Net Zero Energy
- Increased Digester Gas Production Potential
- Cogeneration Expansion
- Digester Gas Management Requirements
- Additional Considerations for Digester Gas Power Generation
- Potential Solar Power Generation Facilities
- Summary of Facilities Required for Achieving Net Zero Energy

9.3.1 Overall Strategy for Achieving Net Zero Energy

As noted previously in this Chapter, the WWTP currently relies on both natural gas and electricity received from PG&E to meet the WWTP energy demands, and only 21 to 23 percent of the current plant energy is obtained from onsite renewable sources (i.e., generation of heat and power using biogas from the WWTP digesters). This percentage of renewable energy generated onsite is expected to decrease to approximately 18 percent (on an annual average basis) under the 2035 Planned scenario. Figure 9-14 shows the current energy sources and demands for the summer and winter conditions and compares this to the anticipated energy sources and demands under the 2035 Planned scenario. As shown, a significant amount of additional renewable energy will need to be generated onsite to reach a net zero energy condition.

The District has already invested in a cogeneration system, including hiring staff to maintain and operate the cogeneration system, and this system has capacity to receive additional biogas generated onsite. There is also limited space available for implementing other onsite renewable energy generation options (like solar). Thus, the best overall strategy for reaching a net zero energy goal at the WWTP will likely involve increasing renewable energy generation using digester gas to the extent feasible.

It will not be possible to increase digester gas production to the levels needed to reach net zero energy without bringing in organic materials to the WWTP and digesting them along with the WWTP solids to create additional digester gas (often referred to as co-digestion). Because the availability of hauled organic materials is uncertain, major investments in infrastructure should proceed only after digester feedstocks are identified to avoid building facilities that cannot be used due to lack of readily available materials.

A stepwise approach toward increasing onsite renewable energy generation by taking advantage of available organics as they are identified has been developed for this Master Plan. This stepwise approach allows the District to take advantage of "low hanging fruit" opportunities for increasing organic loadings to the digesters, while potential hauled waste sources are being identified. This approach would proceed as follows:

- First, increase the amount of digester gas being produced at the WWTP by installing facilities to accept hauled organic wastes
- Then, increase the onsite cogeneration system capacity to take advantage of all the biogas that could potentially be produced in the available digesters.
- Once the digestion capacity is maxed out, increase the digestion capacity.
- Finally, expand cogeneration capacity.

Ultimately, the limiting factor for biogas-generated onsite renewable energy production is the amount of organic materials that would reasonably be hauled to the WWTP for digestion and biogas production. With a very aggressive organic waste receiving program, the District could potentially come close to meeting a net zero energy goal through a co-digestion and onsite biogas energy generation strategy. However, the viability of reaching the second and third steps above will depend on the success of the District's organic feedstock program.

Other onsite renewable energy facilities, such as wind or solar power facilities, could also potentially be used to reach net zero energy. However, given the likely limitations of the site for wind power, solar energy is a more likely option. For this analysis, an assessment of the solar power needs to generate the additional energy that could not be met by a biogas cogeneration system has been developed.

9.3.2 Increased Digester Gas Production Potential

This section addresses the following topics with respect to increased digester gas production:

- Strategies for Increasing Digester Gas Production
- Stepwise Approach for Increasing Digester Gas Production

9.3.2.1 Strategies for Increasing Digester Gas Production

The following strategies can be used to increase onsite renewable energy generation from digester gas:

- Increase WWTP Solids Loadings to Digesters
- Hauled Organic Feedstocks for Co-Digestion
- Increase Digestion Capacity

9.3.2.1.1 Increased WWTP Solids Loading to Digesters

The solids entering the WWTP contain a higher volatile solids (VS) content (and will thus produce more biogas when digested) than the solids from the secondary process. Therefore, increasing the primary solids capture rate will increase biogas production potential at the WWTP.

As discussed previously in this Master Plan, the existing primary sedimentation basins remove approximately 49 percent of the solids entering the WWTP. With the improvements recommended in this Master Plan, the solids capture rate is expected to increase to 65 percent. As noted elsewhere in this Master Plan, these solid capture improvements are recommended to decrease loadings to the downstream processes, thus reducing energy demand associated with the aeration basins and reducing solids loadings to the secondary clarifiers (which have limited capacity).

Enhanced primary treatment options could also be used to increase solids capture in the primary clarifiers beyond 65 percent. These options include: chemically enhanced primary treatment, baffling of the primary clarifiers, and replacement of the primary clarifiers with primary screenings. These enhancement options could increase the primary solids capture rate up to 80 percent. (Some of these options could also be used to help achieve 65 percent removal within the existing basins.) The specific removal rate that could be achieved above 65 percent would need to be evaluated through a pilot testing program.

The benefits of investing in enhanced primary removal to achieve rates beyond 65 percent would include the reduced power demand in the aeration tanks and additional biogas produced by the increased solids captured. It is estimated that aeration tank annual average power demand would be reduced by 140 kW, and that the additional biogas could generate an additional 20 kW of power. These two factors are estimated to result in a savings on energy costs of approximately $160,000 per year, based on a cost of $0.15/kWhr for power from PG&E and $0.31/therm of natural gas.

To increase solids capture from 65 percent to 80 percent through chemical addition in the primaries, the District would need to add alum and polymer to the influent wastewater at an alum dose of 75-100 mg/L and a polymer dose of 2 mg/L. The estimated cost of purchasing the chemicals would be approximately $550,000 per year. In addition, there would be capital and other operational costs associated with the chemical storage and dosing facilities. Because the chemical addition costs are higher than savings, increasing the solids capture from 65 percent to 80 percent through chemical addition is not a cost-effective way to increase digester gas production.

Other strategies for achieving 80 percent solids removal would require increasing the number of clarifiers beyond what is needed to provide 65 percent removal so that the flow through the clarifiers is substantially lower. An 80 percent removal performance would also require that the new clarifiers be optimized for solids removal through baffling. These improvements would require substantially higher capital investment than chemical addition, but would achieve the desired removal at a lower operating cost. On the other hand, this strategy would result in more permanent modifications that may not be needed if reduced primary solids removal performance was desired to provide carbon to the downstream processes for nitrogen removal in the future.

Ultimately, the potential benefits of an enhanced performance strategy would need to be evaluated using pilot testing data along with cost information for the technology being considered. If such an evaluation is desired, it is recommended to proceed after the District has completed the primary treatment improvements discussed in this Master Plan.

As discussed in the following subsection, importing hauled organics can also provide increased biogas generation and at a lower cost. Increasing primary solids removal from 65 to 80 percent is estimated to provide 5.5 cfm of biogas. This gas production is approximately equivalent to the amount of gas produced from 1.5 tons per day of hauled food waste, or a fraction of a truckload.

Given the relative cost and efficiency of gas production, it has been assumed for purposes of the analysis presented herein that the District will not increase the primary solids capture rate above 65 percent. Ultimately, the amount of WWTP solids that can be readily (and cost effectively) diverted to the digesters should be further evaluated after the primary treatment expansion project recommended in this Master Plan is complete and an understanding of the availability of other potential feedstocks is better defined.

9.3.2.1.2 *Hauled Organic Feedstocks for Co-Digestion*

External feedstocks can be added to the digesters to increase the volatile solids loading and thus the amount of biogas produced. The three primary sources of organic materials that could be accepted at the WWTP are FOG, food processing facility waste streams, and organic materials that have been separated from trash destined for a landfill.

Due to its higher volatile content, FOG is the preferred feedstock. However, FOG availability is limited. The District's service area includes approximately 80,000 people, which is expected to generate 3,400 gpd of trapped FOG. However, because of the WWTP's location is near a major freeway, it is estimated that the District may be able to receive up to 12,000 gpd total of FOG from both its service area as well as from external sources. For the purposes of this evaluation, the amount of FOG added to the digesters was assumed to be 12,000 gpd, or four truck loads per day.

Food processing facility waste streams are the second most desirable feedstocks. These materials are generally free of plastics and other contaminants that could cause problems in the digesters. In addition, depending on the food process from which the waste is generated, these sources can produce biogas through anaerobic digestion at rates comparable to those achieved through FOG digestion. To secure the availably of a reliable supply of food processing waste material, the District would need to partner with food processing industries.

With state regulations in place that put restrictions on organic waste disposal in landfills, there is an increasing opportunity for organic waste streams that have been diverted from landfills. These wastes, which are predominantly food-related materials, are the least desirable for onsite biogas generation. First, if not adequately pre-processed, this material has the potential to contain plastics and other materials that may cause issues in the digesters. In addition, these materials have a lower biogas generation potential than FOG and many industrial food processing waste streams. These materials will likely result in more residual solids that require handling and disposal after digestion than other possible sources. Conversely, FOG and high-strength industrial waste streams would likely decrease the amount of residual solids that require handling and disposal after digestion. The primary advantage of the organic materials that are diverted from landfills is their potential availability.

Given the restrictions of the District's site, it is estimated that the District could handle a maximum of six trucks per day of food waste (either from an industrial source or diverted from a landfill) in addition to the four truckloads of FOG received, for a total of ten truckloads per day. One truckload of food waste is equivalent to approximately 21 tons of food waste. Thus, six truckloads a day is equivalent to approximately 125 tons/day of food waste.

For purposes of this analysis, it has been assumed that the 125 tons/day would come from diverted food waste. This is because diverted food waste materials are known to be available and can be assessed with respect to their volatile content and solids generation potential. However, it is recommended that the District pursue other potential food processing waste streams as they can provide more gas generation per ton of material received and generate fewer residual solids that require management and disposal.

9.3.2.1.3 Increased Digestion Capacity

The District is currently moving forward with the construction of a fourth digester at the WWTP that is the same size as Digester No. 3 (the District's largest digester). As discussed in Chapter 8 of this Master Plan, the digester expansion project will provide more than enough digestion capacity for the anticipated 2035 solids loads when the largest unit is out of service. Therefore, during periods when all of digesters are online (which is most of the time) there will be significant capacity available for digestion of hauled organic waste materials. However, even with all four units online, there will not be adequate digestion capacity to handle the potential 125 tons/day of food waste and 12,000 gpd of FOG, in addition to the anticipated WWTP solids.

There are two options available to increase digestion capacity once the capacity of the four digesters is reached: construct a fifth digester or install a recuperative thickening process on one or more digesters. As discussed in Chapter 8, recuperative thickening to achieve high solids digestion involves constructing a relatively small solids thickening process adjacent to the digesters (as well as improving the digester mixing system and adding a new digester heating system) to achieve increased volatile solids loadings (from approximately 0.15 lb/ft^3 to 0.45 lb/ft^3).

A recuperative thickening approach has the potential to be a much smaller footprint than a new digester. However, prior to investing in either approach, the District would also need to consider whether the recuperative thickening system will require a return stream treatment process. As discussed previously in this Master Plan, the District will likely need to provide treatment of high-strength return streams under the BACWA Level 3 nutrient removal scenario. Recuperative thickening results in the generation of a high-strength return stream. Therefore, a recuperative thickening approach to increasing digester capacity could require that a return stream treatment system be constructed at the main treatment plant site. However, under a BACWA Level 3 scenario, a treatment system would also be needed for return stream coming from the facilities at FSL/DLD site.

To provide treatment both return stream the District could either a) construct two return flow treatment systems or b) construct one return stream treatment system located at the main WWTP site with a new, dedicated pipeline to return high-strength flows from the DLD site to the treatment system located at the main WWTP site. For either approach, a recuperative thickening process return stream treatment system would likely need to be located within a portion of Holding Basin 3. Ultimately, a decision regarding expansion to a fifth digester or construction of recuperative thickening would need to consider all the potential site constraints, including the need for return flow treatment. Additional discussion of issues that should be considered when evaluating return flow treatment are provided in Chapter 10 (Recommended Plan).

For purposes of the analysis presented herein, it has been assumed that recuperative thickening would be determined to be the better strategy for expanding the digestion capacity. To accommodate the potential 125 tons/day of food waste, a recuperative thickening process would need to be installed on one of the large digesters. To minimize impacts on available storage capacity and plant ingress/egress, this facility could be in the parking lot adjacent to the new Digester 4. As noted above, a treatment system, if needed would be located in the southeastern portion of Holding Basin 3.

9.3.2.2 Stepwise Approach for Increasing Digester Gas Production

This section presents the stepwise approach that was developed for increased onsite energy generation through increased biogas production. This information is presented in the following sections:

- Approach Overview
- Potential Increased Gas Production
- Potential Power Generated and Cost Savings

9.3.2.2.1 Approach Overview

The recommended approach for increasing digester gas production at the WWTP involves implementing the following steps:

1. Increase solids capture to 65 percent by providing primary clarifier expansion/improvements.
2. Initiate a FOG receiving program to bring in FOG waste up to its maximum potential. Up to 12,000 gpd is assumed to be available. This is equivalent to four truckloads of FOG received per day, or 28 truckloads per week.
3. Initiate an organic waste receiving program to bring in additional materials to maximize available cogeneration capacity. Assuming food waste materials diverted from landfills are used to increase biogas production, approximately six to nine truckloads of material would need to be delivered per week (in addition to 28 truckloads of FOG) to maximize available cogeneration capacity.
4. Expand cogeneration capacity and bring in additional food waste materials to maximize available digester capacity. Approximately 18 to 21 truckloads of food waste material would need to be delivered per week be (in addition to 28 truckloads of FOG) to maximize digestion capacity.
5. Expand digester capacity to maximize FOG and food waste receiving potential. Up to 12,000 gpd of FOG and 125 tons of food waste is assumed to be available. This would be equivalent to approximately 42 truckloads of food waste material and 28 truckloads of FOG delivered per week.

These steps are further detailed in Table 9-3.

Table 9-3. Steps to Increase Biogas Production

Biogas Production Steps		WWTP Solids Capture, percent	FOG Received, gpd	Food Waste Received, tons/day	Total Truckloads Received		Additional Facilities Needed	Limiting Factor
No	Goals				Daily	Weekly		
1	Improved WWTP Solids Capture, No Hauled Waste	65	None	None	0		None	None
2	Maximize FOG Potential		12,000		4	28	• FOG Receiving Facility	FOG Availability
3	Maximize Existing Cogeneration Capacity			18-27[(a)]	~5	34-37	• FOG Receiving Facility • Food Waste Receiving Facility	• Fog Availability • Cogeneration Capacity
4	Maximize Existing Digester Capacity			53 - 62[(a)]	~7	46-49	• FOG Receiving Facility • Food Waste Receiving Facility • Additional Cogeneration Capacity	• Fog Availability • Digester Capacity
5	Maximize Hauled Waste Receiving Potential			125[(b)]	10	70	• FOG Receiving Facility • Food Waste Receiving Facility • Additional Cogeneration Capacity • Recuperative Thickening Process on One Large Digester	• Fog Availability • Food Waste Availability

(a) Loadings will vary depending on solids being produced by the WWTP.
(b) Maximum potential for hauled food waste.

9.3.2.2.2 Potential Increased Gas Production

Annual average and maximum month digester gas production rates were calculated for the steps listed in Table 9-4. For this analysis, the following assumptions were applied:

- WWTP primary solids will have a volatile to total solids ratio of 85 percent
- WWTP secondary solids will have a volatile to total solids ratio of 73 percent.
- Wastewater volatile solids will be reduced by 67 percent in the digesters.
- The FOG received will have a grease content of 12 percent, with a volatile solids content of 95 percent.
- FOG volatile solids will be reduced 90 percent in the digesters.
- Food waste received will have a solids content of 28 percent, with a volatile to total solids ratio of 86 percent.
- Food waste volatile solids will be reduced by 74 percent in the digesters.
- Digester gas will be produced at a rate of 15 cubic feet per pound of volatile solids destroyed.

The amount of additional digester gas produced in each step is described in detail in Table 9-4.

Table 9-4. Digester Gas Production Rates

Biogas Generation Steps		Hauled Organics Truckloads Per Day	Gas Production, cu-ft/day	
No.	Goal		Annual Average	Monthly Maximum
1	65 Percent Primary Solids Capture, No Hauled Waste	0	260,000	310,000
2	Maximize FOG Potential	4	410,000	460,000
3	Maximize Existing Cogeneration Capacity	~5	560,000	560,000
4	Maximize Existing Digester Capacity	~7	750,000	750,000
5	Maximize Hauled Waste Receiving Potential	10	1,080,000	1,130,000

9.3.2.2.3 Potential Power Generated and Cost Savings

The energy generation potential and associated savings in power costs were calculated for each of the gas generation rates shown in Table 9-5 based the following assumptions:

- The amount of biogas and power that would be generated in 2035 with 65 percent solids capture rate in the primary sedimentation basins is the baseline condition, and additional savings are considered beyond this production rate;
- The energy content of the biogas is 550 British thermal units per cubic foot (BTU/cu-ft);
- The engine generator has an efficiency of 30 percent; and
- The cost of natural gas is $0.31/therm, and the cost of electricity is $0.15/KWhr.

The potential power generated and associated savings from the additional digester gas produced at each step is presented in Table 9-5.

Table 9-5. Potential Energy Production and Savings from Increased Biogas Generation

Biogas Generation Steps		Hauled Organics Truckloads Per Day	Annual Average Power Production Potential, kW	Additional Onsite Power Production, 1,000 MWhr/year	Potential Annual Cost Savings, million $/year[(a)]	Potential 30-Year Cost Savings, million $[(b)]
No.	Goal					
1	65 Percent Primary Solids Capture, No Hauled Waste	0	520	0	0	0
2	Maximize FOG Potential	4	830	2.7	0.3	6.6
3	Maximize Existing Cogeneration Capacity	~5	1,120	5.3	0.6	12.9
4	Maximize Existing Digester Capacity	~7	1,500	8.6	1.0	21.2
5	Maximize Hauled Waste Receiving Potential	10	2,180	14.5	1.7	35.8

(a) Assuming electrical cost of $0.15/kWhr and a natural gas cost of $0.31/therm.
(b) Potential cost savings are based on a 30-year net present worth.

9.3.3 Cogeneration Expansion

This section addresses the cogeneration system expansion needed to meet the net zero energy goals laid out in the steps described above. The topics addressed include:

- Cogeneration Engine Sizing
- Potential Onsite Renewable Energy Generated
- Electrical System Improvements for Cogeneration System Expansion

9.3.3.1 Cogeneration Engine Sizing

The current cogeneration capacity is 1,400 kW. This includes two 500 kW engines and one 400 kW engine. The current cogeneration engines are typically available 80 percent of the time, on average, due to downtime and maintenance. This means the current cogeneration system is expected to have an average capacity of 1,120 kW. This cogeneration capacity sets an upper limit for biogas that can be used to produce energy. If the District wants to produce more than 1,120 kW of energy on average, an additional cogeneration engine will need to be installed.

As shown previously in Table 9-5, another cogeneration engine would be needed at Step 4, once the hauled waste volumes exceed approximately four truckloads per day of FOG and one truckload per day of food waste. This is equivalent to approximately 34 to 37 truckloads of hauled waste received per week.

A new cogeneration engine could be sized to use the available biogas either on an annual average or maximum monthly basis. Table 9-6 provides an estimate of the amount of energy that could potentially be produced at each implementation step under the annual average and maximum month conditions. Note that there is no difference between the annual average and maximum month conditions under Steps 3 and 4, because these steps are limited by the capacity of either the cogeneration engine (Step 3) or digesters (Step 4). Thus, in maximum month solids generation conditions for the WWTP, the District would either flare excess gas generated (Step 3) or accept less hauled materials (either Step 3 or 4).

As shown in Table 9-6, the additional biogas available that results from progressing from Step 3 to Step 4 could produce 380 kW of power. Assuming the new engine operates at 90 percent of its capacity due to downtime, this equates to a required engine capacity of approximately 420 kW. Moving from Step 4 to Step 5, the available biogas could produce an additional 680 kW to 770 kW of power. Again, assuming the new engine operates at 90 percent of its capacity due to downtime, this equates to a required engine capacity of approximately 760 kW and 860 kW. Combined, the total potential engine capacity increases could range from 1,180 kW to 1,280 kW.

Table 9-6. Energy Production Potential

Biogas Generation Steps		Hauled Organics Truckloads Per Day	Power Production Potential, kW	
No.	Goal		Annual Average	Monthly Maximum
1	65 Percent Primary Solids Capture, No Hauled Waste	0	520	610
2	Maximize FOG Potential	4	830	920
3	Maximize Existing Cogeneration Capacity	~5	1,120	1,120
4	Maximize Existing Digester Capacity	~7	1,500	1,500
5	Maximize Hauled Waste Receiving Potential	10	2,180	2,270

The cogeneration system could be expanded into the area previously occupied by the fuel cells (between Clarifiers 2 and 3). This site could accommodate one large engine to meet the capacity needs at Step 5 or two smaller engines - one installed when progressing from Step 3 to Step 4 and another when progressing to Step 5. For this analysis, it has been assumed that two engines would be installed, as follows:

- A 500 kW engine would be installed at Step 4, providing a total average cogeneration capacity of 1,570 kW. This generation potential slightly exceeds the available biogas generated in Step 4. When biogas is not available to maximize capacity, the District could either operate the engines below their rated capacity or use supplemental natural gas to maximize the generating potential.
- An additional 750 kW engine would be installed at Step 5, increasing the total cogeneration capacity to 2,250 kW. This generation rate is slightly lower than the maximum month biogas production rate assumed at Step 5.

The approximate sizing of each cogeneration engine system would be 10 ft by 45 ft. Except for the service entrance gear, all the required equipment – including the engine generator, the heat recovery system, and the emission controls – would be mounted either inside or on top of the 10 ft by 45 ft enclosure container. These two enclosures should be able to fit within the area previously occupied by the fuel cell system.

9.3.3.2 Potential Onsite Renewable Energy Generated

Table 9-7 shows the renewable energy generation potential and total energy generation potential under each of the five steps, assuming a 500 kW engine is installed at Step 4 and an additional 750 kW engine is installed at Step 5. The table also shows the renewable and total energy deficits under annual average and summer month conditions. The renewable onsite energy deficit represents the difference between the total power generated from biogas and the total WWTP power demand. The total onsite energy deficit represents the difference between the total power generated onsite and the total WWTP power demand assuming natural gas is used when biogas is not available. As shown in Table 9-7, the District could get very close to the net zero energy goal on an annual average basis through a cogeneration program. However, non-renewable energy would still be needed during the peak summer months.

Table 9-7. Onsite Power Generation Deficits at Each Potential Energy Generation Step

Steps	Energy Generation Potential		Annual Average			Summer Months		
	Power from Digester Gas, kW	Total Power Generated Onsite, kW	WWTP Power, Demand[(a)], kW	Renewable Onsite Energy Deficit[(c)], kW	Total Onsite Energy Deficit[(c)], kW	WWTP Power, Demand[(a)], kW	Renewable Onsite Energy Deficit, kW	Total Onsite Energy Deficit, kW
1	520	1,120	2,190	1,670	1,070	2,600	2,080	1,480
2	830	1,120	2,200	1,370	1,080	2,610	1,780	1,490
3	1,120	1,120	2,220	1,100	1,100	2,630	1,510	1,510
4	1,500	1,570[(b)]	2,230	730	660	2,640	1,140	1,070
5	2,180	2,245[(d)]	2,270	90	25	2,680	500	435

(a) Power demand increases with increased digester gas treatment.
(b) Additional 500 kW engine installed, with 90 percent availability.
(c) The WWTP would be producing more energy than it consumes on an annual average basis.
(d) Additional 500 kW and 750 kW engine installed, both with 90 percent availability.

9.3.3.3 Electrical System Improvements for Cogeneration Expansion

There should be conduits and other infrastructure in place at the abandoned fuel cell site to accommodate two new cogeneration engines. However, some electrical systems improvements are likely to be necessary. For example, the existing cogeneration engines are connected to Distribution Panel C, which is the same distribution panel that distributes energy out to the plant. Distribution Panel C has a bus rating of 4,000 amps. If an additional cogeneration engine was added, this distribution panel would need to be evaluated for adequate capacity. If Distribution Panel C does not have enough capacity, a new electrical tie-in may need to be installed.

9.3.4 Digester Gas Management Requirements

This section addresses the digester gas management considerations that will need to be considered when implementing the net zero energy goals laid out in the steps described above. The topics addressed include:

- Digester Gas Treatment
- Gas Conveyance
- Gas Storage

9.3.4.1 Digester Gas Treatment

The existing digester gas treatment system is sized for 180 cfm. However, the system currently operates at a treatment rate of approximately 140 cfm. At this reduced treatment capacity, some of the available biogas must be flared because the treatment system cannot process it. Given these capacity limitations, combined with the efficiency issues discussed previously, a new digester gas treatment system is recommended. This design would address the deficiencies of the current treatment system described in Section 9.2.2, while also performing at projected production rates in 2035.

In general, anticipated peak biogas production rates should be used as the basis of sizing the gas treatment system to avoid having the gas treatment system being the limiting factor on biogas use.

Table 9-8 shows the anticipated annual average, maximum month and peak biogas generation rates anticipated at each step. For this analysis, the peak generation rates are assumed to be ten percent higher than the maximum monthly digester gas production value. As shown, digester gas generation rates are expected to almost quadruple from the import of hauled organic materials.

Table 9-8. Digester Gas Treatment Requirements

Biogas Generation Steps		Hauled Organics Truckloads Per Day	Digester Gas Generated, cfm		
No.	Goal		Annual Average	Maximum Month	Peak
1	65 Percent Primary Solids Capture, No Hauled Waste	0	180	220	240
2	Maximize FOG Potential	4	280	320	350
3	Maximize Existing Cogeneration Capacity	~5	390	390	430
4	Maximize Existing Digester Capacity	~7	520	520	570
5	Maximize Hauled Waste Receiving Potential	10	750	780	860

This new gas treatment system would consist of the following components (operated in series):

- Hydrogen sulfide (H_2S) removal, which includes a separator and two Sulfatreat vessels operated in a lead-lag configuration with an H_2S monitoring sensor. Two structural platforms are also assumed to be needed to aid in the replacement of media. One of these platforms can also aide in the replacement of media in the nearby siloxane removal system.
- Air compressors that are skid mounted with a coalescing filter to remove moisture from the gas.
- A refrigerated dryer system, which includes a refrigeration unit, heat exchangers and coalescing filter to remove water from the digester gas.
- Siloxane removal system, which includes two siloxane removal vessels operating in a lead-lag configuration with a particulate filter to ensure media particles do not enter the gas flow.

A modular gas treatment approach is recommended to allow for expansion as biogas production increases over time. This approach involves installing gas treatment units sized for future growth, and installing compressors, as needed, to meet near-term biogas generation rates. The larger treatment modules will minimize the overall footprint area, which is critical for this site. In addition, having excess treatment capacity does not impact performance and simply means that the treatment system will require less frequent media changes. On the other hand, having smaller compressors will provide for more optimized energy efficiency.

In addition to biogas treatment, the improvements needed to reach Step 3 include the installation of hauled organic waste receiving facility and a FOG receiving facility, the latter of which is already under design. Thus, it is reasonable to assume that the District could reach Step 3 within a short period. Expansion beyond Step 3 will require significant investment in cogeneration and digestion capacity expansions, and the ability to get to these later steps is less certain.

For the reasons listed above, it is recommended that the first "module" include a gas treatment system sized for Step 3. As shown in Table 9-8, this would require a 430-cfm treatment system. This capacity is approximately half of the capacity anticipated at Step 5. Thus, two treatment modules would ultimately be installed. To provide the greatest flexibly with respect to compressor capacity, four compressors, sized at 110 cfm is recommended for each of the 430 cf treatment modules.

Table 9-9 provides a summary of the number of treatment units and compressors that would be provided at each step. As shown, the compressor capacity would be the limiting feature under most conditions, except at Steps 3 and 5. This table also shows that it may not be feasible to treat gas at the peak gas generation rate with the number of compressors provided in each step. If added capacity was desired however, an additional compressor could be added to increase the overall system capacity.

Table 9-9. Digester Gas Treatment System Sizing

Biogas Generation Steps		Hauled Organics Truckloads Per Day	Peak Digester Gas Generation Rate, cfm	Treatment System			
No.	Goal			Number of Treatment Modules	Number of Compressors	Total Capacity, cfm	Limiting Feature
1	65 Percent Primary Solids Capture, No Hauled Waste	0	240	1	2	220	Compressor Capacity
2	Maximize FOG Potential	4	350		3	330	Compressor Capacity
3	Maximize Existing Cogeneration Capacity	~5	430		4	430	Treatment Capacity
4	Maximize Existing Digester Capacity	~7	570	2	5	550	Compressor Capacity
5	Maximize Hauled Waste Receiving Potential	10	860		8	860	Treatment System

The first gas treatment module could be constructed behind Building D, which is where the current gas treatment system is located. The second treatment module would need to be constructed in a different location, and a site located in the parking area east of Digesters 1 and 2 is assumed. A preliminary layout of the gas treatment systems is presented on Figure 9-15.

To maintain near continuous operation of the existing cogeneration system, the new treatment facility would be operable before the existing system is decommissioned. As shown on Figure 9-15, one of the sulfatreat units would likely need to be constructed where the existing gas treatment system is located. However, only one sulfatreat unit is needed for biogas treatment when the media in the treatment unit is completely new. Therefore, construction of the new gas treatment system behind Building D could be phased, where the first sulfatreat unit and remaining treatment system is brought online before taking the old system offline. Then the second sulfatreat unit could be constructed after the old system is decommissioned.

Figure 9-15 also shows the planned location of the fourth digester and the FOG receiving facility, along with a potential location for a food waste receiving facility and the recuperative thickening process. As shown, there is limited space available for these facilities on the site, and the recuperative thickening system and the second gas treatment module would be located on the existing employee parking area, and could block a portion of the driveway to the Administration and Operations Building. With these potential new facilities, the District may need to develop additional employee parking around Storage Basin 3 and provide a covered walkway between the new parking area and the Administration and Operations Building. In addition, as shown on Figure 9-15, the main truck route through the WWTP site would remain clear, and this pathway could be used by staff to access the parking available directly in front of Administration and Operations Building.

9.3.4.2 Gas Conveyance

The capacity of the existing digester gas conveyance system, including the pipelines that carry biogas from the digesters to the treatment system and from the treatment system to the cogeneration engines is not currently known. Discussions with District staff do indicate that there may be pressure constraints, as the gas system must currently operate at a higher pressure than needed for the cogeneration engines to account for the losses that occur between the first engine and the last engine.

As the District increases digester gas production by adding external feedstocks, additional conveyance capacity may be needed. In addition, the gas conveyance system between the gas treatment system and the abandoned fuel cell site may not be sized adequately to convey the biogas needed to operate a 1,250 kW cogeneration system at this location.

Additional investigation into the gas conveyance capacity is recommended as part of the gas treatment system project discussed in the previous section. This analysis should include development of a piping pressure drop model of the conveyance system, using anticipated peak flows from each digester. This analysis should consider the impacts of increasing conveyance capacity on the required compressor sizing and the potential energy-saving benefits that could be realized if larger pipes were installed.

9.3.4.3 Gas Storage

The benefits of a biogas storage system should also be evaluated when considering a future expansion of the cogeneration capacity.

Digester gas storage systems are typically employed to manage gas flow to the engine generators when that flow exceeds the capacity of the engine. In the case of the District's system, the engine capacity far exceeds the gas generating capacity and the available digester gas can be burned in either of the other two rich burn engines (Engines 1 and 3) when blended with natural gas. From an energy generation standpoint, gas storage would only be needed if the biogas generation rates were at or near the engine capacity. Under the potential future condition, it may make sense to store digester gas that cannot be used immediately for energy generation. However, biogas production rates can also be controlled by managing feed rates to the digester. Therefore, the District should consider whether optimized hauled feedstock management can adequately control gas production rates before an investment in gas storage is made.

Gas storage could also be considered for holding excess digester gas when the digester gas production rate increases over the capacity of the digester gas treatment system. As discussed above, it is recommended that a new digester gas treatment system be installed that slightly exceeds the maximum month generation rate. This approach will remove the gas treatment system as a bottleneck in the process. Gas storage could also be used to hold biogas in the event the gas treatment system is not operating. However, cost savings from storing digester gas for the brief periods when the treatment system is out of service is likely not justified. Options for providing equipment redundancy could be considered when designing the digester gas treatment system if downtime is a major concern.

9.3.5 Additional Considerations for Digester Gas Power Generation

Additional considerations that need to be accounted for when planning and implementing a co-digestion project are discussed below under the following topics:

- Feed Stock Management
- Solids Management
- Digester Heating and Mixing
- Additional Air Pollution Control Equipment

9.3.5.1 Feed Stock Management

Having the ability to meter FOG and food waste streams into the digesters at a relatively uniform feed rate by providing hauled waste storage is important for digester health and gas production management. Adequate hauled waste storage also can help avoid delays during hauled waste deliveries. Thus, a hauled waste storage facility would be a critical component of a co-digestion program. In addition, separate storage facilities would be recommended for FOG and food wastes to allow for simultaneous delivery of these materials (assuming the plant can handle the necessary traffic). Finally, depending on the source of feedstock, specialized receiving equipment (e.g., heavy debris separators like rock traps, grinders, and strainers) may also be required for debris management.

Odor control could also be an issue for feed stock management. Odor control would be needed for either the FOG or food waste truck receiving stations only if the feedstock is exposed to the atmosphere. If the FOG or food waste feedstock is hauled in an enclosed tanker truck and pumped into the tanks without exposure, odor control is not needed at the truck unloading station. However, odor control would be recommended for the subsurface storage tank(s). If the waste receiving facilities include additional onsite processing or other factors that involve atmospheric exposure, these facilities would require robust odor control.

Adequate space should be reserved for the equipment described above, as well as for roadways for haulers to access the feedstock receiving system. Ideally, the location of hauled waste receiving facilities would be close to the digesters. Depending on the nature of the feedstock, conveyance may be difficult, so limiting the conveyance distance would be helpful. However, easy access for haulers would encourage participation and reduce impact on operations at the WWTP.

Recommended locations for a potential future FOG and/or food waste receiving facilities are identified in Chapter 8. These locations allow for reasonable truck access, and in the case of the food waste receiving facility, provide for the facilities to be in an odor-controlled building.

Except for Step 5, where the digester that is equipped with recuperative thickening would receive a higher volume of waste than the other digesters, it is recommended that the hauled waste feeding equipment be set up to allow for distribution of hauled waste into the digesters proportional to their capacities. The analysis presented herein assumes this approach. Dedicating one digester for hauled waste digestion introduces operational difficulties. First, because the District will require three digesters be available for WWTP solids when one of the digesters is taken out of service, the District would have to stop receiving hauled wastes altogether whenever a unit was down for maintenance. Whereas the District could still receive hauled waste at a reduced rate during a maintenance event if the hauled wastes were blended with WWTP solids in the available digesters. Second, if one digester receives a significantly greater volume of hauled waste than others, that system would produce digester gas at higher rates than the other digesters, thus potentially requiring additional gas conveyance and safety equipment. Therefore, unless all four of the digesters are designed to operate with higher gas production rates, the District could not alternate which digester received the hauled wastes.

9.3.5.2 Solids Management

Increasing the digester gas being generated at the WWTP will also increase the production of digested solids that will need to be managed by the District. The actual increase in solids production will be dependent on the sources of organic materials being sent to the digester. A conservative assessment of the amount of digested solids at each energy generation step was calculated based on the following assumptions:

- FOG will not increase biosolids generation rates. This is a slightly conservative assumption. Evidence suggests that FOG can decrease solids generation because it boosts the microorganism population resulting in higher breakdown of the WWTP solids.
- Food waste received will have a total solids content of 28 percent, a volatile to total solids ratio of 86 percent, and the food waste volatile solids will be reduced of 74 percent in the digesters. This is also a conservative assumption and is based on a food waste stream that consists of organic material diverted from a landfill. As noted previously, higher volatile content waste streams that are hauled to the WWTP from a food processing industry could result in zero additional solids production, and in some cases, could reduce solids production.

Table 9-10 shows the additional solids that could potentially be produced, based on the assumptions above, as compared to the 2035 with 65 percent solids capture baseline. As shown, the solids generation rates would almost double between Step 1 and Step 5.

Table 9-10. Additional Solids Generated from Co-Digestion

Biogas Generation Steps		Hauled Organics Truckloads Per Day	Digested Solids Production, dry tons/year	Additional Digested Solids Production, dry tons/year
No.	Goal			
1	65 Percent Primary Solids Capture, No Hauled Waste	0	2,620	0
2	Maximize FOG Potential	4	2,620	0
3	Maximize Existing Cogeneration Capacity	~5	3,610	990
4	Maximize Existing Digester Capacity	~7	4,930	1,320
5	Maximize Hauled Waste Receiving Potential	10	7,260	2,330

Table 9-11 shows the estimated additional solids management and disposal costs that would be associated with the increase in solids production. These costs were calculated using the estimated project costs for expanding from partial dewatering from the digesters to full dewatering from the digesters (shown in Chapter 8) and dividing that value by the additional dry tons per year that would be dewatered between those two scenarios. The estimated unit project cost is $3,150 per dry ton dewatered per year. The estimated operating costs were calculated by dividing the annual operating costs for full dewatering from the digesters by the total dry tons dewatered in that scenario. The estimated unit operating costs is approximately $470 per dry ton dewatered. This operating cost includes dewatering chemicals, increased electricity use, and hauling and disposal – where the combined cost of hauling and disposal is estimated to be $50 per wet ton. These unit capital and operating costs were then multiplied by the additional dry tons that would be generated under the co-digestion scenarios explored herein. The total lifecycle costs were then calculated as the sum of the project costs and the 30-year operating costs.

As shown in Table 9-11, the costs associated with biosolids management could be significant. However, some of these costs would be offset by tipping fees received for accepting the hauled waste materials. Additional discussion of the costs, energy savings, and revenues associated with an expanded biogas cogeneration system through incorporation of hauled wastes is provided in Section 9.4.

Table 9-11. Potential Cost Dewatering and Disposal of Additional Solids Generated from Co-Digestion

Biogas Generation Steps		Additional Hauled Organics Truckloads Per Day	Additional Digested Solids Production, dry tons/year	Potential Costs of Solids Dewatering, million $			
No.	Goal			Project Costs	Annual Operating Costs	30-Year Operating Cost[(a)]	Total Lifecycle Costs
1	65 Percent Primary Solids Capture, No Hauled Waste	-	-	-	-	-	-
2	Maximize FOG Potential	4	-	-	-	-	-
3	Maximize Existing Cogeneration Capacity	~1	990	3.1	0.5	10.0	13.1
4	Maximize Existing Digester Capacity	~2	1,320	4.2	0.6	13.3	17.5
5	Maximize Hauled Waste Receiving Potential	3	2,330	7.3	1.1	23.6	30.9
Total Potential Solids Handling Costs		**10**	**4,640**	**14.6**	**2.2**	**46.9**	**61.5**

(a) 30-year present worth value of annual operating costs.

9.3.5.3 Digester Heating and Mixing

To accommodate the co-digestion program, the digesters would need to include a robust mixing system and adequate heating capacity for the additional waste stream.

The mixing system of the current digesters should be adequate as long as the solids content of the digester is not significantly increased. This is not expected to be a problem with FOG addition. Food waste is typically added in at a solids content of 10 percent solids or less. As more food waste is added to the digester, the solids content of the materials fed to the digesters is expected to increase to 4.0 and 4.8 percent solids for Steps 3 and 4, respectively (assuming the feed stocks are evenly added to each digester). However, the solids content in the digester itself would be lower than what is in these feed rates as the solids are digested. Digester mixing systems typically operate with roughly 5 percent solids, so additional mixing improvements should not be necessary unless feed stocks are not added evenly across digesters. In Step 5, one of the large digesters with the high solids digestion system should have a higher solids content, while the other digesters would maintain the solids content assumed for Step 4. However, the high solids digestion system includes a new mixing system to mix a higher solids content.

The capacity of the digester heat exchangers should also be considered as new feedstocks are introduced. Heat exchanger capacity is expressed as the amount of heat transferred per hour. Digesters 1 and 2 each have heat exchangers with capacities of 0.77 MM BTU/hr, while Digester 3 has a heat exchanger with a capacity of 1.24 MM BTU/hr. If Digester 4, which will be the same size as Digester 3, has the same heat exchanger capacity, the total heat exchanger capacity will be 4.02 MM BTU/hr.

The digester heating needs can be calculated as the sum of the convective heat losses from the digester and the heating demands to bring the incoming solids up to required temperature of 98 °F. For the WWTP digesters, these two values are calculated as follows:

- The convective losses have been calculated at 0.55 MM BTU/hr and 1.49 MM BTU/hr, in the summer and winter, respectively.
- The incoming solids need to be increased to 98 °F from 70 °F and 64.6 °F in the summer and winter, respectively. Assuming a heat demand of 8.33 BTU per gallon per degree F, a heat demand of 233 BTU/gal in the summer and 278 BTU/gal in the winter is needed.

These two values can be used to back calculate the heating capacity as follows:

$$Heating\ Capacity\ \left(\frac{gal}{day}\right) =$$

$$\frac{Heat\ Exchanger\ Capacity\ \left(\frac{MMBTU}{hr}\right) - Convectrive\ Heat\ Loss\ (\frac{MMBTU}{hr})}{Heat\ Required\ for\ Incoming\ SOlids\ \left(\frac{BTU}{gal}\right) x\ 10^{-6}\ \left(\frac{MMBTU}{BTU}\right)\ x\ 24\ (\frac{hr}{day})}$$

Using the equation, the total capacity of all heat exchangers is 357,000 and 218,000 gallons per day in the summer and winter, respectively. This heating capacity should be adequate through Step 4, which requires 178,000 gallons/day of heating at the maximum month condition. Like the digester mixing system, the addition of the high solids digestion system on one digester also will include a new heat exchanger for higher capacity.

9.3.5.4 Additional Air Pollution Control Equipment

Chapter 4 includes discussion of the BAAQMD's proposed Rule 11-18, which could impact development of a co-digestion project and expansion of the cogeneration system if additional air pollution control equipment is needed for either existing or expanded digestion or cogeneration equipment. Assuming the rule is adopted, its requirements would need to be considered during preliminary design of equipment that could increase emissions of toxic air pollutants.

Whether Rule 11-18 would impact the existing or proposed WWTP facilities has not been evaluated at this time. Therefore, this Chapter does not include further discussion of the rule or its impacts.

9.3.6 Summary of Potential Biogas Cogeneration Facilities for Achieving Net Zero Energy

Figure 9-16 shows the sources of energy that would be available to meet the annual average energy demands for each of the Biogas Generation steps discussed in this Chapter, where the primary sources of energy would be onsite generation using biogas, onsite generation using natural gas, and purchasing electricity from PG&E. The additional facilities required at each step are:

- Step 1:
  - A fourth digester (under design)
  - A new gas treatment system
- Step 2:
  - A FOG receiving facility (under design)
  - Expansion of the gas treatment system
- Step 3:
  - A food waste receiving facility
  - Expansion of the gas treatment system
  - Expansion of the biosolids dewatering system
- Step 4:
  - An additional cogeneration capacity (500 kW)
  - Expansion of the gas treatment system
  - Expansion of the biosolids dewatering system

- Step 5:
  - A high solids digestion system on one of the large digesters
  - An additional cogeneration capacity (750 kW)
  - Expansion of the gas treatment system
  - Expansion of the biosolids dewatering system

Expanding the biogas cogeneration system beyond Step 3 presented herein requires a major investment in cogeneration capacity and/or digestion capacity, as well as a major expansion of the solids dewatering and disposal systems. Moreover, it is not clear whether it will be possible to obtain the hauled waste materials needed to reach the net zero energy target defined in Step 5.

9.3.7 Potential Solar Power Generation Facilities

The previous sections outlined a strategy for moving toward a net zero energy goal by increasing digester gas production through a hauled waste co-digestion program. As discussed, there would be a net onsite renewable energy deficit even if the most aggressive strategy (Biogas Generation Step 5) is pursued. As shown in Table 9-7, the required solar acreage needed to reach net zero energy under Step 5 is 90 kW. Applying a typical design value of approximately 200 kW of net annual average solar generating capacity can be provided on five acres, an estimated 2.3 acres of solar panels would need to be installed. These solar panels could be installed on the roof of several larger buildings at the main WWTP site and in Holding Basins 2 and 3.

It is also unclear whether the District will be able to reach the co-digestion targets outlined in Steps 3, 4 and 5 due to the availability of high-value hauled organic waste streams and the potential costs associated with management of additional solids. If this is the case, then a significant amount of additional solar generating capacity would be necessary to reach the net zero energy target. Table 9-12 shows the required solar acreage needed to reach net zero energy under Steps 3, 4 and 5 based on the annual average renewable energy deficit values that were shown in Table 9-7 and the assumption that approximately 200 kW of net solar generating capacity will require approximately five acres of solar panels. As shown, up to 28 acres of solar panels would be required if the cogeneration system is not expanded beyond Step 3.

Table 9-12. Additional Solar Power Generation and Area Required for Annual Average Conditions

Biogas Generation Steps		Power from Digester Gas, kW	WWTP Power, Demand[a], kW	Onsite Renewable Energy Deficit [b], kW	Additional Solar Area Needed, acres
No.	Goal				
3	Maximize Existing Cogeneration Capacity	1,120	2,220	1,100	28
4	Maximize Existing Digester Capacity	1,500	2,230	730	18
5	Maximize Hauled Waste Receiving Potential	2,180	2,270	90	2.3

[a] Power demand increases with increased digester gas treatment.
[b] From Table 9-7.

The solar acreage shown in Table 9-12 represent annual average conditions. Higher WWTP power demands are expected in the summer and lower power demands are expected in the winter. Table 9-13 shows the onsite total energy production deficit that would occur in the summer months if the solar facilities that were presented in Table 9-12 to achieve net zero energy are installed. This value represents the amount of electricity that would need to be purchased from PG&E. As shown, under the three net zero energy scenarios shown, the power purchased from PG&E during the summer months would be less than the current purchase amount, which averages to approximately 580 kW.

Table 9-13. Solar Power Generation Deficit in Summer Months

Biogas Generation Steps		Additional Solar Area Needed for Net Zero Energy, acres	Summer Months		
No.	Goal		WWTP Power, Demand[(a)], kW	Onsite Renewable Energy Deficit with Solar, kW	Onsite Total Energy Deficit with Solar, kW
3	Maximize Existing Cogeneration Capacity	28	2,630	410	410
4	Maximize Existing Digester Capacity	18	2,640		340
5	Maximize Hauled Waste Receiving Potential	2.3	2,680		345

(a) Power demand increases with increased digester gas treatment.

9.3.8 Potential Layout of Facilities

Figure 9-17 shows a potential site layout for the two net zero energy scenarios described above. The food waste receiving station, FOG receiving facility, high solids digestion system, and gas treatment system were all placed close to the digester system to minimize required piping and pressure loss. As noted previously, the additional cogeneration engines are in the area previously occupied by the abandoned fuel cell system to take advantage of existing space and infrastructure in this area.

Figure 9-17 also shows the potential location for construction of a solar power facility that could be provided to reach net zero energy if expansion beyond Steps 3 or 4 was determined to not be economically feasible. As shown, it is assumed that the solar facility would be constructed within a portion of the DLD site. Thus, even with the solar power option, the District would need to expand the biosolids dewatering system to accommodate the loss of disposal capacity associated with the DLD site.

9.4 POTENTIAL NET ZERO ENERGY PROJECT COSTS AND REVENUES

Digester gas production will be increased through the stepwise approach detailed in the previous section. The financial feasibility of each digester gas production step depends on the estimated costs and potential revenues, which vary for each step. The initial steps (1 and 2) can be more clearly defined than those of later steps. Thus, the potential cost and revenue information is presented as follows:

- Estimated Cost and Revenues for Initial Steps
- Cost Considerations for Expansion beyond Step 2
- Summary of Potential Costs and Revenues
- Potential Grant Funding Opportunities

9.4.1 Estimated Cost and Revenues for Initial Steps

The District is currently moving forward with the initial steps (Steps 1 and 2), which include improving primary solids capture and adding a FOG receiving station; the potential cost and revenues associated with these two steps has generally been defined. The potential costs and revenues associated with these two projects include:

- Fog Receiving Facility Project Costs
- Gas Treatment System Project Costs
- Energy Production Cost Savings
- Potential Tipping Fee Revenues
- Summary of Potential Costs and Revenues

9.4.1.1 FOG Receiving Facility Project Costs

The FOG receiving facility includes the truck receiving station, equipment to connect to the FOG trucks, the tanks for the FOG, and the piping and pumps into the digesters. The estimated project cost of the FOG receiving facility is $1.2 million dollars. This project cost was calculated from an estimated capital cost of $0.94 million for the FOG receiving facility that was provided by District staff.

9.4.1.2 Gas Treatment System Project Costs

As discussed in Section 9.3.4, a new gas treatment system should be installed to replace the current gas treatment system. For each of the treatment systems, the H_2S removal, refrigerated dryer system and the siloxane removal system would be sized to handle 430 cfm of flow. To reach Step 2, three, 110 cfm compressors would also be installed. For this analysis, it has been assumed that the compressors would increase gas pressure to 35 psi. As discussed previously, a more detailed assessment of the gas pressure requirements will need to be completed in conjunction with an assessment of the gas conveyance system.

A summary of the estimated capital costs associated with constructing one gas treatment system module and the necessary compressors for each step is provided in Table 9-14. These costs were developed following the procedures detailed in Chapter 5; cost estimating details are provided in **Appendix H-14**.

Table 9-14. Estimated Cost for Gas Treatment System

Project Component	Estimated Cost, million $	
	Step 1	Step 2
Hydrogen Sulfide Removal	0.7	0
Air Compressor and Coalescing Filter	0.7	0.4
Refrigerated Dryer System	0.7	0
Siloxane Removal System and Filter	0.6	0
OPCC	**2.7**	**0.4**
Construction Contingency, 10 percent	0.3	0.04
Total Estimated Construction Cost	**3.0**	**0.4**
Engineering, Legal and Administrative Costs, 35 percent[(a)]	1.0	0.2
Total Project Costs	**4.0**	**0.6**
Total Project Costs with Initial Project	**-**	**4.6**

(a) Calculated as a percentage of the OPCC

9.4.1.3 Energy Production Cost Savings

Cost savings from the production of energy are presented in Table 9-5. Cost savings consider the savings from annual power generated onsite less the amount of natural gas that would need to be purchased, using Step 1 as a baseline. The annual cost savings under Step 2 is estimated as $310,000 per year; net present value of annual cost savings for Step 2 was calculated to be $6.6 million.

9.4.1.4 Potential Tipping Fee Revenues

The District will receive a tipping fee for the receipt of FOG. For the purposes of the analysis, a tipping fee of $0.04/gallon was assumed. This tipping fee was based on the fee currently received at the EBMUD facility. Tipping fees are sensitive to the supply and demand of the surrounding areas and will vary as the District attempts to increase digester gas production. For Step 2, the District will add 12,000 gpd of FOG to the Digesters. The tipping fee for 12,000 gpd of FOG equates to $175,000 per year in revenue or a net present value of $3.7 million over a 30-year period, calculated as discussed in Chapter 5.

9.4.1.5 Summary of Costs Versus Potential Revenues

The costs associated with Step 2 include the construction of a new gas treatment system and a FOG receiving facility. The total project costs associated with these facilities are estimated to be approximately $5.8 million. The potential savings and revenues under Step 2 are $310,000 each year for energy savings and $175,000 in tipping fee revenue. Using a simple payback period calculation, the project costs associated with Step 2 would be recuperated within 12 years.

9.4.2 Cost Considerations for Expansion Beyond Step 2

The potential costs and revenues associated with expansion of the biogas cogeneration system beyond Step 2 are addressed in this section under the following topics:

- Food Waste Receiving Facility
- Cogeneration Expansion
- High Solids Digestion
- Gas Treatment, Storage and Conveyance
- Solids Handling
- Potential Revenues and Savings

9.4.2.1 Hauled Organic Waste Receiving Facility

The cost of a hauled organic waste receiving station will vary depending on the type of food waste received and whether onsite processing is required. For comparison, the FOG receiving facility is estimated to cost $1.2 million. However, the food waste receiving station may also need debris removing equipment or blending equipment.

If the District partners with a food waste processing company, the processing facilities may be co-owned by the District and the private company. The initial investment in facilities would be negotiated along with the tipping fee. A larger tipping fee might require more initial District investment. For these reasons, the costs of the food waste receiving facilities will need to be explored in further depth as the District gets closer to implementing these steps. This would include the evaluation of multiple food waste sources, the cost needed to build the processing facilities, and the tipping fee received.

9.4.2.2 Cogeneration Expansion

Cogeneration expansion costs include the costs for the additional engines. As discussed in Section 9.3.3, increasing digester gas production to Step 4 would include adding a 500-kW engine. Increasing digester gas production to Step 5 would include adding a 750-kW engine. The components associated with each expansion step would include the engine generator, an exhaust gas heat exchanger, an engine jacket water and aftercooler expansion tank, jacket water pump, after water pump, heat exchangers, emission controls, PG&E interconnection switchgear, and sound-control enclosure. The project costs for this equipment, is estimated to be $5.1 million and $5.5 million for the 500 kW and 750 kW cogeneration engines, respectively, based on quotes received from engine generator vendors.

For the cogeneration system, significant savings could be realized if a 1,250-kW engine were installed instead of two smaller engines. A 1,250-kW cogeneration engine is estimated to cost $6.3 million, thus an overall savings of $4.3 million could be realized. However, the initial investment would be higher.

The cost savings per kWhr of purchasing and burning natural gas, compared to purchasing power from PG&E can be determined from the following equation:

$$Cost\ Savings\ per\ kWhr\ Produced\ \left(\frac{\$}{kWhr}\right) =$$

$$Cost\ of\ Electricity\ \left(\frac{\$}{kWhr}\right) - \frac{Cost\ of\ Natural\ Gas\ \left(\frac{\$}{therm}\right)}{8.8\frac{kWhr}{therm^2}}$$

$$- Maintenance\ Costs\ of\ 0.03\ \left(\frac{\$}{kwhr}\right)$$

At the current cost of electricity and natural gas as $0.15/kWhr and $0.31/therm, respectively, the estimated cost savings would be $0.08 per kWhr produced. Assuming a 90 percent availability of the additional 750 kW capacity, the engine would produce 5.9 million kWhr-year. Thus, the savings would be approximately $470,000 per year. As presented above, the difference in cost between the 500-kW engine and the 1,250-kW engine is $1.2 million. Thus, using a simple payback period calculation, the added cogeneration system capacity would have a two and a half-year payback.

Other advantages of installing a larger engine initially would include allowance for more biogas to be burned if generation rates exceed those predicted at Step 4, potential reduced need for gas storage during peak production times, and a smaller overall footprint.

[2] Assumes a generator efficiency of 0.3.

Finally, the size of the cogeneration system expansion will need to consider the near-term energy demands that are anticipated. For the analysis presented herein, the WWTP energy demands are based on the estimated 2035 Planned conditions. However, the actual energy demands in 2035 will depend on what treatment is required (or desired in the case of potable reuse opportunities).

9.4.2.3 High Solids Digestion

The high solids digestion system would be installed on one of the large digesters. Cost estimates for a high solids digestion system were developed in Chapter 8 for a high solids digestion system installed on two of the small digesters. Because the large digester has approximately 90 percent of the volume of the combined smaller digesters, the cost for the high solids digestion system should be roughly the same. The total project cost estimated in Chapter 8 for the high solids digestion system is $9.6 million

9.4.2.4 Gas Treatment, Storage and Conveyance

Costs for the gas treatment expansion are based on the modular costs described above. The number of modules and compressors is discussed in Section 9.3.4.1. Step 3 includes the cost for one treatment module with four compressors. Step 4 includes the cost of two modules with a total of five compressors. Step 5 includes the costs of two modules with eight compressors. Costs for the gas treatment expansion are presented below in Table 9-15.

Table 9-15. Estimated Additional Cost for Gas Treatment

Project Component	Estimated Cost for Initial Steps, million $	Estimated Additional Cost, million $		
		Step 3	Step 4	Step 5
Hydrogen Sulfide Removal	0.7	0.0	0.7	0.7
Air Compressor and Coalescing Filter	1.1	0.4	0.8	1.9
Refrigerated Dryer System	0.7	0.0	0.7	0.7
Siloxane Removal System and Filter	0.6	0.0	0.6	0.6
OPCC	**3.1**	**0.4**	**2.8**	**3.9**
Construction Contingency, 10 percent	0.3	0.04	0.3	0.4
Total Estimated Construction Cost	**3.4**	**0.4**	**3.1**	**4.3**
Engineering, Legal and Administrative Costs, 35 percent[(a)]	1.2	0.2	1.0	1.4
Total Project Costs	**4.6**	**0.6**	**4.1**	**5.7**
Total Project Costs with Initial Project		**5.2**	**8.7**	**10.3**

(a) Calculated as a percentage of the OPCC

As discussed previously in this Chapter, biogas storage can be used if the biogas production rates exceed the capacity of the gas treatment system or the cogeneration system, or if part of these systems are down for maintenance. The sizing of a gas storage system will depend on a number of factors. Ultimately, the decision to install gas storage will need to be evaluated based on the potential revenue benefits that could be realized from increased ability to use digester gas.

Finally, as previously discussed, the capacity of the biogas conveyance system will be needed from the digester gas treatment system to the new cogeneration engines. A conveyance system was constructed along with the fuel cell system to convey gas from the digesters to the fuel cell. However, the sizing of this system and its capacity are not known. For Steps 4 and 5, which include new cogeneration engines, it is unknown if the piping is adequate to handle digester gas that would be sent to the new cogeneration engines. The costs associated with conveyance system improvements cannot be determined based on the information that is currently available.

9.4.2.5 Solids Handling

The cost of solids handling was estimated in in Table 9-11. Project costs ranged from $3.1 to $14.6 million and annual operating costs ranged from $0.5 to $2.2 million per year. As discussed previously, these costs are highly variable depending on the volatile content of the organic materials that are ultimately received. In addition, these costs assume that the building for the solids handling equipment would not need to be larger to handle the increased solids that would need to be processed from the increased organic loadings to the digesters.

9.4.2.6 Potential Revenues and Savings

Revenues from each option include both the savings associated with not having to purchase power from PG&E as well as the tipping fee received for the receipt of FOG or food waste. The estimated annual power savings at each step were presented in Table 9-5. The tipping fee for FOG was discussed in Section 9.4.1.4 and is estimated at $175,000 per year. Assuming the same a $0.04/gallon tipping fee at a 10 percent solids content, a tipping fee of approximately $0.05 per pound of dry solids received may be realized. This results in a tipping fee of approximately $100 per dry ton received. The estimate tipping fee revenue for food waste are presented in Table 9-16.

Table 9-16. Summary of Potential Tipping Fees From Hauled Food Waste

Steps	Food Waste Received		Annual Revenue, million $/year	Net Present Value, million $
	wet tons/day	dry tons/day		
3	26.8	7.5	0.3	6.4
4	62.3	17.4	0.6	12.9
5	125	35.0	1.3	27.9

(a) Net Present Value is based on a 30-year period.

9.4.3 Summary of Potential Costs and Revenues

This section provides a summary of the identified costs and revenues associated with a potential net zero energy program at the WWTP.

9.4.3.1 Potential Costs and Revenues for an Expanded Biogas Cogeneration Program

The cost considerations discussed above are summarized below in Table 9-17. For the project to be financially feasible, a reasonable payback period would need to be realized. As shown in Table 9-17 the payback period for the improvements needed to reach net zero energy increases significantly in Steps 4 and 5.

The payback calculated for facilities required beyond Step 2 in Table 9-17 does not include the food waste receiving facility. If the facility is something like the FOG Receiving Facility (or if the FOG Receiving Facility can be used for both FOG and food waste) then the payback period would not change significantly from what is shown. As noted previously, the District may also be able to establish a public-private partnership for ownership of the receiving facility. However, if a more elaborate receiving facility needs to be constructed by the District to provide pre-treatment of the hauled waste or to blend the hauled waste solids with water to achieve a more dilute stream to feed into the digesters, then the payback period for the facilities could increase significantly.

A significant portion of the costs associated with increased onsite energy generation through a hauled waste program in Steps 4 and 5 of Table 9-17 is the costs associated with management and disposal of the additional solids materials that are received from outside sources. As discussed previously in this Chapter, the amount of solids that would ultimately require dewatering and disposal is dependent on the volatile content of the material that is being digested. Thus, careful consideration of these impacts must be considered as the District is expanding its organics waste receiving program beyond receiving high-volatile content FOG.

Table 9-17. Summary of Potential Revenues and Savings for Expanded Biogas Generation

Component	Initial Steps (1 and 2)	Step 3	Step 4	Step 5
Project Costs for Cogeneration System Expansion Components, million $				
FOG Receiving Facility	1.2	-	-	-
Food Waste Receiving Facility	-	Unknown	-	-
Cogeneration Expansion	-	-	5.1	5.5
High Solids Digestion	-	-	-	9.6
Gas Treatment	4.6	0.6	3.5	1.6
Gas Storage and Conveyance	Unknown			
Solids Handling Facilities	-	3.1	4.2	7.3
Total Known Project Costs	**5.8**	**3.7**	**12.8**	**24.0**
Cumulative Known Project Costs	**5.8**	**9.5**	**22.3**	**46.3**
Cumulative Revenues and Savings, million $/year				
FOG Tipping Fee	0.2	0.2	0.2	0.2
Food Waste Tipping Fee	-	0.3	0.6	1.3
Power Production Savings	0.3	0.6	1.0	1.7
Total Annual Revenues and Savings	**0.5**	**1.1**	**1.8**	**3.2**
Annual Solids Handling Costs, million $/year				
Cumulative Solids Dewatering and Disposal Costs	-	0.5	1.1	2.2
Net Revenue/Savings and Simple Payback on Cumulative Project Costs				
Net Revenue Savings, million $/year	0.5	0.6	0.7	1.0
Simple Payback, years	<12	<16	<32	<47

9.4.3.2 Potential Costs and Revenues for an Expanded Biogas Cogeneration Program Coupled with Solar Power Generation to Achieve Net Zero Energy

To achieve net zero energy, additional solar facilities would need to be installed. Moreover, given some of the payback periods shown in Table 9-17 at Steps 3 through 5, it may be more reasonable for the District to pursue a solar project in lieu of increased onsite organics digestion. Table 9-18 shows the potential costs and payback associated with achieving net zero energy at Steps 3 through 5 by meeting additional onsite energy demands with a solar installation located at the DLD site and/or WWTP plant site (as shown on Figure 9-17). As shown, the costs for solar include the cost to dewater and dispose of additional biosolids that can no longer be applied to the DLD, due to reduced capacity.

Comparing Table 9-16 to Table 9-18 shows that beyond Step 3 the addition of solar produces a slightly more favorable payback period.

Table 9-18. Summary of Potential Revenues and Savings for Expanded Biogas Generation Coupled with Solar Power Generation

Component	Step 3[(a)]	Step 4	Step 5
Project Costs for Cogeneration System Expansion Components, million $			
FOG Receiving Facility	-	-	-
Food Waste Receiving Facility	Unknown	-	-
Cogeneration Expansion	-	5.1	5.5
High Solids Digestion	-	-	9.6
Gas Treatment	0.6	3.5	1.6
Gas Storage and Conveyance	Unknown		
Solids Handling Facilities	3.1	4.2	7.3
Total Known Project Costs	**3.7**	**12.8**	**24.0**
Cumulative Known Project Costs	**9.5**	**22.3**	**46.3**
Cost of Solar Required to Get to Net Zero	**30.3**	**20.1**	**2.8**
Total Cumulative Costs with Solar	**39.8**	**42.4**	**49.1**
Cumulative Revenues and Savings, million $/year			
FOG Tipping Fee	0.2	0.2	0.2
Food Waste Tipping Fee	0.3	0.6	1.3
Power Production Savings	2.0	1.9	1.8
Total Annual Revenues and Savings	**2.5**	**2.7**	**3.3**
Annual Solids Handling Costs, million $/year			
Cumulative Solids Dewatering and Disposal Costs	0.8	1.3	2.2
Net Revenue/Savings and Simple Payback on Cumulative Project Costs			
Net Revenue Savings, million $/year	1.8	1.5	1.1
Simple Payback, years	<23	<29	<45

(a) Known project costs for Step 3 were added to known project costs from Step 2 in Table 9-17 to calculate cumulative costs.

9.4.3.3 Potential Cost and Revenues for a Small-Scale Solar Power Generation System

The District may consider adding the 2.3 kW solar facilities at the WWTP site now, as means of offsetting current WWTP energy demands. A solar analysis was completed to assess the potential costs and savings of this system over a 25-year period. This cost analysis calculated the energy costs, based on time of use energy demands for each year, both with and without solar. Electricity costs for each year were then compared and a savings was calculated for each year. Finally, a NPV of the total savings over the 25-year period was then calculated.

This analysis accounts for plant's site-specific time of use energy costs based on the following:

- The District's seasonal time of use power consumption data from January 2014 to July 2017.
- Current time of use PG&E tariffs assuming the E-20P rate schedule.
- A utility cost escalation of 3.5 percent per year.

The analysis also calculates the site-specific solar energy production based on the following:

- Solar production rates at specific WWTP location.
- The actual number of hours of sunlight each day of the year at specific WWTP location.
- An annual average production rate of 200 kW per 5 acres.
- A reduction in solar production due to panel age of 0.5 percent per year.

Finally, the costs associated with the solar facilities was calculated based on the following assumptions:

- A rated capacity of 1,000 kW of energy generated per 5 acres.
- A solar panel cost of $6 per watt of rated capacity, which accounts for added costs associated with the distributed system.
- A solar maintenance cost of $0.01/kWhr of production capacity (Assuming labor costs of $150 per hour, and dividing this costs by total annual average solar output).

Assuming the District stays on the E-20P rate schedule, the electrical power cost savings over a 25-year period have an NPV of $1.94 million compared to a cost of $2.76 million for a 2.3 acre solar installation. This would result in a loss of $0.8 million (as shown as the "Result NPV" in the **Appendix O-1**).

However, once solar panels are installed, PG&E would offer the District the option to switch to the E-20R-P tariff schedule. If the District were to switch to the E-20R-P tariff schedule with solar panels, the District could realize a payback in approximately 20 years. On the E20R-P rate schedule, the electrical power cost savings over a 25-year period have an NPV of $3.49 million compared to a solar installation cost of $2.76 million. The total savings over the 25-year period has an NPV of $0.73 million (as shown as the "Result NPV" in the **Appendix O-2**).

Although this analysis indicates modest payback over the 25-year period, the savings associated with a smaller solar installation are not substantial. However, if the District can take advantage of the potential funding sources through a third-party agreement (as discussed further below), the potential payback would be better.

9.4.4 Potential Funding Sources

Grant funding opportunities can be used to partially reduce the cost of the installation of the cogeneration system, thus reducing the total payback period anticipated. Currently, the Greenhouse Gas Reduction Grant and Loan Program, which supports the California Global Warming Solutions Act (Assembly Bill 32), provides funds to support expanded organics infrastructure. Another program, called the Self Generation Incentive Program, provides incentives for new energy, emerging generation technologies such as waste heat to power, biogas, and energy storage.

For solar installations, funding is available in the form of federal tax credits. This source can cover approximately 30 percent of the solar installation costs. However, because the District does not pay federal taxes, this credit is not available. One potential avenue for realizing the benefits of the federal tax credit would involve establishing a third-party agreement (often referred to as P3) for the solar facility. Under this scenario, a private entity would own the solar facilities and the District would buy power from this party at a rate that is lower than the power rate paid to PG&E. With this approach, the cost of the solar installation would be reduced, and some of these savings would be passed on to the District.

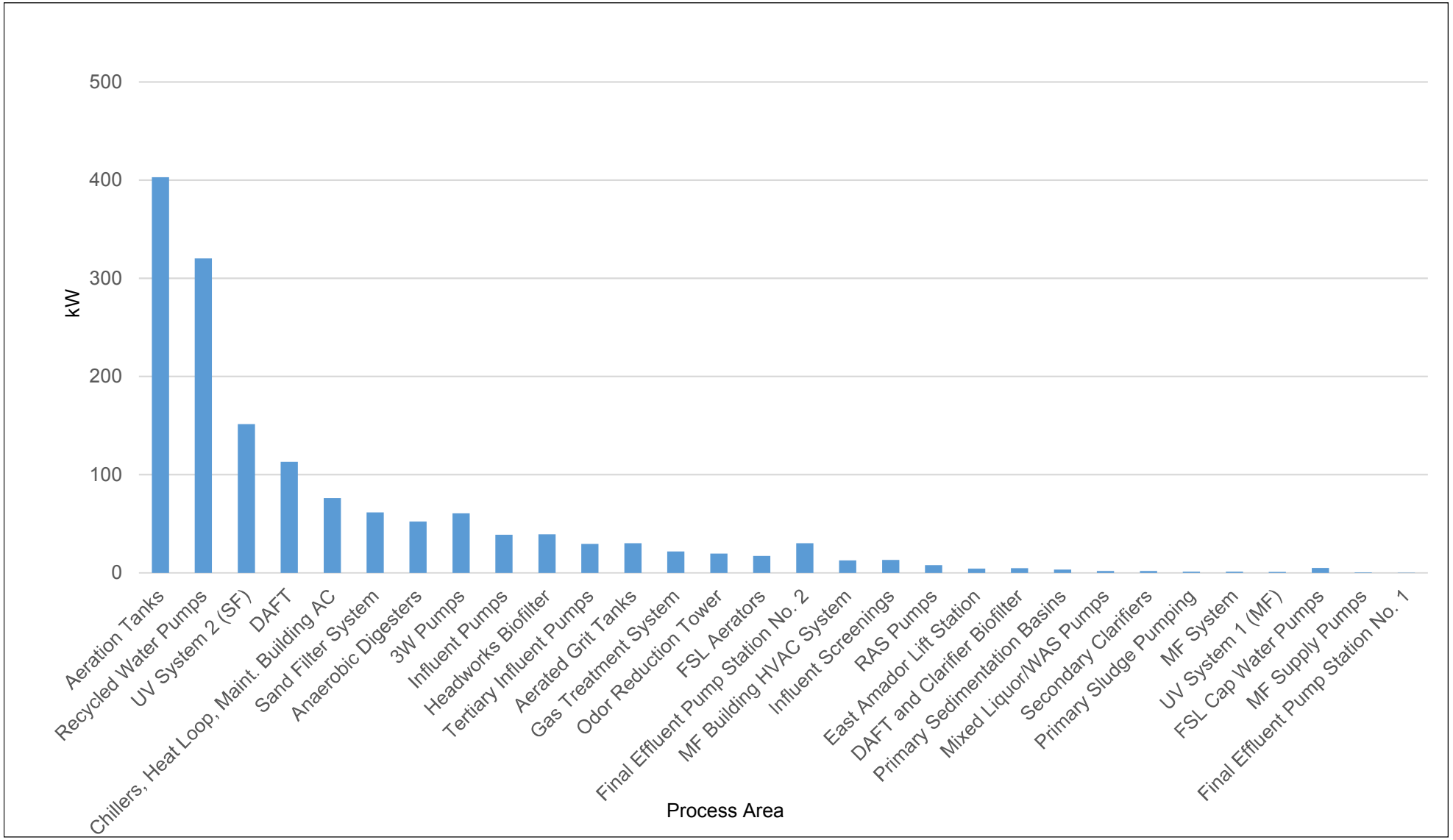


Figure 9-1

Current Power Demand - Summer

Dubin San Ramon Services District
Wastewater Treatment Plant
Master Plan

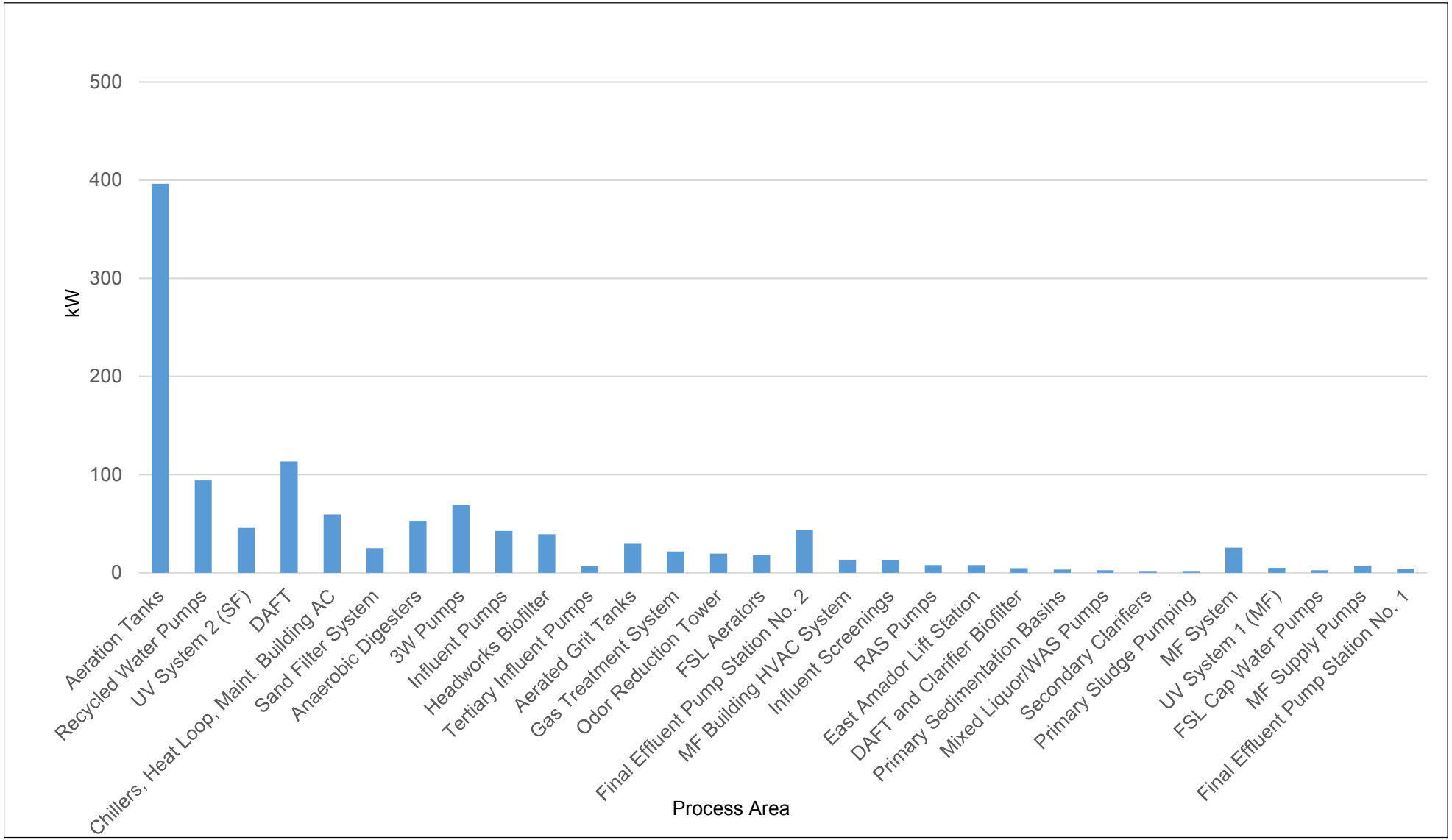
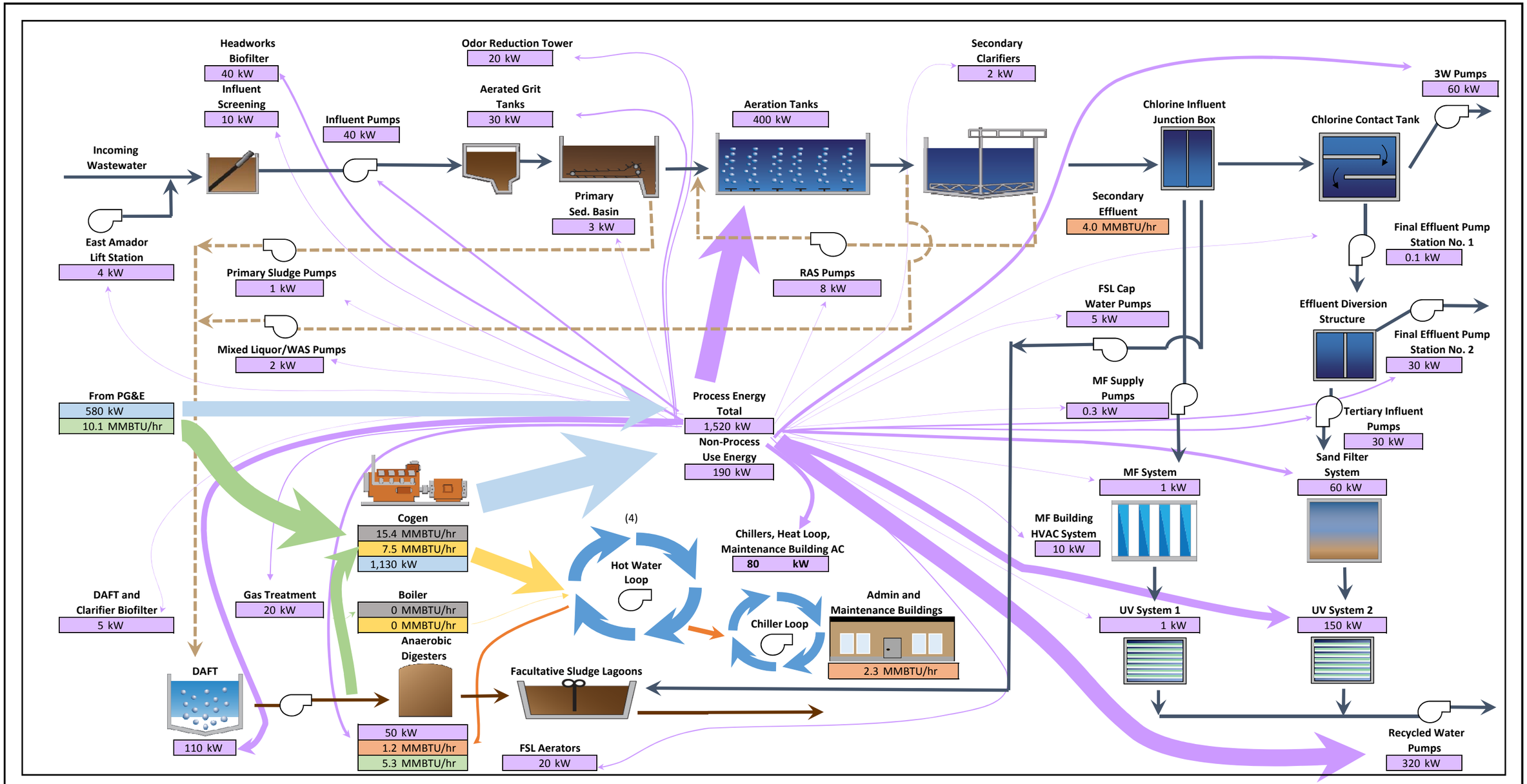


Figure 9-2

Current Power Demand - Winter

Dubin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Symbology

1,000 Power Supply	Power Supply
1,000 Power Demand	Power Demand
1,000 Gas Supply	Gas Supply
1,000 Gas Demand	Heat Supply
1,000 Heat Supply	Heat Output
1,000 Heat Output	Water Loop

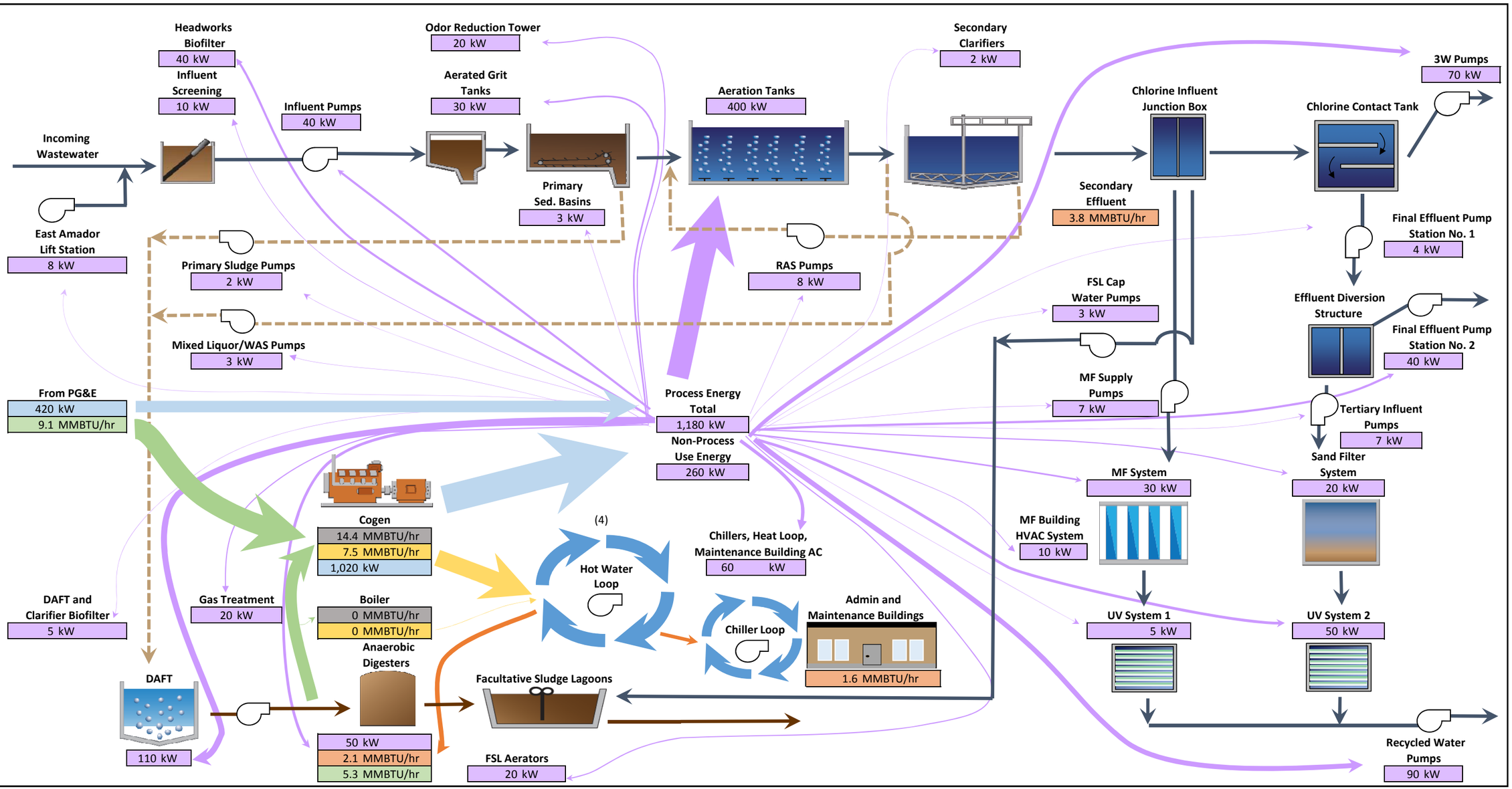
Notes:
1. Digester heat production calculated assuming current gas production of 160 scfm.
2. Digester heat demand based on six month average heat loss.
3. Power delivered to FSL Aerators separately and is not included in electricity total.
4. Hot Water Loop discharges 4.0 MMBTU/hr of waste heat to the Secondary Effluent.

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Figure 9-3

Current Conditions - Summer (May - October)

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Symbology

1,000 Power Supply	Power Supply
1,000 Power Demand	Power Demand
1,000 Gas Supply	Gas Supply
1,000 Gas Demand	Heat Supply
1,000 Heat Supply	Heat Output
1,000 Heat Output	Water Loop

Notes:
1. Digester heat production calculated assuming an increase in influent flow to 14.0 mgd.
2. Digester heat demand based on six month average heat loss.
3. Power delivered to FSL Aerators separately and is not included in electricity total.
4. Hot Water Loop discharges 3.8 MMBTU/hr of waste heat to the Secondary Effluent.

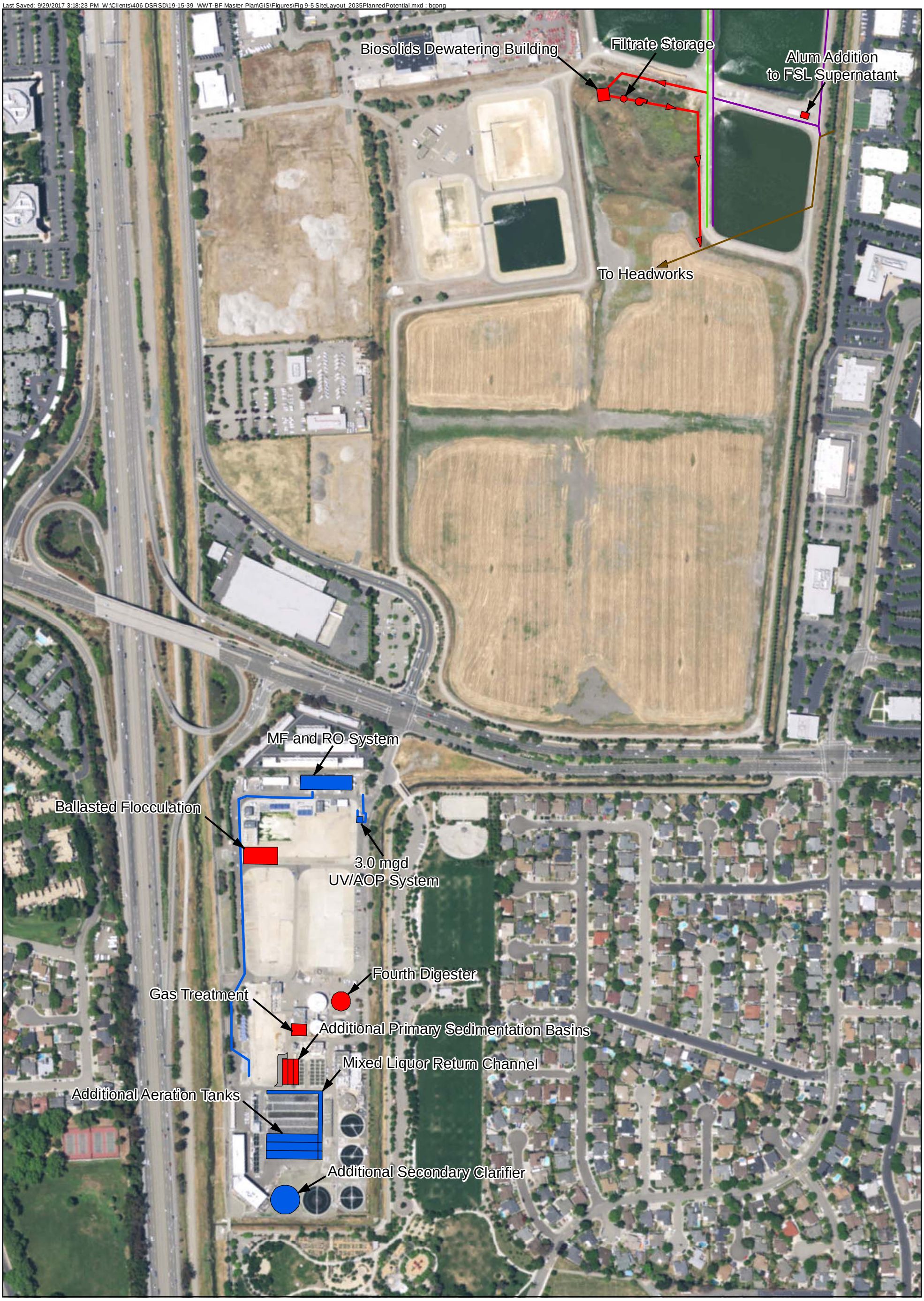
WEST YOST ASSOCIATES
Consulting Engineers

Figure 9-4

Current Conditions - Winter (November - April)

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- 2035 Planned Facilities
- 2035 Potential Facilities
- Existing FSL Supernatant Pipeline
- Existing Harvested Biosolids Pipeline
- Existing Sanitary Sewer Pipeline
- Additional Paved Area

N

0 175 350

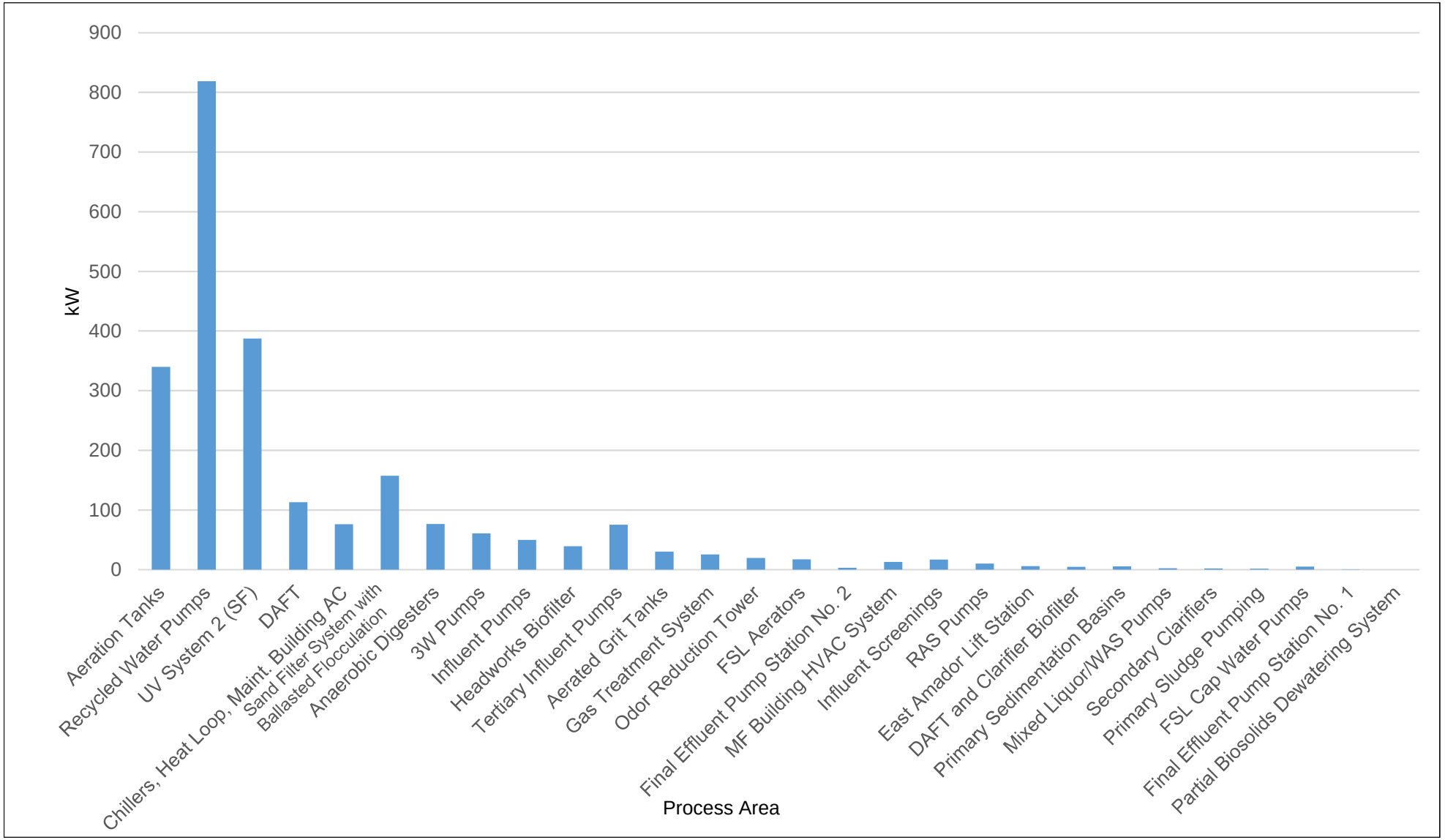
Scale in Feet

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Figure 9-5

Facilities Included in the 2035 Planned and 2035 Potential Energy Balance Scenarios

Dublin San Ramon Services District
Wastewater Treatment Plan
Master Plan

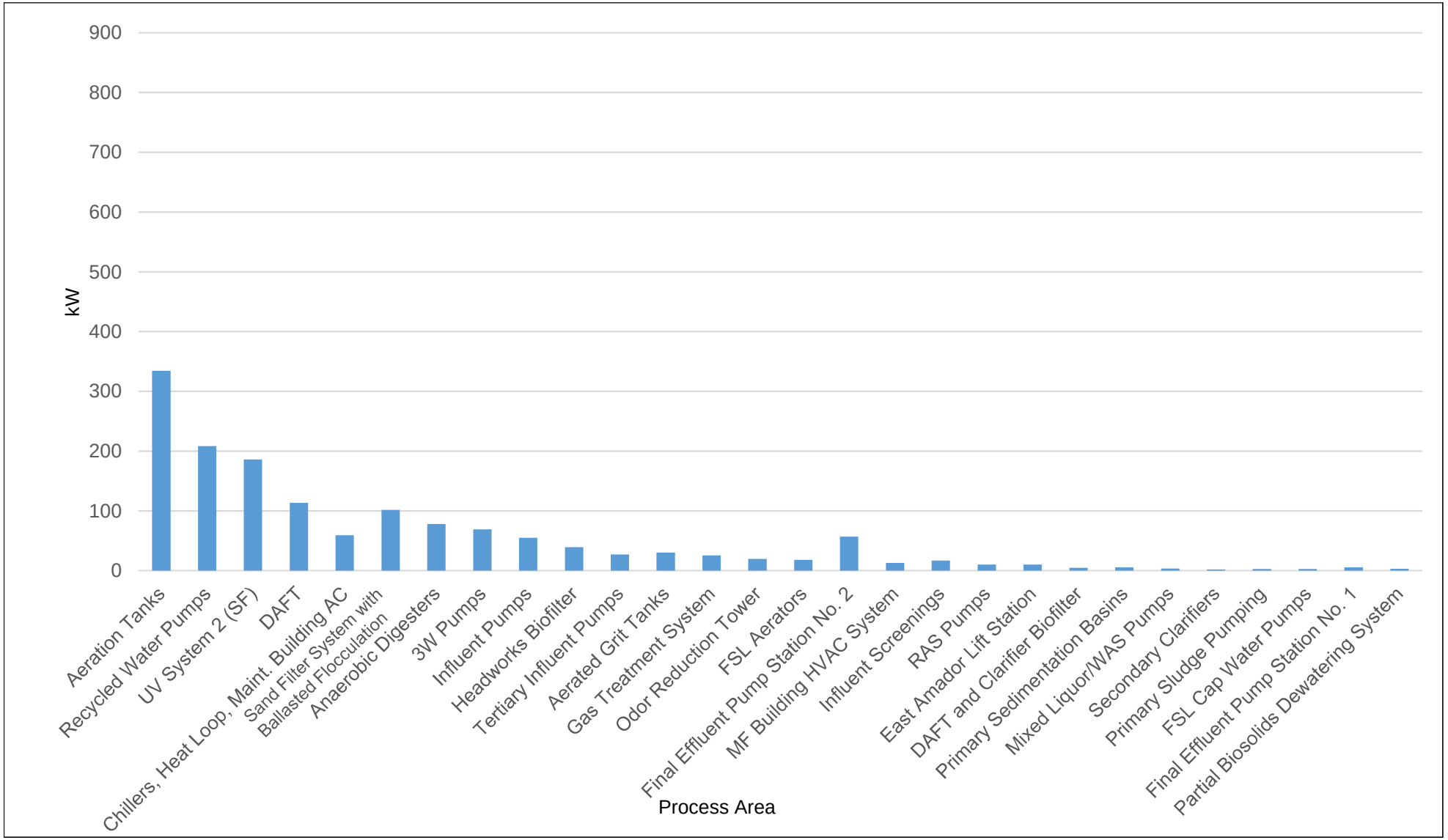


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Figure 9-6

Planned 2035 Power Demand - Summer

Dubin San Ramon Services District
Wastewater Treatment Plant
Master Plan

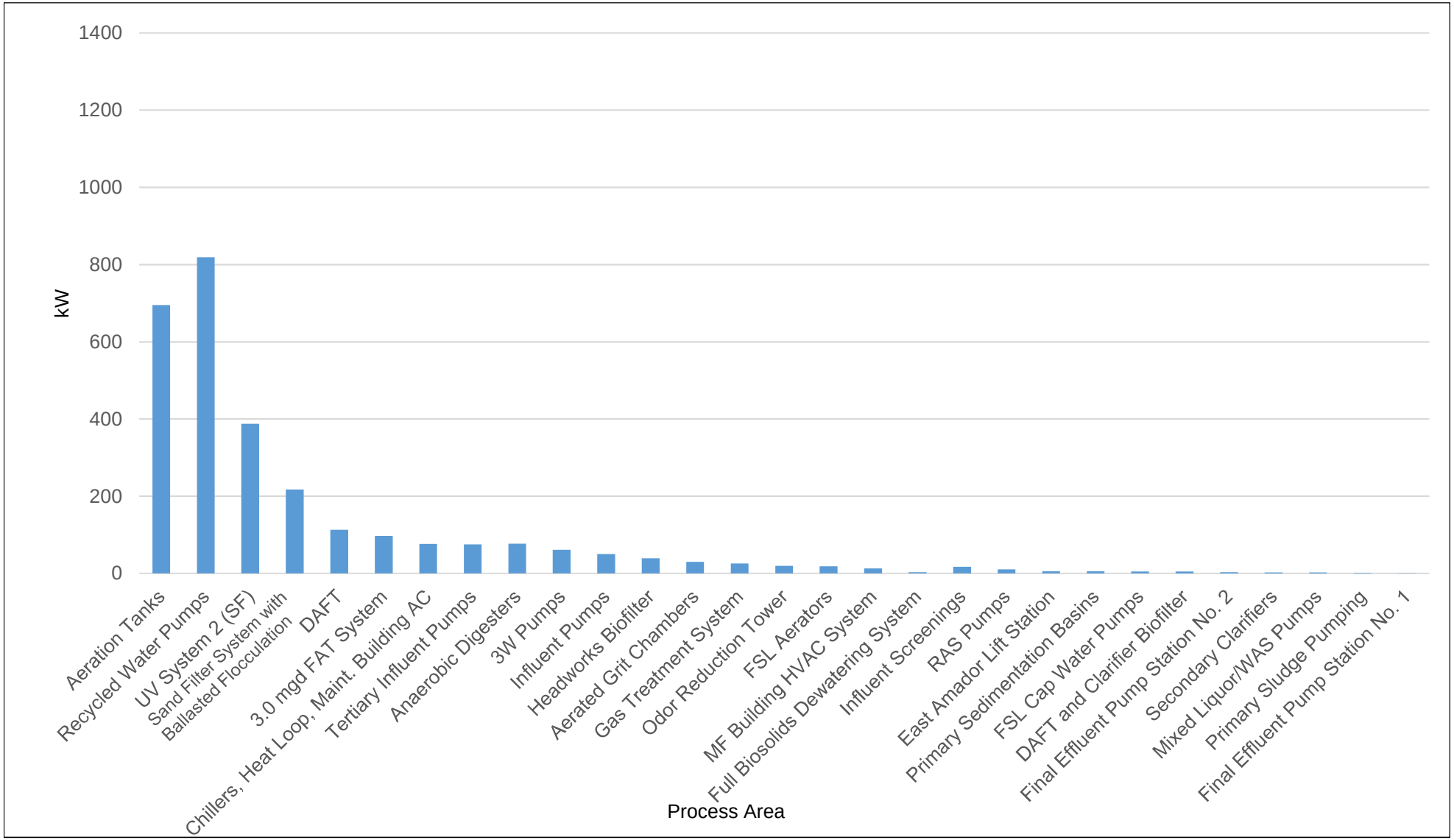


WEST YOST ASSOCIATES

Figure 9-7

Planned 2035 Power Demand - Winter

Dubin San Ramon Services District
Wastewater Treatment Plant
Master Plan

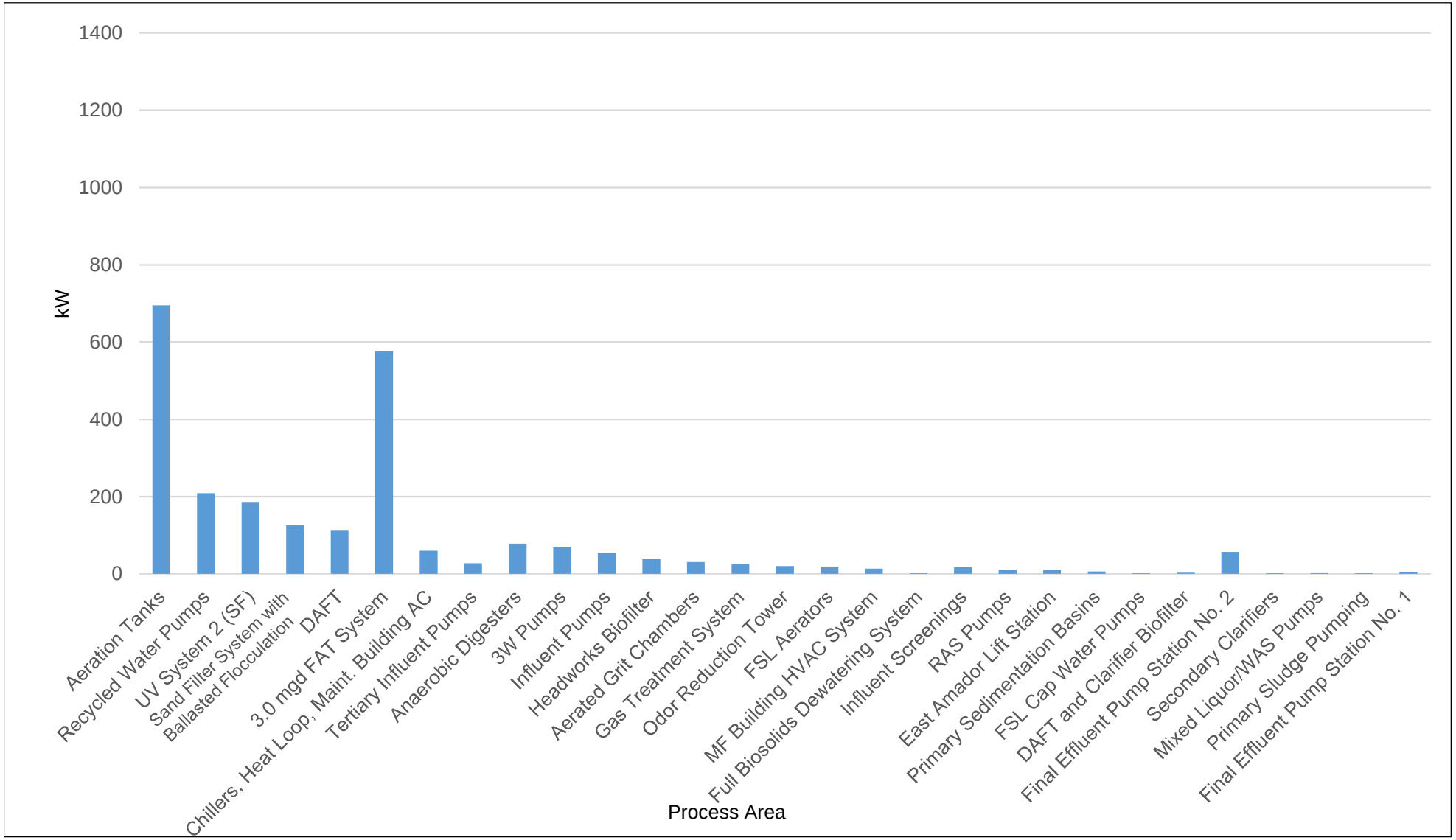


WEST YOST ASSOCIATES

Figure 9-8

Potential 2035 Power Demand - Summer

Dubin San Ramon Services District
Wastewater Treatment Plant
Master Plan

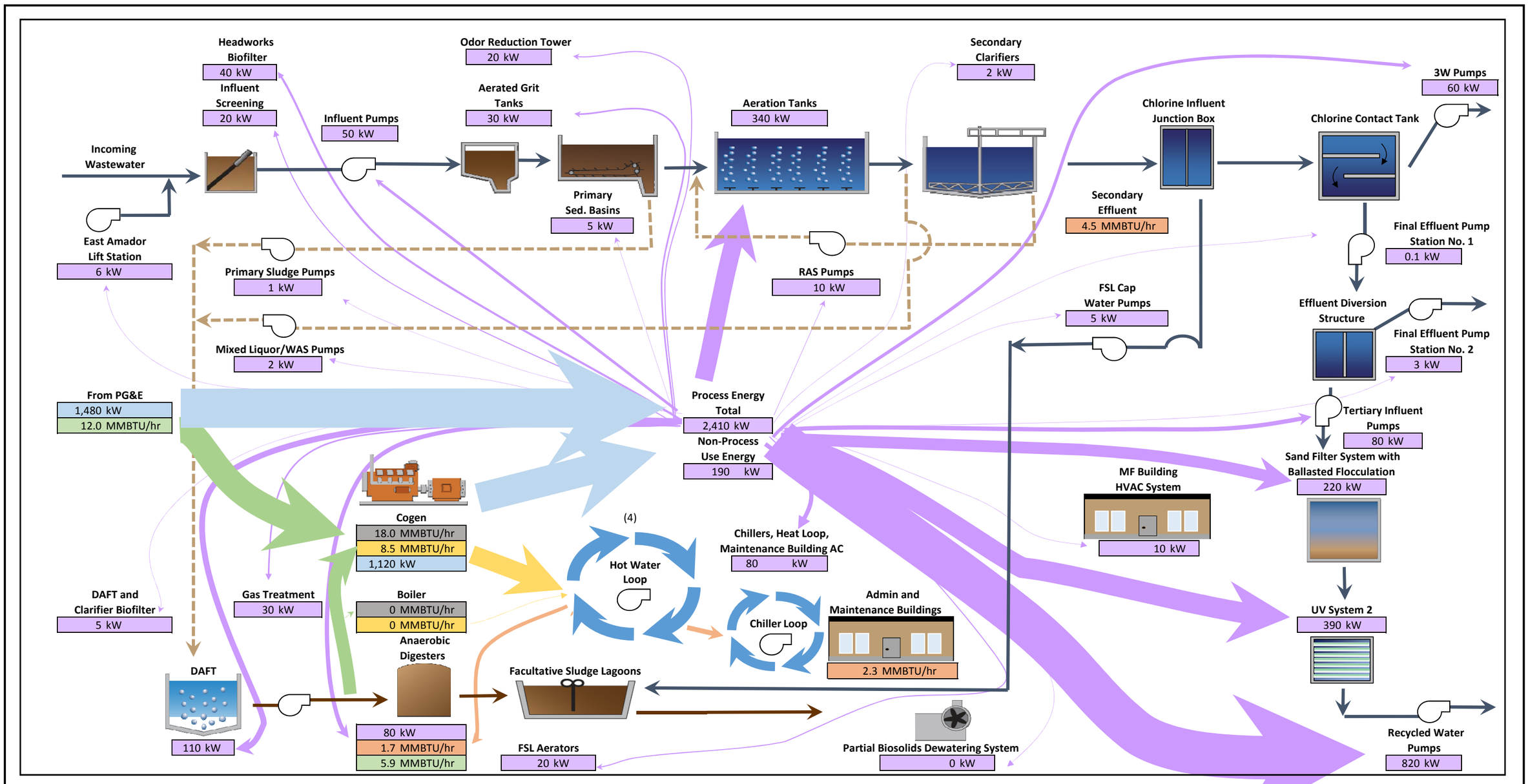


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Figure 9-9

Potential 2035 Power Demand - Winter

Dubin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Symbology

1,000 Power Supply	Power Supply
1,000 Power Demand	Power Demand
1,000 Gas Supply	Gas Supply
1,000 Gas Demand	Heat Supply
1,000 Heat Supply	Heat Output
1,000 Heat Output	Water Loop

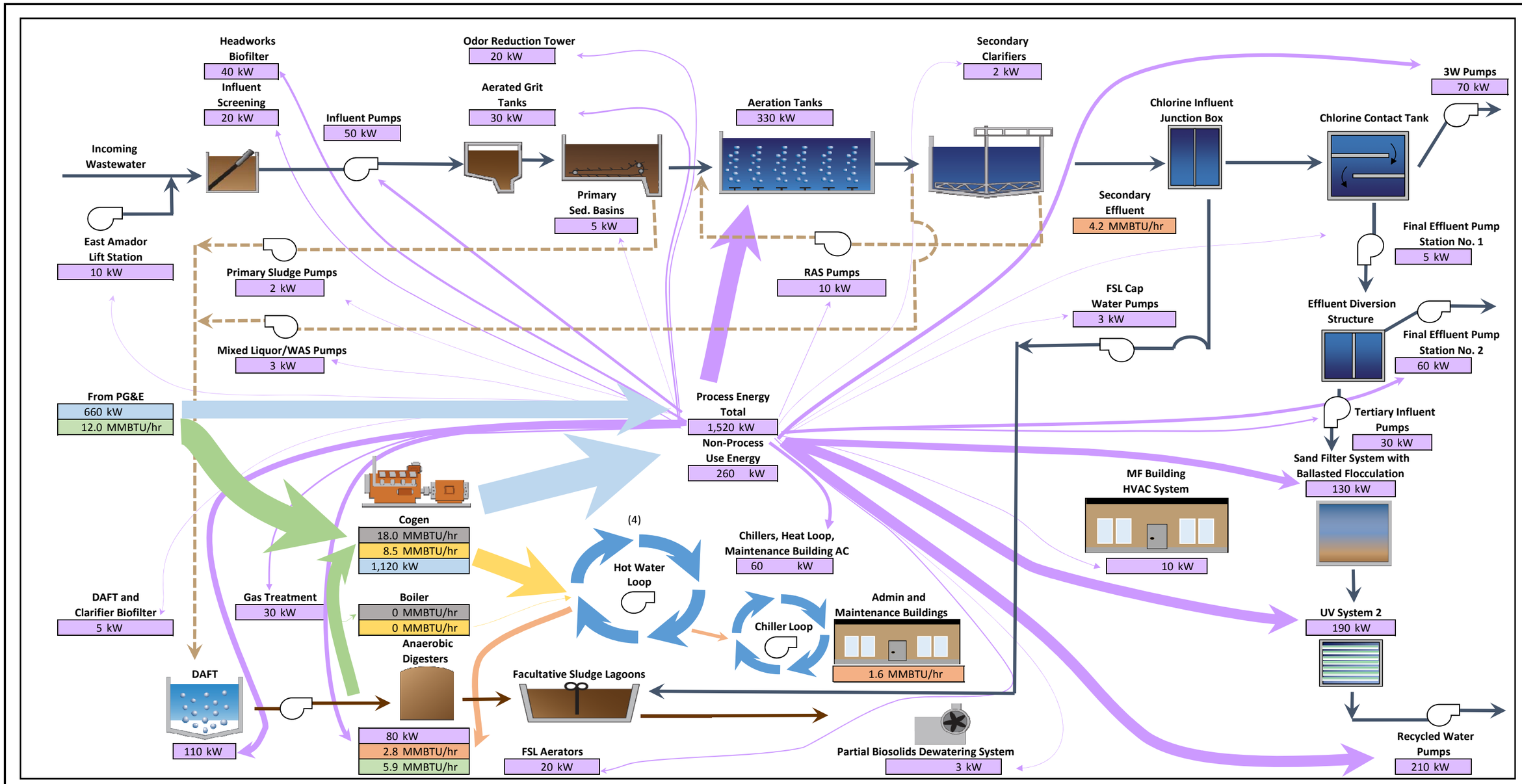
Notes:
1. Digester heat production calculated assuming an increase in influent flow to 14.0 mgd.
2. Digester heat demand based on six month average heat loss.
3. Power delivered to FSL Aerators separately and is not included in electricity total.
4. Hot Water Loop discharges 4.5 MMBTU/hr of waste heat to the Secondary Effluent.

WEST YOST ASSOCIATES Consulting Engineers

Figure 9-10

Planned 2035 Facilities - Summer (May - October)

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Symbology

1,000 Power Supply	Power Supply
1,000 Power Demand	Power Demand
1,000 Gas Supply	Gas Supply
1,000 Gas Demand	Heat Supply
1,000 Heat Supply	Heat Output
1,000 Heat Ouput	Water Loop

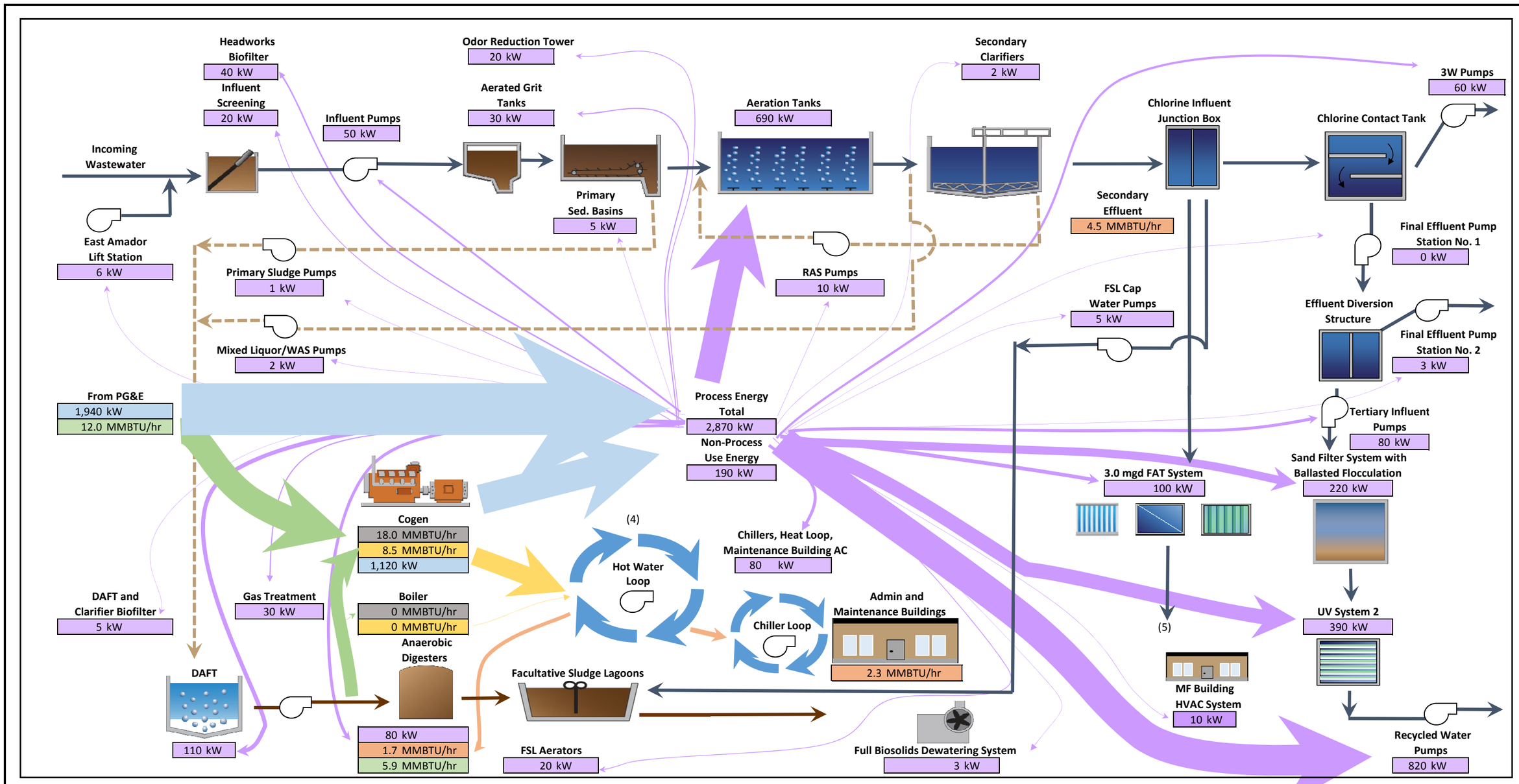
Notes:
1. Digester heat production calculated assuming an increase in influent flow to 14.0 mgd.
2. Digester heat demand based on six month average heat loss.
3. Power delivered to FSL Aerators separately and is not included in electricity total.
4. Hot Water Loop discharges 4.2 MMBTU/hr of waste heat to the Secondary Effluent.

WEST YOST ASSOCIATES
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Figure 9-11

Planned 2035 Facilities - Winter (November - April)

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Symbology

1,000 Power Supply	Power Supply
1,000 Power Demand	Power Demand
1,000 Gas Supply	Gas Supply
1,000 Gas Demand	Heat Supply
1,000 Heat Supply	Heat Output
1,000 Heat Output	Water Loop

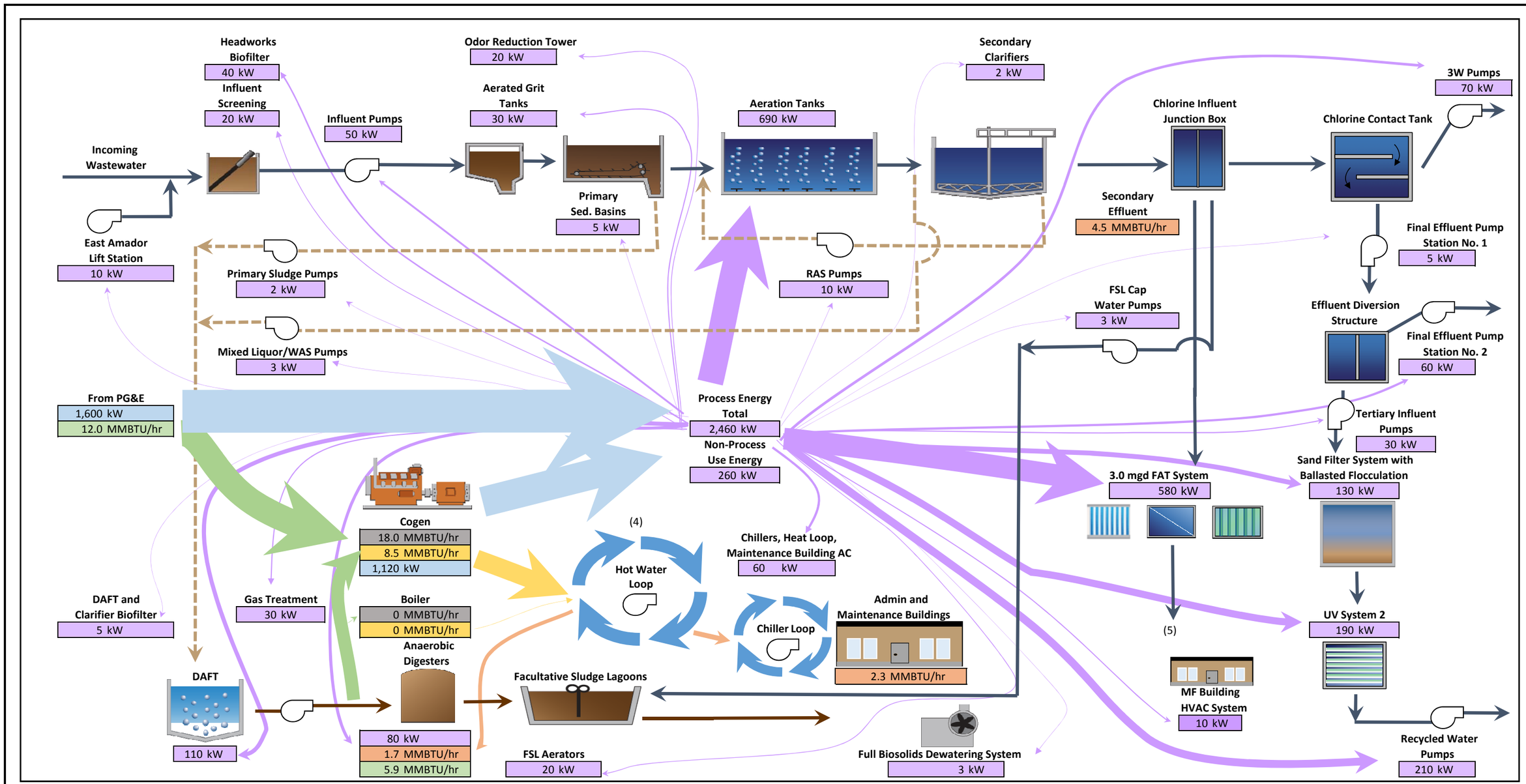
Notes:
1. Digester heat production calculated assuming an increase in influent flow to 14.0 mgd.
2. Digester heat demand based on six month average heat loss.
3. Power delivered to FSL Aerators separately and is not included in electricity total.
4. Hot Water Loop discharges 4.5 MMBTU/hr of waste heat to the Secondary Effluent.
5. Discharge of FAT system unknown, pumps not included in energy balance.

WEST YOST ASSOCIATES Consulting Engineers

Figure 9-12

Potential 2035 Facilities - Summer (May - October)

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Symbology

1,000 Power Supply	Power Supply
1,000 Power Demand	Power Demand
1,000 Gas Supply	Gas Supply
1,000 Gas Demand	Heat Supply
1,000 Heat Supply	Heat Output
1,000 Heat Output	Water Loop

Notes:
1. Digester heat production calculated assuming an increase in influent flow to 14.0 mgd.
2. Digester heat demand based on six month average heat loss.
3. Power delivered to FSL Aerators separately and is not included in electricity total.
4. Hot Water Loop discharges 4.5 MMBTU/hr of waste heat to the Secondary Effluent.
5. Discharge of FAT system unknown, pumps not included in energy balance.

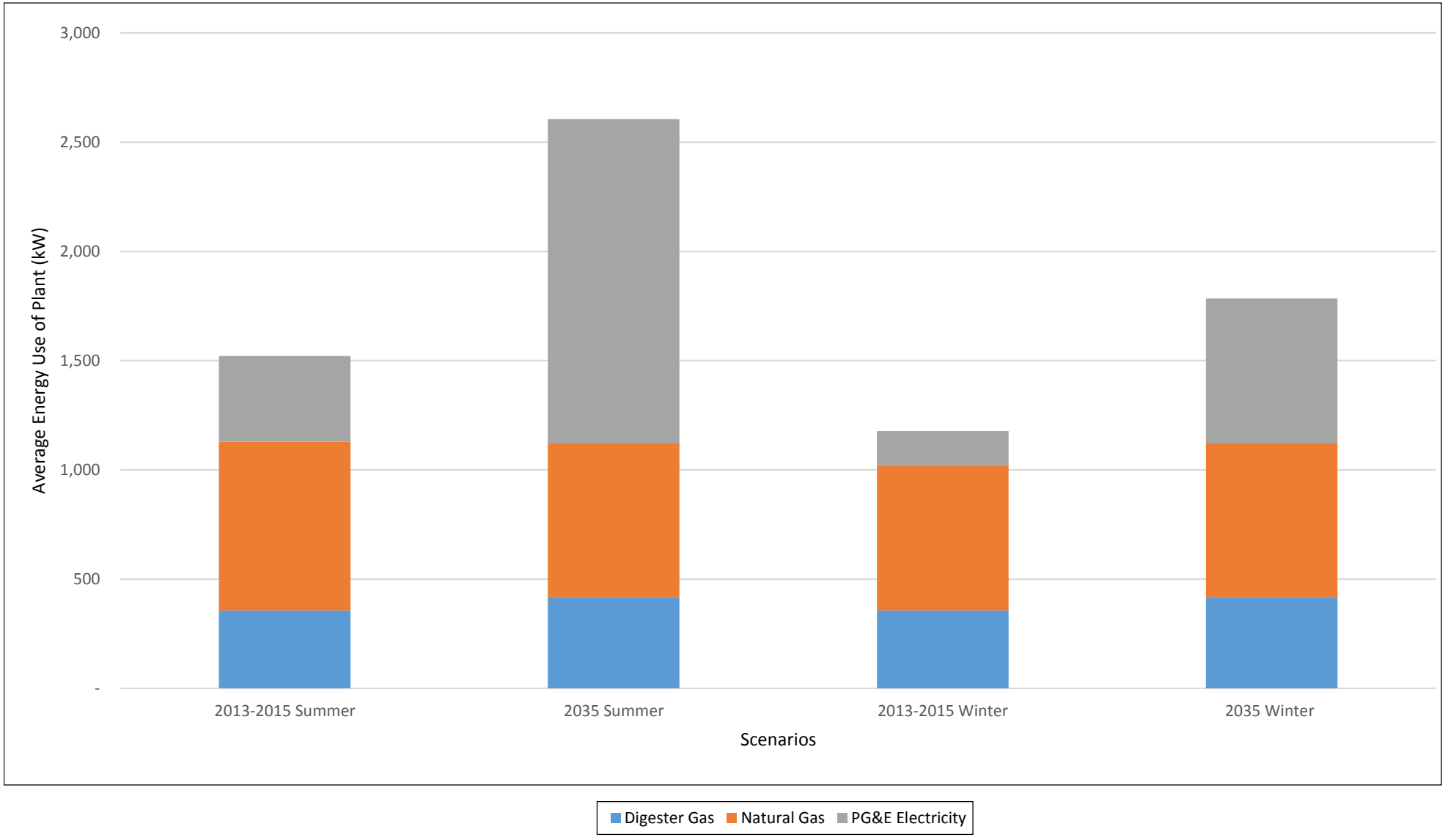
WEST YOST ASSOCIATES Consulting Engineers

Figure 9-13

Potential 2035 Facilities - Winter (November - April)

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Notes:

1. The 2013-2015 scenarios represents a 49% solids capture.
2. The 2035 scenarios represents 65% solids capture.
3. Cogeneration design capacity is 1,400 kW. Actual capacity is estimated at 80% of their design capacity, to acount for maintenance and downtime.
4. From September 2013 to September 2015 the cogeneration system generated 1,130 kW, slightly more than the estimated capacity.

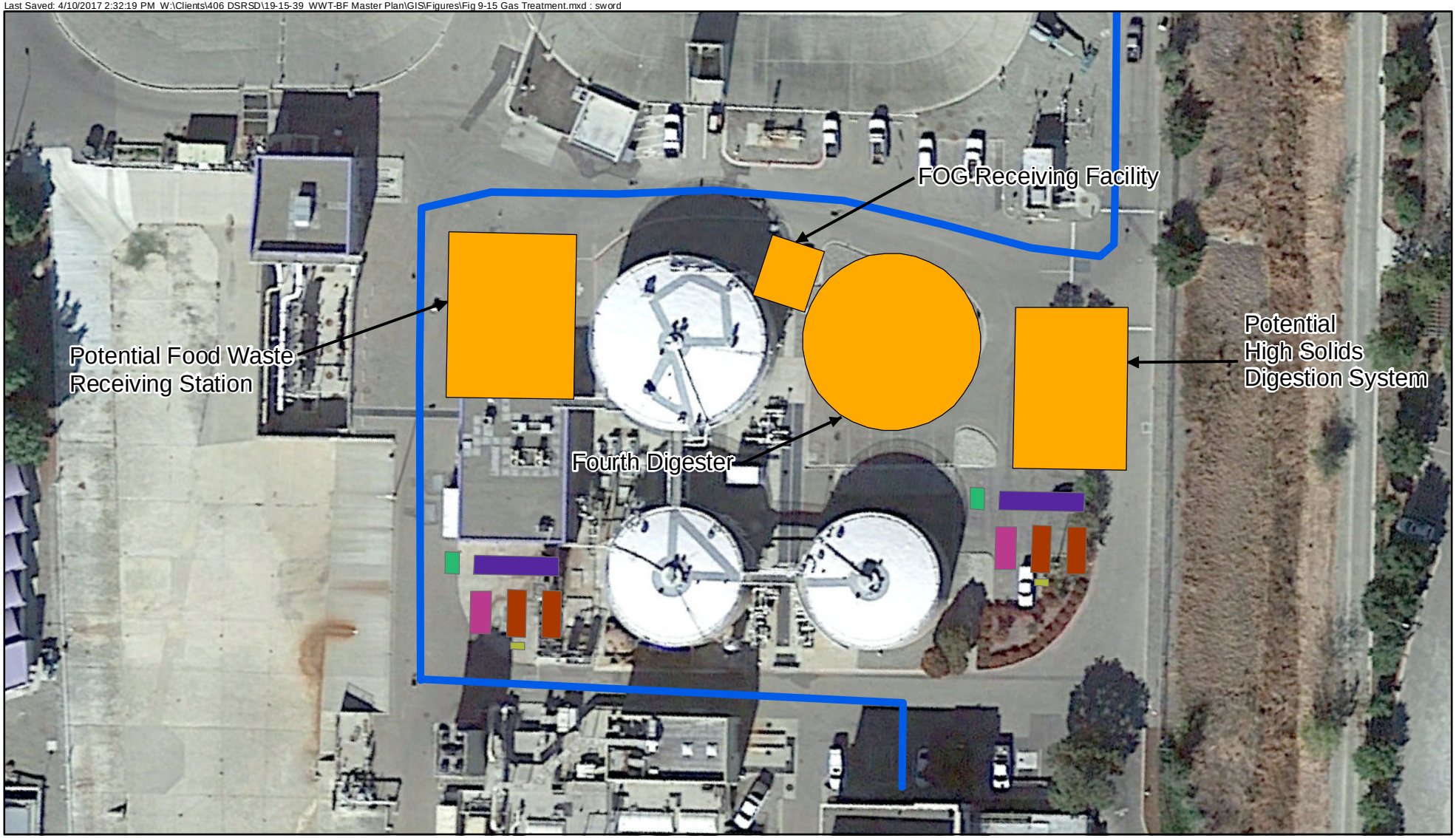
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Figure 9-14

Current Available Energy Sources for Current and Planned 2035 Facilities

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

Last Saved: 4/10/2017 2:32:19 PM W:\Clients\406 DSRSD\19-15-39 WWT-BF Master Plan\GIS\Figures\Fig 9-15 Gas Treatment.mxd : sword



Symbology

- Route for Administration and Operations Building Access
- Facilities to Increase Digester Gas Production

Gas Treatment System

- Separator
- Sulfatreat
- Compressor
- Refridgerated Dryer
- Siloxane Removal

0 30 60

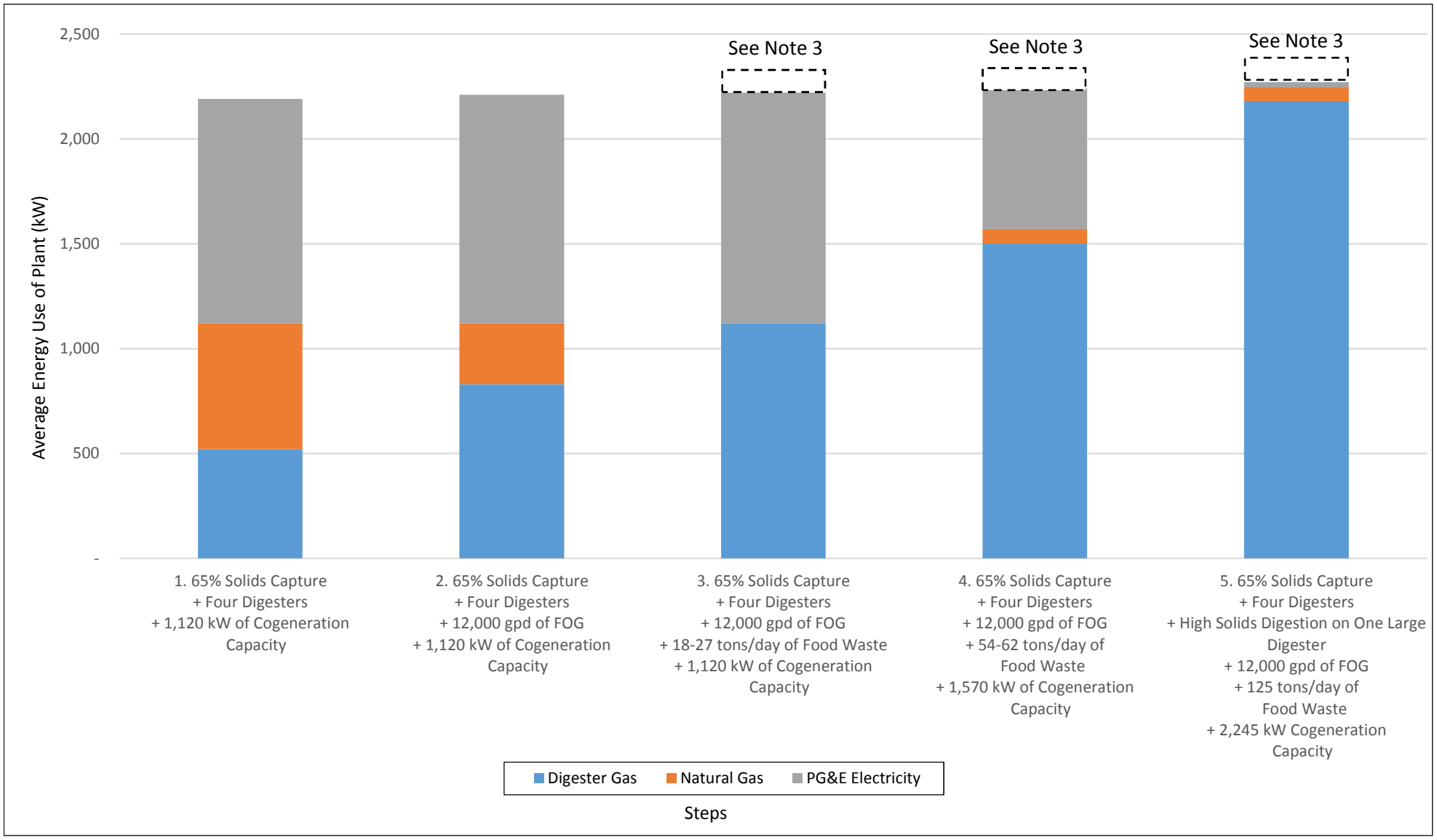
Scale in Feet

WEST YOST ASSOCIATES

Figure 9-15

Gas Treatment System Schematic

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan



Notes:
1 One truck load is equivalent to 3,000 gallons of FOG.
2. One truck load is equivalent to 20.8 tons of food waste.
3. Addition of new facilities to allow for increased onsite renewable energy generation will also increase plant energy demands.
4. Cogeneration capacity was assumed to be 80% and 90% of design capacity for existing and new cogeneration engines, respectively.

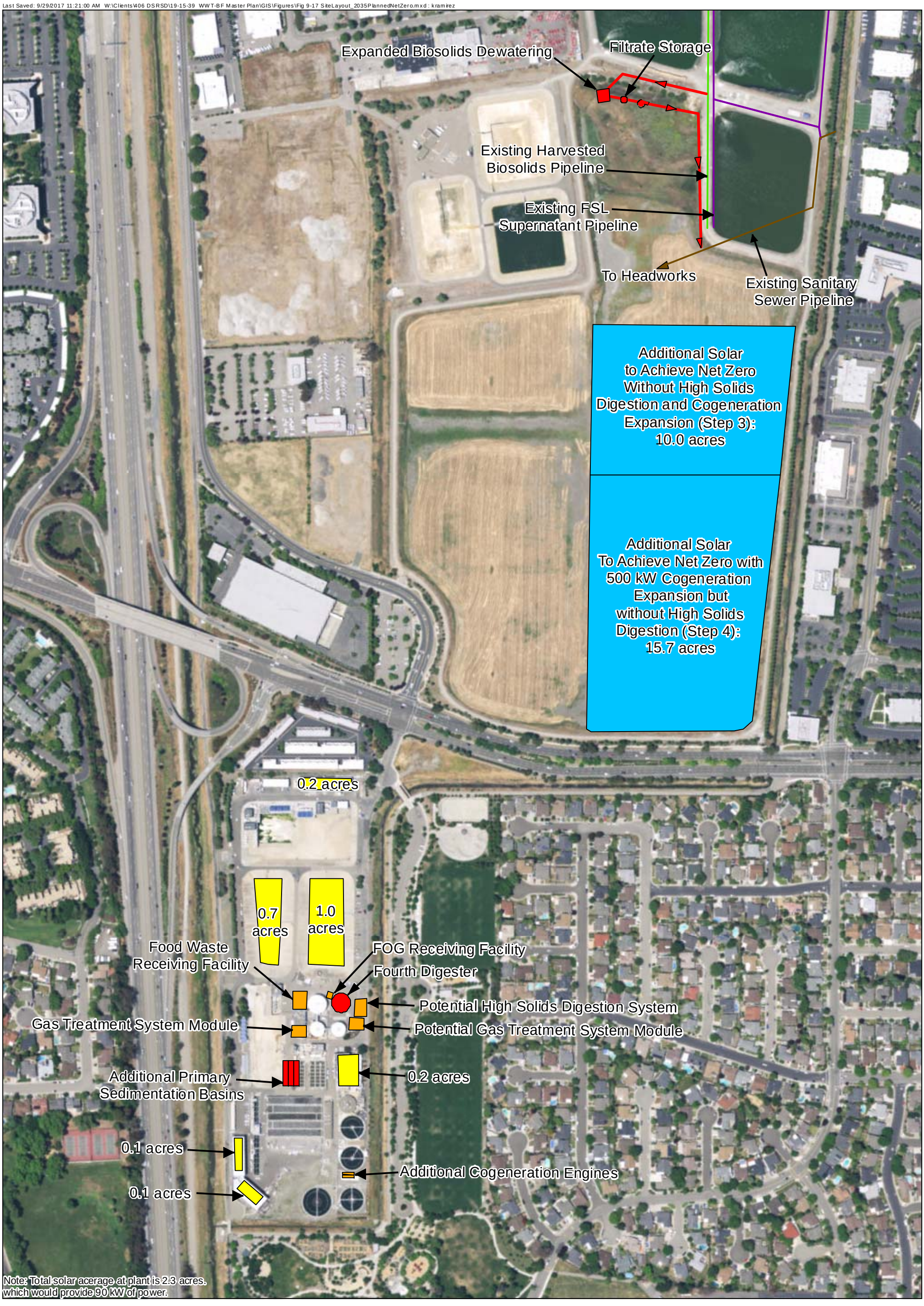
WEST YOST ASSOCIATES

Figure 9-16

Potential Energy Supplies for Meeting Planned 2035 Annual Demand

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- 2035 Planned Facilities
- Potential Facilities to Increase Onsite Digester Gas Cogeneration Production
- Solar Panels Needed for Net Zero with High Solids Digestion and Cogeneration Expansion (Step 5)
- Potential Additional Solar Panels for Net Zero without High Solids Digestion and Cogeneration Expansion

N

0 175 350

Scale in Feet

WEST YOST ASSOCIATES

Figure 9-17

Site Layout Facilities Needed for Net Zero, Planned 2035

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

CHAPTER 10
Recommended Program

This Chapter presents a facilities plan for the WWTP based on the information that was available during the development of this Master Plan. Also presented is a recommended program for evaluating and implementing the projects identified in the facilities plan. This program is presented as three sequential Implementation Plans that are correlated to the District's fiscal year (FY) planning process. The major topics addressed are as follows:

- Facilities Plan
- Near-Term Implementation Plan (FY 17/18 to FY 20/21)
- Mid-Term Implementation Plan (FY 21/22 to FY 25/26)
- Long-Term Implementation Plan (FY 26/27 to FY 35/36)
- Capital Improvement Program (CIP) Summary

10.1 FACILITIES PLAN

This section presents a summary of recommended WWTP improvements identified in this Master Plan, and provides the rationale behind their selection. For some of the improvements that are needed, a specific facility recommendation cannot be made based on currently available information. In these instances, the facility limitations and current uncertainties are identified, and the assumptions that were applied to provide a reasonably conservative basis for estimating the potential costs and facilities layout are presented. Additional discussion of the future evaluations needed to better define certain facilities is provided in the Implementation Strategy sections of this Chapter.

The information presented in this section is organized in accordance with the Chapters and Technical Memoranda (included as appendices) presented in this Master Plan, as follows:

- Liquid Treatment (Chapter 6)
- Potable Reuse (Chapter 7)
- Solids Treatment (Chapter 8)
- Energy Management (Chapter 9)
- Odor Control (Appendix D, as referenced in Chapter 3)

Figure 10-1 provides a conceptual layout of all the facilities that are needed, or potentially needed, at the main WWTP site by 2035. Figure 10-2 provides a conceptual layout of facilities that are needed, or potentially needed, at the DLD site by 2035. The layouts shown approximate the size and location for each recommended or potential future improvement. New, major liquid and solid conveyance pipelines that would be needed to connect the various facilities into the WWTP treatment trains are also shown on these figures. Figure 10-3 provides a recommended implementation schedule for the facilities planned or anticipated over the 20-year planning period.

10.1.1 Liquid Treatment

There are four major liquid treatment system projects discussed in Chapter 6, as follows:

- WWTP Hydraulic Improvements
- Primary Treatment Improvements/Expansion
- Secondary Clarification Improvements/Expansion
- Nutrient Removal Improvements

Recommendations regarding implementation of each of these projects is provided below.

10.1.1.1 WWTP Hydraulic Improvements

As detailed in Chapter 6, there are hydraulic constraints in the WWTP that limit flows downstream of the secondary clarifiers. To address this issue, the following improvements are required:

- Remove the wall between the CCT influent channel and CCT
- Remove the weir in the chlorine junction box

Implementing this project will allow the District to increase the conveyance capacity downstream of the secondary clarifiers to 46.1 mgd, just under the projected 2035 influent PHWWF of 46.7 mgd. Should peak flows exceed 46.1 mgd, flow equalization would be needed.

As discussed in the Storage Needs Assessment TM (Appendix C) and Chapter 6, the District also does not currently have the means to reliably control and meter the primary effluent flows that are diverted. As flows into the WWTP increase, this operational control will be critical to maintaining adequate storage capacity. The recommended improvements include a flow meter and a SCADA-controlled modulating control valve on the 24-inch diameter primary effluent flow diversion pipeline.

Both of the improvements discussed above should be straightforward to implement and will improve plant peak flow operations. Therefore, it is recommended that both projects be completed early in the near-term implementation period.

10.1.1.2 Primary Treatment Improvements/Expansion

As discussed in Chapter 6, the primary sedimentation basins are not adequately sized to process the District's wastewater based on typical design standards. Chapter 6 identifies construction of three additional sedimentation basins as a reasonable approach for addressing the identified capacity deficiencies because it represents a conservative expansion alternative in terms of construction cost and space requirements. Alternatives to expanding the number of sedimentation basins are as follows:

- Construct fewer than three primary sedimentation basins, and provide performance enhancing hydraulic baffling on existing and/or new primary sedimentation basins
- Construct fewer than three primary sedimentation basins, and provide chemically enhanced primary treatment (CEPT) with metal salt addition
- Construct fine screens in lieu of new primary sedimentation basins

Advantages and disadvantages of each approach to addressing the primary treatment capacity issues, including adding more sedimentation basins, are presented in Table 10-1.

Table 10-1. Summary of Primary Treatment Alternatives to Increase Capacity

Concept	Pros	Cons
Construct three new primary sedimentation basins	• Operator familiarity • Reliable/well defined performance	• Larger footprint • Impacts on available storage • Higher construction costs
Construct less than three primaries and add baffles to improve performance	• Proven approach for enhancing solids capture • Little to no impact on operations • Reduces the number of new primaries needed • Relatively straightforward testing • Straightforward installation and implementation	• Requires extensive modeling to quantify benefits and verify how many primaries would be needed
Construct less than three primaries and operate system as CEPT	• Proven approach at enhancing solids capture • Additional benefit of odor control if iron based metal salt used • Reduces the number of new primaries needed	• Additional chemicals to handle • Chemical unit costs are dependent on variables outside the plant's control • Potential impact of chemicals on the primary structure and equipment and integrity
Maintain existing primaries and add fine screens	• Compact footprint • Solids capture may exceed conventional primary sedimentation basin performance • Ability to control the solids thickness to the upper limit of pumping (about 8 percent solids)	• Emerging technology with not many full-scale installations • Might require multiple units to operate and maintain • Concerns over blinding of the screens and the ability to handle peak flows

The District recently solicited proposals for the primary sedimentation basin improvements project, which asked proposers to consider the options detailed in Table 10-1 and develop a recommended strategy. Based on this process, the District decided to proceed with implementing the option of enhancing hydraulic baffling on the existing primary sedimentation basins, thus allowing for the construction of less than three primaries. The specific configuration and costs for the primary sedimentation improvements project will be defined under the forthcoming preliminary design efforts. Therefore, for purposes of space and budget planning in this Master Plan, the approach of adding three primaries is assumed.

The primary expansion project is expected to be completed in early 2020.

10.1.1.3 Secondary Clarification Improvements/Expansion

As discussed in Chapter 6, the secondary clarifiers are not adequately sized to process the District's wastewater at the Master Plan design conditions given the existing settleability of the activated sludge leaving the aeration basins. Chapter 6 presented an evaluation of the following alternatives for addressing secondary clarifier capacity limitations:

1. Addition of a fifth secondary clarifier
2. Alum addition to the FSL supernatant to address struvite concerns and facilitate year-round anaerobic selector operation of the aeration basins
3. Implementation of side-stream phosphorus recovery to facilitate year-round anaerobic selector operation of the aeration basins
4. Implementation of a parallel stream MBR process

The third alternative, which involves side-stream phosphorus recovery, was ruled out for several reasons detailed in Chapter 6, but primarily because phosphorus recovery has only been demonstrated to be economically attractive for much larger facilities and improvements in economics are not expected to improve the timeline needed to address the secondary clarification issues. However, it is recommended that the District remain apprised of advances in phosphorus recovery technologies and look for opportunities to implement this option if conditions become more favorable in the future. Additional discussion regarding this recommendation is provided in the Long-Term Implementation Plan section.

Costs for the remaining three alternatives were evaluated, and alum addition was found to be the least expensive alternative by a significant margin (see Table 6-9). The subjective rankings of the alum addition and fifth secondary clarifier alternatives were not significantly different, but alum addition had a slightly higher ranking than the addition of a fifth secondary clarifier (see Table 6-12). The higher subjective ranking for the alum addition alternative was primarily related to the fact that this facility would be located at the FSL site and provides the greatest flexibility for implementing future treatment options. The MBR alternative was the lowest ranked, primarily due to the fact that is an entirely new biological treatment process that must be operated parallel to the existing system and that it requires space within the equalization facilities for construction.

Chapter 6 also discusses that the ultimate selection of the preferred attentive should consider the potential benefits of the parallel stream MBR approach with respect to a potential future potable reuse project. This evaluation was presented in Chapter 7. As shown in Table 7-7, a parallel stream MBR is not a cost-effective pretreatment strategy for potable reuse. Indeed, the parallel stream MBR concept would have the most expensive lifecycle cost of the potable reuse pretreatment alternatives evaluated; therefore, a parallel stream MBR is not recommended.

Based on the information presented in both Chapters 6 and 7 and further discussions with District staff, the recommended approach for addressing the secondary clarifier deficiency is alum addition to the FSL return stream. This is primarily due to the significantly lower cost of this alternative and that there is a potential that future nitrogen removal may not be required at the WWTP. In addition, alum addition does allow for a more straightforward operation of the WWTP. Specifically, as discussed in Chapter 6, adding a metal salt (i.e., alum) to the FSL return stream would allow the District to operate the existing aeration basins with an anaerobic selector year-round. As shown on Figure 10-2, the preferred location identified for the alum addition facilities is at the DLD site, near the recently installed FSL return stream flow meter.

The existing secondary clarifiers should have adequate capacity for another five to ten years. However, the recommended alum addition project should be straightforward to implement and will improve plant operations during the winter months, while providing for some energy savings. Therefore, it is recommended that the alum addition project be completed during in near-term implementation period.

10.1.1.4 Nutrient Removal Improvements

As discussed in Chapter 6, the specific future nutrient removal requirements that may be applied to the WWTP are uncertain. The need for nutrient removal could be driven either by new limitations for San Francisco Bay discharges, or by the need to meet potable reuse conditioning goals. For purposes of defining the facilities layout and costs in this Master Plan, it has been assumed that the nutrient removal facilities' construction will be driven by a new Nutrients Watershed Permit that requires compliance with the BACWA Level 2 treatment requirements defined in Chapter 6.

As discussed in Chapter 6, it is assumed that the following WWTP facility improvements would be necessary to meet these BACWA Level 2 requirements:

- Construct three additional aeration basins
- Modify all the existing and new basins to provide three zones:
  - An initial anaerobic zone for biological phosphorus removal
  - A secondary anoxic zone to provide for denitrification
  - A third aerobic zone where nitrification will occur
- Install internal mixed liquor recycle facilities on each basin to pump mixed liquor from the end of the aerated zone to the anoxic zone. These mixed liquor pumping facilities would be sized to convey flow at 2 to 3 times the average daily influent flow rate

- Construct one additional secondary clarifier
- Install two additional blowers

Additional analysis will be necessary to further define the specific facilities that will be necessary. Specifically, there are significant uncertainties with respect to constituents of concern, whether the future nutrient limitations will be applied as concentration-based or load-based, and whether limits will be applied seasonally or year-round. Each of these factors has the potential to significantly influence the type and/or configuration of the required facilities. In addition, the District should consider what new technologies have emerged that could be cost-effectively incorporated into the design to reduce costs and/or improve efficiency. A recommended plan for these evaluation activities is provided later in this Chapter.

As described in Chapter 4, the Nutrients Watershed Permit has a five-year permit cycle, and the current permit will be renewed in 2019. However, as indicated in Chapter 6, the 2019 Nutrients Watershed Permit is not expected to include new limitations. Therefore, if nutrient limitations are applied, they will not be applied before the 2024 Nutrients Watershed Permit is adopted. There will be at least a five to ten-year compliance timeline for the Bay Area dischargers to upgrade their treatment plants to meet the 2024 Nutrients Watershed Permit requirements, thus putting final compliance in 2029 at the earliest, but more likely closer to 2034.

It is likely that there will be a multi-year evaluation period between compliance with the 2024 requirements (between 2029 and 2034) and when new, more stringent requirements are applied. Thus, it is not expected that more stringent requirements than those adopted in 2024 will be applied within the 20-year planning horizon of this Master Plan. For this reason, the costs for achieving levels of nutrient removal higher than BACWA Level 2 are not included in this Master Plan. However, the layout of potential BACWA Level 3 facilities are shown on Figure 10-1 to demonstrate the space constraints that should be considered in the long-term.

10.1.2 Potable Reuse

The District is interested in developing a potable reuse project in the next ten-year period. In support of defining the preferred strategy toward implementing this goal, this Master Plan presents options for implementing a potable reuse treatment project at the WWTP. Three potential potable reuse projects at the WWTP site are discussed in Chapter 7, as follows:

- Rehabilitation of the CWR MF/UV facilities to allow for potable reuse
- Rehabilitation of the CWR MF/UV facilities to allow for raw water or raw water or treated water augmentation
- Construction of expanded potable reuse facilities to allow for raw water or treated water augmentation and minimize discharges to the San Francisco Bay

Recommendations regarding implementation of each of these projects, including incorporation of the findings from a regional potable reuse study being completed under a separate effort, are provided below.

10.1.2.1 Rehabilitation of CWR Facilities for Potable Reuse

In parallel to this Master Plan, the District is participating in a separate study to evaluate potable reuse on a region-wide basis. As further discussed in the Implementation Strategy sections of this Chapter, the District will need to consider the results from both this Master Plan and the current regional study before confirming a preferred approach to potable reuse.

If a regional study is not pursued, a logical step toward implementation of a potable reuse project at the WWTP would be to rehabilitate the CWR facilities. Thus, for purposes of defining a facilities layout and costs in this Master Plan, it is assumed that rehabilitation of the CWR facilities will be the first potable reuse project implemented by the District. Moreover, it is assumed for this Master Plan that the goal of the initial CWR rehabilitation project would be to provide the facilities needed to produce approximately 3.0 mgd of water that meets potable reuse water quality standards for reservoir water augmentation. This assumption is based on an understanding from District staff that the regional study has identified augmentation of the Chain of Lakes which is planned to supply the Del Valle Water Treatment Plant (WTP) as a likely near-term end use for potable water produced in the tri-valley area.

Current regulations require that treatment beyond the FAT process is needed for reservoir water augmentation. As an example, the Pure Water San Diego project, which involves reservoir water augmentation under conditions similar to those that would occur at the Chain of Lakes, includes an ozonation with BAC pretreatment process, followed by MF, RO, and then UV/AOP (City of San Diego, 2017). To be consistent with the San Diego project, the facilities represented under the Ozonation plus BAC pretreatment alternative discussed in Chapter 7 are assumed to be constructed for the initial phase project.

As discussed further in the Near-Term Evaluations section of this Master Plan, the District should evaluate the desired treatment approach for the CWR rehabilitation project prior to selecting the preferred treatment train. Factors that should be considered are: 1) the intended use of the CWR water supply, which will be informed from the regional potable reuse study being completed; 2) emerging potable reuse regulations and guidelines, particularly with respect to what treatment processes will result in log-removal credits, and 3) nitrogen removal requirements for discharges to the San Francisco Bay, where full NDN treatment could provide pretreatment benefits for a potable reuse project.

As shown on Figure 10-3, the District has several major projects that will need to be implemented at the WWTP in the next few years to address treatment capacity issues. To better distribute the strain on District staff and budget resources, it is recommended that the District postpone implementation of a CWR facilities rehabilitation project until the bulk of the other near-term projects are nearing completion, approximately FY 20/21 or FY 21/22. Waiting a few more years prior to implementation also allows time for further assessment of a potential regional project and for potable reuse and San Francisco Bay nutrient regulations to be further defined. It is recommended that the District continue to evaluate potable reuse throughout the near-term period, so that design and construction can begin near the beginning of the mid-term period, with a goal to achieve project completion by 2027.

The project costs presented in the Master Plan for the CWR facilities rehabilitation are also considered indicative of the District's share of costs associated with an initial phase of a region-wide potable reuse project. However, if the preferred potable reuse strategy is a regional approach, then the implementation timeline could be contingent on factors that extend beyond the District's control – particularly if the preferred strategy involves construction of the potable reuse treatment facilities at another location. It is possible, however, that the preferred potable reuse strategy will be a regional approach that involves treatment at the District's WWTP. If the regional solution includes construction of treatment facilities at the District's WWTP site, it is suggested that the District consider other project needs at the WWTP when defining the implementation timeline to distribute the District's staff and budget resources more evenly.

10.1.2.2 Rehabilitation of CWR Facilities for Treated Water Augmentation

It is expected that a second phase of potable reuse facilities development will occur within the planning horizon of this Master Plan. However, the timing, costs, and framework for this second phase will depend on several factors that cannot be defined at this time. For example, if a CWR rehabilitation project is implemented as a first phase, the second phase could involve bringing in additional water supplies to expand the period of operations, expand the treatment capacity, or upgrade the treatment system to provide for treated water augmentation. The project could also involve investing in a regional program that is not located at the WWTP site.

For purposes of providing a facility cost and layout in this Master Plan, it has been assumed that the second phase would involve upgrading the CWR facilities to provide for treated water augmentation. At this time, it is expected that the costs identified herein are considered indicative of the District's likely investment in a second phase of potable reuse, including the District's share of costs associated with the second phase of a region-wide potable reuse project.

As discussed in Chapter 7, the specific facilities that would be required beyond the ozone plus BAC to allow for treated water augmentation are currently not known. However, based on information that is available, it has been assumed that a chlorine disinfection contactor and treated water storage tanks would be required. In addition, facilities would be needed for full NDN – additional aeration basins, an additional secondary clarifier, and a mixed, liquor return channel.

One of the primary goals of phasing the potable reuse facilities would be to provide the District staff with an opportunity to operate a smaller and/or less complex treatment system prior to investing in a more complex program. Therefore, the District should plan for several years of operation of the initial potable reuse treatment facilities prior to initiating a second phase of the potable reuse project. In addition, if an upgrade to allow for treated water augmentation is desired, the District should wait until the regulations for this use are clearly defined in an adopted state policy. Assuming the District initiates the first phase of potable reuse at the beginning of the mid-term period (FY 21/22), the project would likely come online by 2027. Thus, assuming a few years of operation is desirable, the District would be expected to begin implementation of a second phase of the potable reuse system in 2031/2032, with construction estimated to be completed around 2035. This timeline also is expected to provide time for regulations for treated water augmentation be developed and adopted.

10.1.2.3 Expanded Potable Reuse Facilities to Minimize Discharges to the San Francisco Bay

As discussed in Chapter 7, the WWTP is not expected to have an available treated effluent supply for potable reuse during the summer months because most of the treated effluent will need to be directed to current/planned disinfected tertiary recycled water customers. Thus, unless an alternative supply is identified, any potable reuse treatment facility that is constructed at the WWTP will only be operated during part of the year. The potable reuse water production cost is calculated in part by dividing the total capital (fixed) cost of the facilities by the volume of water delivered. Therefore, the production cost for potable water in terms of dollars/gallon-year, or dollars/acre foot-year is significantly higher when water is not produced during some months of the year. This issue was demonstrated in Chapter 7.

Due to elevated production costs for a District-only project, it is unlikely that an investment beyond the rehabilitation of the existing CWR facilities would be justified. Therefore, costs for implementing a larger potable reuse project to eliminate San Francisco Bay discharges outside of a regional project are not included in the Master Plan CIP.

However, the regional study could conclude that a regional potable reuse treatment facilities larger than the 3.0 mgd CWR facilities should be constructed at the DSRSD site. Under this scenario, the District would need to reserve space at the WWTP for the facilities. It is recommended that the site identified in this Master Plan for the expanded onsite potable reuse system near the southwestern corner of the DLD site be used for such a regional system. This location is shown on Figure 10-2.

10.1.3 Solids Treatment

There are four solids treatment system projects discussed in Chapter 8, as follows:

- Anaerobic Digester Expansion
- Biosolids Thickening/Dewatering Improvements to Address DLD Capacity Limitations
- Full Dewatering Expansion
- Potential Supercritical Oxidation Process Implementation

Recommendations regarding implementation of each of these projects is provided below.

10.1.3.1 Anaerobic Digester Expansion

Chapter 8 presents an evaluation of the following two alternatives for meeting the long-term solids treatment needs for the WWTP:

- Construction of a fourth digester, and
- High solids digestion, accomplished by installation of recuperative thickening equipment on two of the existing digesters.

The analysis of these two alternatives demonstrated that the fourth digester provides for more hydraulic loading capacity, but the high solids digestion alternative provides for more volatile solids loading capacity. Thus, the high solids digestion alternative allows for increased loadings of hauled organic waste materials, which would allow for increased biogas production.

The analysis also showed that the lifecycle cost for adding a fourth digester is much lower than for high solids digestion (see Table 8-5). However, the high solids digestion could provide for additional economic benefits that would offset some of the costs. Adding a fourth digester has a significantly higher total subjective rating because it does not add a new treatment process or create a new, high-strength return stream at the WWTP (see Table 8-6).

The analysis presented in Chapter 8 also considered whether the high solids digestion alternative would provide benefits over adding a fourth digester with respect to addressing the DLD capacity issues. Specifically, because the digesters would produce treated biosolids at a higher solids content if a recuperative thickening process was added, the potential benefits of having these thicker solids on the sizing of a future dewatering project was considered. This analysis concluded that the benefits are limited, and do not justify the added cost of the recuperative thickening process.

Finally, with respect to the benefits associated with increased organic loadings, it was concluded that both options provide some increased capacity to receive hauled waste, although the additional digester would provide less than the high solids digestion option. Given the uncertainties with how much hauled waste may ultimately be available, the need for the increased capacity for hauled waste is uncertain. Moreover, if hauled waste digestion proves to be a viable concept, the District could still add a recuperative thickening process in the future. This concept and approach was further evaluated in Chapter 9.

Based on the information above, the District decided to proceed with design of a fourth anaerobic digester. The District also decided to install a FOG receiving station concurrently with the new digester. The District had previously worked to develop a design for this project, and was able to complete the design concurrently with the development of this Master Plan Report. The project recently bid and awarded at a construction fee of $7.8 million. The District expects project completion by the end of 2018.

10.1.3.2 Biosolids Thickening/Dewatering Improvements to Address DLD Capacity Limitations

As discussed in Chapter 8, the DLD site appears to have inadequate capacity to accommodate the existing biosolids hydraulic loadings, at least in wet years. Therefore, an alternative disposal strategy is needed to accommodate the solids that cannot be applied to the DLD site. The following alternatives for addressing this limitation were evaluated:

1. Mechanically thicken all FSL solids for DLD site application in summer (no offsite disposal).
2. Dewater a portion of digested solids for offsite disposal year-round.
3. Dewater a portion of FSL solids for offsite disposal in winter.
4. Dewater all FSL solids for DLD site application in summer (no offsite disposal).

Based on the cost analysis in Chapter 8, capital costs are lowest for Alternatives 2 and 3 (see Table 8-9), and the lifecycle cost is lowest for Alternative 3 (see Table 8-11). From a subjective ranking, Alternatives 2 and 3 ranked the highest. Thus, one of these two partial dewatering alternatives is recommended.

Chapter 8 also considers the costs that would be associated with going to full dewatering of FSL or digester solids depending on which of the four thickening/dewatering alternatives were initially implemented (see Table 8-19). This analysis shows that either Alternative 2 or Alternative 3 would result in the lowest long-term costs for full dewatering, depending on whether full dewater was from the digesters or the FSLs. If dewatering from the FSLs is likely to be feasible in the future (meaning the District loses the ability to use the DLD site, but not the FSLs), then Alternative 3 emerges as the overall low-cost option. If dewatering from the digesters is ultimately required (meaning the District loses the ability to use both the DLD site and the FSLs), then Alternative 2 emerges as the overall lowest cost option. Therefore, either Alternative 2 or Alternative 3 would be preferred.

For purposes of space and budget planning in this Master Plan, Alternative 2 is assumed. This alternative involves dewatering solids from the digester. Because few wastewater treatment plants include FSLs, there is more experience and certainty in dewatering digested solids. Thus, the feasibility of this strategy is more certain. In addition, it is expected that odor control will be needed because the digester solids have a higher volatile content. Therefore, Alternatives 2 is a more conservative approach from a facility requirement standpoint. Through the studies recommended in this Chapter, the District will evaluate whether a lower cost approach can be implemented.

Additional evaluations are needed to confirm the final selected alternative. Specifically, there are a number of uncertainties with respect to the volume of solids that must be processed, the appropriate odor control for the dewatering system, dewaterability of solids from the FSLs versus the digesters, and the costs for hauling and disposal. These factors should be further evaluated prior to selecting a preferred approach. Additional discussion of this recommended effort is provided later in this Chapter.

An analysis was developed to assess the potential timing of when the solids dewatering project should be implemented. The following specific assumptions were applied in this analysis:

- The solids content in the FSLs will increase over time as more solids are added to the FSL, and excessive solids accumulation occurs once stored biosolids reach a target of 10 percent solids. This is the approximate level beyond which the biosolids would be difficult to convey to and spread on the DLD site.
- Assuming a 50 percent volatile reduction, the mass of solids generated in the FSLs each year starts at about 1,720 dry tons per year in 2015 and increases linearly up to 2,200 dry tons per year in 2035. The annual mass of solids produced is spread evenly amongst the six lagoons.
- Approximately 1,210 dry tons of biosolids are harvested annually, and each FSL is harvested once every six years. Thus, the accumulated biosolids would be between approximately 500 and 1,000 dry tons per year (spread out over six lagoons).
- A range of starting conditions have been evaluated since the exact degree to which the FSL solids accumulation has occurred is uncertain. These scenarios include a worst-case condition of starting 100 percent full and a best-case of starting empty. Two additional scenarios evaluated are 10 percent and 20 percent full, which are assumed to be closer to current conditions.

Results of this analysis indicate that the FSLs would reach target percent solids between 2022 and 2025. Thus, it is currently estimated that the District would need to have the new dewatering facilities online between that time period. For purposes of this Master Plan, it has been assumed that the District would begin facilities design in FY 20/21.

10.1.3.3 Full Dewatering Expansion

There are number of factors that could result in a conclusion that application of solids on the DLD site and/or storage of liquid biosolids in the FSLs is no longer desired. Such factors include:

- New regulatory requirements that are applied to either the FSL or DLD facilities that require major capital investment.
- Increased demands for the use of the FSL or DLD properties for some other purpose, such as potable reuse treatment water facilities, solar facilities, or storage of treated effluent/recycled water.
- Increased value of biosolids with high volatile solids/nutrient content and/or increased value of phosphorus, thus driving an approach with full dewatering of solids directly from the digesters.

As discussed in Chapter 8, the project that is implemented to address the current DLD capacity issues discussed above will involve constructing a building and much of the major infrastructure associated with a full dewatering project. Because of this, expansion to full dewatering (whether dewatering from the FSLs or digesters) will require relatively minor improvements. The expansion project would primarily consist of adding additional dewatering equipment, expanding/modifying the conveyance facilities, adding odor control facilities (if not already provided), and/or expansion of the return flow equalization facilities (see Table 8-13). Therefore, the layout of the mechanical biosolids dewatering system would not change significantly when/if the District implements full dewatering.

Given the wide-ranging factors that could drive a decision to abandon the FSLs and/or DLD, the need for, and timing of, a full dewatering project is uncertain. Therefore, a preferred approach for implementing a full dewatering cannot be defined at this time. For purposes of space and budget planning in this Master Plan, it is assumed that the District would need to expand the partial dewatering system discussed above to a project that provides full dewatering from the digesters. This approach is more conservative with respect to the size and cost of the dewatering facilities. This is because the FSLs provide significant volatile solids reduction, thus reducing the total volume of solids that require dewatering and disposal.

Under the full dewatering from the digesters approach, the FSLs would be decommissioned in addition to the DLDs. Therefore, this alternative is less conservative with respect to total footprint required for solids management. Based on the recommendations in this Master Plan, the DLD area could be reused for other WWTP-related facilities, such as the potable reuse treatment facilities as discussed in Chapter 6 and/or the solar facilities discussed in Chapter 9. Similarly, as discussed in the Wastewater Storage Assessment TM (Appendix C), the FSLs could be repurposed for storage of treated effluent to help maximize the District's recycled water production capabilities. Given these potential opportunities/demands for the DLD and FSL properties, it is not recommended that the District transfer or repurpose these areas for non-WWTP uses within the planning period of this Master Plan.

As shown in Figure 10-3, the Master Plan schedule identifies the full dewatering project as occurring between FY 28/29 and FY 30/31. This timeline is recommended to place the dewatering project between the potential nutrient removal project and the potential treated water augmentation potable reuse project, thus distributing the District's resources more evenly over the long-term implementation period. The actual implementation timeline will depend on a wide-range of factors that cannot be reasonably predicted at this time. It is also possible that a full dewatering project will not be implemented within the Master Plan planning period if continued operation of the FSL and DLDs is demonstrated to be an ongoing cost-effective approach for solids management.

10.1.3.4 Potential Supercritical Oxidation Process Project

As discussed in Chapter 8, the District is considering partnering with Synagro and SCFI to implement a supercritical oxidation process (i.e., AquaCritox^{TM}) project at the WWTP site, near the existing FSLs. This new facility would mix liquid solids from the WWTP with solids received from multiple agencies throughout the Bay Area, and would process up to approximately 25,340 pounds of biosolids per day (expressed as dry weight). Chapter 8 presents an analysis of the potential benefits to the District that could be realized from a solids management perspective if a supercritical oxidation process project is implemented.

Chapter 8 includes an analysis of different potential liquid biosolids sources that could be provided to SCFI to support operation of the supercritical oxidation process. Assuming the District will implement either the Alternative 2 or Alternative 3 partial dewatering strategy discussed above, the analysis of liquid biosolids sources shows the following:

- Sending digested solids from a recuperative thickening process would have the highest overall economic benefit to the District (see Table 8-22). However, as discussed previously, the savings realized from this approach would not offset the added cost of the recuperative thickening equipment (as compared to adding a fourth digester).
- The savings associated with sending undigested solids to the AquaCritox^{TM} process are similar to sending digested solids. Therefore, the cost of conveying undigested solids to the SCFI facility are not justified.

Given the information above, the best strategy will involve sending digested solids to the AquaCritox^{TM} process.

Ultimately, it is only recommended that the supercritical oxidation process be implemented if a financial package can be provided by SCFI that indicates financial benefits for the District. Based on the current available information, the savings are potentially limited. Additional discussion of potential costs and savings associated with a SCFI project is provided in the Near-Term Strategy section of this Chapter.

To gain the most benefit from the savings, the District would need to make a decision regarding the project prior to implementation of the partial dewatering project discussed above. Therefore, once more information is available, a final decision on implementing this process should be made.

For purposes of this Master Plan, the recommended timing for implementation of a supercritical oxidation process are identified. However, costs associated with this project are not included because it is assumed that costs would be covered through a development agreement with SCFI.

The District is recommended to make a final decision about the viability of a supercritical oxidation process at the WWTP by the end of 2018, such that the analyses are completed concurrently with the evaluation of partial dewatering alternatives discussed above.

10.1.4 Energy Management

Chapter 9 presents a discussion of potential strategies for achieving net zero energy demand at the WWTP. The projects related to this goal that are recommended for inclusion in the Master Plan CIP are as follows:

- FOG Receiving Facility
- Biogas Treatment System
- Hauled Organics Waste Receiving Facility
- Expanded Digestion and Cogeneration Facilities
- Solar Facilities

10.1.4.1 FOG Receiving Facility

The addition of the fourth digester will provide excess digester capacity and allow for receipt of hauled wastes at the WWTP for co-digestion with WWTP biosolids and production of additional biogas. FOG is commonly used for co-digestion and is a preferred feedstock. FOG also may help reduce the volume of digested biosolids, and thus has a benefit with regards to biosolids disposal. For these reasons, the District recently decided to construct a FOG receiving station with the new digester; this facility should be online by the end of 2018.

10.1.4.2 Biogas Treatment System

The existing biogas treatment system at the WWTP is reaching the end of it useful life. The system is also operating at its maximum capacity, and the District sometimes cannot process all the biogas produced. Moreover, with the addition of the FOG receiving station, even more biogas will require treatment. Finally, the existing biogas treatment system has various inefficiencies. For these reasons, a new gas treatment system is recommended.

As discussed in Chapter 9, a treatment system that is sized for the biogas needed to run the existing cogeneration engines is recommended. The recommended approach involves installing gas treatment units sized for the engines, but installing compressors only as needed to meet the biogas generation rates. The larger treatment modules, which will minimize the overall footprint area, should not impact performance. In addition, the smaller compressors will provide for more optimized energy efficiency.

Given the benefits that will be realized with the installation of the biogas treatment system, it is recommended that this project proceed early in the near-term implementation period.

10.1.4.3 Hauled Organics Waste Receiving Facility

As discussed in Chapter 9, it is unlikely that there would be enough FOG available to maximize the digestion and cogeneration capacity at the WWTP. Therefore, the District will need to receive additional organic feedstocks to produce enough biogas to power the existing cogeneration engines.

The specific equipment and storage/conveyance facilities that would be required to accommodate acceptance of alternative organic waste feedstocks cannot be identified until specifics regarding the feedstock are known. Indeed, if a feedstock is identified that has a similar solids content as FOG, it may be possible to use the FOG receiving facility for both operations. Moreover, the economics of the alternative feedstock program would likely be structured in a way to pay for any capital improvements. It is also possible that all or a portion of the receiving facility costs would be covered under a potential third-party agreement for acceptance of the hauled waste. Given these uncertainties, the cost for an alternative feedstock receiving facility is not defined in the CIP. However, a recommended location for a future receiving facility has been identified. In addition, this Chapter discusses the recommended timing and strategy for implementing an expanded co-digestion program.

As discussed above, the new FOG receiving station is expected to be online by the end of 2018. Once this project is in place, the District will be able to gain a better understanding of the potential impacts associated with accepting hauled waste at the WWTP. It is also recommended that the new, larger gas treatment system be brought online as soon as possible. With this facility in place, the District would have the capacity to treat additional biogas generated from hauled waste. Therefore, with both projects completed, the District will be able to bring in waste streams for co-digestion/biogas generation.

Given the need to implement these other two projects prior to bringing in alternative organic waste feedstocks, coupled with other staff time commitments in the near-term period, it is recommended that the District target the mid-term period as the desired timeline for implementing an expanded co-digestion project that relies on feedstocks beyond FOG. State regulations require a 75 percent reduction in organics disposed in landfills by 2025. Therefore, the recommended implementation plan will provide the benefit of allowing time to learn from other facilities' experiences, while still having a co-digestion project online prior to the 2025 deadline for organics reduction from landfills.

10.1.4.4 Expanded Digestion and Cogeneration Facilities

The analysis presented in Chapter 9 (Table 9-17) shows that expanding the WWTP's onsite energy generation to Step 4 (which involves increasing cogeneration capacity, expanding gas treatment, and expanding biosolids dewatering) or Step 5 (which involves installing a recuperative thickening process, further increasing the cogeneration capacity, further expanding the gas treatment system, and further expanding the biosolids dewatering system) would have an unfavorable payback potential of 30 and 50 years, respectively. In addition, it is unclear whether a robust hauled organics waste acceptance program can be supported at the main WWTP site given its location in an urban area and the fact that the hauled waste facilities would be a source of odors and that the program would result in increased truck traffic. Given these uncertainties, costs are not included in this CIP for expansion of the digestion system or cogeneration system to accommodate an expansion to Steps 4 and 5.

However, it is likely that costs for energy generation from biogas will become less expensive as more projects at other facilities are implemented and the technologies to support these programs evolve. Moreover, the District may determine through operation of the initial hauled waste program that the odor and traffic issues associated the operation can be mitigated. Therefore, the District should continue to evaluate options for increased onsite energy generation through organics digestion.

The District should also reserve the former fuel cell area for potential expansion of the cogeneration facilities and reserve space just east of the new digester for the potential recuperative thickening system and expansion of the biogas treatment system. These locations are indicated on Figure 10-1.

10.1.4.5 Solar Facilities

There is limited space available for a solar installation the WWTP. At the main WWTP site, the only area available for solar panels would be the building rooftops and within the footprint of the holding basin. In total, this area would provide approximately 2.3 acres of solar panels, which should accommodate an approximate 500-kW system capable of generating approximately 100 kW of energy on average. This generating capacity is less than 10 percent of the total WWTP demand. The analysis presented in Chapter 9 indicates that the District could realize an approximate 20-year payback from this system.

If the District can work with a third party to develop a P3 agreement for a solar power facility at the WWTP site, federal tax credit incentives associated with solar power installation could be realized. However, given the relatively small size of the system and the need to distribute the solar panels around the site, it is unclear whether the District could attract a third party for a P3 agreement.

In the future, if the DLD site is decommissioned, this area would provide an opportunity for a large solar project. As discussed in Chapter 9, a solar facility that encompasses less than half the DLD area could produce the power for the WWTP beyond what can be produced from the existing cogeneration engines. Moreover, because the DLD site would provide one, large and contiguous system, the District could likely attract several opportunities for a P3 agreement with a third party. Therefore, this approach should be considered if the DLD site is abandoned in the future.

10.1.5 Odor Control

An Odor Control Plan, and supporting TM No. 2 - Odor Control Improvement/Expansion Recommendations, were prepared as part of this Master Plan (Appendix D). These documents provide an evaluation of existing odor control facilities and identify recommended improvements. These documents also present a recommended strategy for expanding the odor control system to the primary sedimentation basins and the aeration basin inlet channel.

10.1.5.1 Existing Facility Improvements

The Odor Control Plan identified replacement of the ORT media, or wholesale replacement of the ORT with an alternative technology, as a recommended project to increase sulfide removal rates from 96 percent to 99 percent or greater. As discussed in TM No. 2, it is recommended that either a chemically-based packed tower or activated carbon process be installed in place of the ORT. Both technologies have a smaller footprint than the ORT, thus providing space for additional odor control units to serve the primary clarifiers and the aeration basin inlet channel in the future (see discussion below). Further evaluation is needed to identify the preferred technology. In addition, as discussed in Section 10.2.2, this analysis should consider whether a whole-plant system located at the headworks biofilter location (and incorporating the functions of this facility) is a more cost-effective, long-term strategy. However, for purposes of this Master Plan, an activated carbon filter system is assumed.

The existing ORT cannot provide the odor removal rates achievable by other conventional technologies, and performance is not expected to improve. Therefore, the District should address the ORT issues during the near-term implementation period.

10.1.5.2 Odor Control Expansion

As discussed in the Odor Control Plan, previous odor control studies have indicated that additional odor control could be provided by adding covers to, and providing odor control for, the primary sedimentation basins, the aeration basin inlet channel, and the unaerated anaerobic selector. Discussions with District staff indicate that the next logical expansion of the odor control system would be limited to the primary sedimentation basins and the aeration basin inlet channel.

To provide odor control for the ORT sources, the primary sedimentation basins, and the aeration basin inlet channel, two sodium hypochlorite packed towers (10-foot diameter each) or two activated carbon filters (12-foot diameter each) would be needed. Although a slightly larger footprint than the existing ORT (12 feet by 16 feet), two chemically based packed towers or activated carbon facilities could fit nearly with the area where the ORT is currently located. As noted above, additional analysis is needed to further compare the capital and operating costs of these two approaches. As noted above, this Master Plan assumes that an activated carbon filter system will be installed.

10.2 NEAR-TERM IMPLEMENTATION PLAN (FY 17/18 TO FY 20/21)

The Master Plan implementation for the near-term period (2017 through 2021) strategy is presented below. Recommendations are presented for facilities that should be designed and constructed, studies or evaluations that should be completed, and emerging issues and topics that should be monitored.

10.2.1 Near-Term Design/Construction Projects

In the near-term period, the District will need to design and construct several projects to address existing capacity and system deficiency issues. These projects are summarized in Table 10-2. The anticipated years when design and construction would be initiated are also shown.

Table 10-2. Recommended Near-Term Construction Projects

Project	Recommended Timing	Begin Design/ Environmental Review	Begin Construction
Liquid Treatment			
Primary treatment expansion	Design services are currently under contract	FY 17/18	FY 18/19
Primary effluent diversion	The relatively minor projects could be designed and constructed simultaneously with the primary sedimentation basin project	FY 17/18	FY 18/19
Hydraulic improvements		FY 17/18	FY 18/19
Alum addition for FSL return stream	This project will reduce plant complexity and improve overall performance and should be implemented as soon as practicable	FY 19/20	FY 20/21
Solids Treatment			
Fourth anaerobic digester	Project has recently been bid	FY 16/17	FY 17/18
Supercritical oxidation	A decision about this project should be made before design of the partial dewatering project begins	FY 18/19[(a)]	FY 20/21[(a)]
Energy Management			
FOG receiving station	Project has recently been bid (with fourth digester)	FY 16/17	FY 17/18
Gas treatment system	Design of the gas treatment facility should begin as soon as possible so it will be online concurrent with the FOG receiving facility	FY 17/18	FY 18/19
Odor Control			
Existing Odor Control Facility Improvements	This project could be completed soon after completing the primary sedimentation basin project, thus offsetting the potential odor increases	FY 19/20	FY 20/21

[(a)] Estimated dates for project development, if the project is determined to be feasible.

10.2.2 Near-Term Evaluations

In the Near-Term Implementation Period, the District will need to complete the following evaluations:

- Alum Addition Jar Testing
- CWR Facilities Potable Reuse Plan
- Biosolids Dewatering Facilities Plan
- Supercritical Oxidation Process Financial Analysis
- Energy Demand Reduction Projects
- Odor Control Improvement/Expansion Plan

10.2.2.1 Alum Addition Jar Testing

Jar testing is needed to support the design of the alum addition facilities. The purpose of this testing would be to confirm/verify dosing rates, which will dictate the sizing of the alum storage facility. Jar testing can be completed in an approximate two-month period.

10.2.2.2 CWR Facilities Potable Reuse Plan

As discussed previously in this Chapter, additional analysis is needed to confirm the preferred approach for potable reuse and whether it will involve rehabilitation of the CWR facilities. To develop a CWR facilities potable reuse plan, the following actions need to be completed:

- Confirm Regional Decision(s)
- Identify Potential Supplemental Water Supplies
- Confirm Desired End Use(s)
- Conduct CWR Facilities Condition Assessment
- Select Preferred Treatment Train

10.2.2.2.1 Confirm Regional Decision(s)

Under a separate effort, the District is conducting a potable reuse study in partnership with the Cities of Livermore, Pleasanton, the California Water Service Company, and the Zone 7 Water Agency. The purpose of this study is to evaluate the feasibility of a region-wide potable reuse project. With this regional approach, treated WWTP effluent would represent a portion of the regional water portfolio, potentially being used as one source of supply to a potable reuse treatment system. Information presented in this Master Plan is intended to provide some of the details needed to compare alternative reuse options.

The regional potable reuse study is expected to be complete in FY 17/18. Once this study is complete, the District will have a better understanding of the range of potable reuse opportunities in the tri-valley area and a decision regarding the preferred potable reuse project direction (regional or independent) can be made. The District may determine that investment in a regional project would better serve the District's needs than rehabilitation of the CWR facilities. Moreover, the regional study may demonstrate that locating a regional potable reuse treatment system at the DSRSD WWTP site is desirable and/or that rehabilitation of the CWR facilities as a demonstration/training facility is a logical first step for the regional project.

10.2.2.2.2 Identify Potential Supplemental Water Supplies

As noted previously in this Chapter, the potable reuse water production costs will be elevated under a District-only project because the treatment system will only be supplied with a water supply (i.e., the treated WWTP effluent) part of the year. Therefore, if a regional project is not pursued, the District should continue to evaluate whether there are other water supplies available that could support a potable reuse project at the WWTP site. Potential supply options that could be considered include: treated effluent from the City of Livermore's wastewater treatment plant; capture of dry-season runoff from stormwater channels that serve urban areas around the WWTP; or raw wastewater supplied from the Central Contra Costs Sanitary District collection system to the headworks of the District's WWTP.

10.2.2.2.3 Confirm Desired End Use(s)

Chapter 7 of this Master Plan defines recycled water production costs for a potable reuse treatment system at the WWTP under various potable treatment scenarios. However, the costs and facilities layout presented herein only represent a portion of the total project costs, as the analysis does not include the facilities needed to convey the treated water to the ultimate end use site. Therefore, if a regional solution is not selected, then the District will need to further define the preferred end use of the potable water generated at the WWTP.

A focused study, that considers the information developed from the regional evaluation but also takes into account the District's specific needs, is recommended to define the long-term potable reuse implementation strategy before a significant investment in infrastructure is made. However, any evaluation of alternatives that involve raw water or treated water augmentation will include some uncertainty until State regulations define the required treatment trains for these types of projects.

As noted previously, discussions with District staff indicate that reservoir water augmentation is an option for potable reuse in the tri-valley area. Moreover, because regulations for raw water/treated water augmentation are not likely to be adopted for a five to ten-year period, implementation of a raw water/treated water augmentation project is not recommended in the near and mid-term timeframe. Therefore, if implemented, an initial potable reuse project at the WWTP would likely involve reservoir water augmentation as an end use. Moreover, based on information provided from District staff, the likely reservoir that would be augmented is Lake I in the Chain of Lakes that is operated by Zone 7 and is planned to ultimately supply water to the Del Valle WTP.

Another potential approach for the District's initial potable reuse project would be to develop the CWR facilities as a "demonstration and training" facility. Under this scenario, the treated water would be blended with disinfected tertiary recycled water and discharged to the District's recycled water customers, thus providing water quality benefits such as reduced salinity to this supply. This approach would also provide District staff (or others) experience with the challenges of meeting operational requirements associated with generating a potable reuse water supply, without the risks associated with non-compliance. The experience gained from operating a smaller potable reuse facility could be readily applied to operating larger facilities, thus providing for efficient startup and operation of a larger project. District staff could also offer training to staff involved with other potable reuse projects in the Bay Area. This strategy could also provide the District with more flexibility to implement a raw/treated water project after the applicable regulations are defined.

Finally, regardless of the identified preferred strategy for the near-term potable reuse project, the District should first gain experience operating a potable reuse facility prior to delivering water to the ultimate potable reuse end user. This initial operating period will define the operational strategies needed to provide reliable treatment and avoid potential issues once deliveries begin. This strategy will be particularly important if treated water augmentation is the ultimate goal, and the benefits of this approach are likely to be realized even if the ultimate project involves a more complex treatment process.

Table 10-3 provides a summary of these two potential initial approaches for development of a District-only potable reuse project using the CWR facilities. Specific pros and cons of each approach are identified.

Table 10-3. Potential Phases for a District-Only Potable Reuse Project

Potential Initial Potable Reuse Phase	Initial Use of Potable Water Supply	Potential Second Potable Reuse Phase	Driver for Change in Operations	Pros	Cons
Demonstration Project	Distributed to irrigation customers	Reservoir Augmentation	An agreement is reached with Zone 7 for augmentation of the Chain of Lakes.	• Provides an opportunity for the District to become familiar with potable reuse operations before investing in major infrastructure beyond the WWTP boundaries.	• The District would expend significant operating costs to produce water of higher quality than needed until the second phase project is developed. • A second phase may never be implemented because an end use cannot be cost-effectively developed.
		Raw Water Augmentation	An agreement is reached with Zone 7 for augmentation of the Del Valle WTP supply.		
		Treated Water Augmentation	Regulations allow for production of treated drinking water from a facility at the WWTP site.		
Reservoir Augmentation	Provided to Zone 7 Water Agency via Discharge to the Chain of Lakes	Raw Water Augmentation	An expansion in discharge volumes is desired, and expanded project is considered raw water augmentation.	• Most (if not all) potable water generated is put to high-value use. • Ensures there is an agreement in place for long-term use – regardless of whether a second phase is implemented.	• Requires facilities to deliver water to the Chain of Lakes, which would reduce funding available for other improvements.
			Operational difficulties can be cost-effectively addressed by discharging directly to the Del Valle WTP.		
		Treated Water Augmentation	Regulations allow for production of treated drinking water from a facility at the WWTP site and production costs are reduced by providing a higher level of treatment at the WWTP and distributing treated water directly to DSRSD customers.		

10.2.2.2.4 Conduct CWR Facilities Condition Assessment

Additional assessment of the CWR facilities rehabilitation requirements is necessary to further assess the costs and viability of this approach. Assuming the efforts described above conclude that CWR facilities rehabilitation is a potential and/or desired strategy, the next logical evaluation step would involve a detailed condition assessment of the facilities to identify what specific pieces of equipment can be reused, and what requires replacement to get the system fully operational.

10.2.2.2.5 Select Preferred Treatment Train

As discussed previously, this Master Plan assumes that the facilities to be provided under a CWR rehabilitation is the FAT process defined in current potable reuse regulations (MF, RO, UV/AOP) along with the ozonation plus BAC pretreatment facilities discussed in Chapter 7. Under current regulatory guidelines, this treatment train would be required if a reservoir water augmentation potable reuse project is pursued. However, as discussed further below, other treatment strategies may be viable.

Assuming the District does decide to rehabilitate the CWR facilities, the District should further evaluate and select the preferred treatment approach based on the desired reuse strategy. The additional evaluation of treatment approaches should consider the following:

- **Reliable Operations:** Regardless of the intended use of the CWR water supply, the approach of not providing pretreatment ahead of the conventional FAT process is not recommended. This is primarily because a pretreatment system will improve the membrane performance, and require less extensive membrane system maintenance. Given the District's desire to implement higher levels of potable reuse in the future, gaining experience handling a wastewater that has been pre-treated is recommended. In addition, eliminating operational issues associated with treating secondary effluent in the FAT process would be useful if the WWTP is used as a training facility.
- **Emerging Potable Reuse Regulations**: Current guidelines are evolving for potable reuse as DDW approves different treatment processes for log-removal credits. Therefore, some of the pretreatment alternatives discussed (like ozonation as a stand-alone process or full NDN may satisfy the treatment requirements for higher levels of potable reuse even though current guidelines would not allow for this approach).
- **Nutrient Removal Requirements for San Francisco Bay Discharges**: The full NDN pretreatment process may be needed for San Francisco Bay discharges within a similar timeline when the potable reuse project is developed and the mutual benefits of this approach should be considered.

Stand-alone ozonation was the lowest cost pretreatment strategy identified for the CWR rehabilitation project (see Table 7-7). This approach may be preferred if: 1) adequate log-removal credits can be achieved with ozonation as a stand-alone process to allow for the desired near-term use of the CWR facilities water supply, and 2) it is not clear whether nitrogen removal will be necessary for continued San Francisco Bay discharge. However, if ozonation alone cannot provide the required log-removal credits for the intended reuse project (along with the FAT process), then the District will need to consider either ozonation plus BAC, full NDN, or possibly a combination of full NDN plus ozonatoin/BAC.

Ozonation plus BAC had the highest total subjective ranking of the pre-treatment alternatives identified for the CWR rehabilitation project (see Table 7-6). This is primarily because ozonation plus BAC alternatives is currently accepted to be a process that reduce pathogens for potable reuse treatment (thus allowing for higher uses of the treated water). In addition, this strategy does not require that the District commit to a nutrient removal approach at this time. However, the full NDN approach may be preferred if: 1) potable reuse guidelines are modified to provide log-removal credit for a full NDN treatment process making this a viable pretreatment approach for the intended project, and 2) it is clear that nitrogen removal will be required for San Francisco Bay discharges and alternative emerging technologies are not considered viable in the timeframe required. Under these circumstances, the full NDN pretreatment approach would be the most cost-effective pre-treatment strategy because it essentially adds no cost for stand-alone pretreatment.

Even if all the necessary log-removal credits could not be achieved with full NDN plus the FAT process, it could be determined/demonstrated that full NDN plus ozonation is a viable approach to gaining the desired credits. Again, because full NDN may be needed for San Francisco Bay discharges, this approach would be more cost effective than the ozonation plus BAC approach that has been assumed to be necessary in the absence of NDN pretreatment.

Finally, research is currently underway to demonstrate whether ozonation plus BAC could be an acceptable replacement for the RO membrane process in a FAT treatment system. If DDW defines as a viable approach for FAT, a treatment train that does not include RO, the capital and operating costs of potable reuse would significantly decrease. This emerging opportunity should evolve over the next two to three-year period, and the potential for savings with this approach is one reason to postpone development of the potable reuse project for a few years.

10.2.2.3 Biosolids Dewatering Facilities Plan

As discussed previously in this Chapter, additional analysis is needed to confirm the preferred partial dewatering alternative for addressing the DLD capacity issues. To develop a facilities plan for this project, the following actions need to be completed:

- Confirm FSL Volatile Reduction/Solids Accumulation
- Identify Preferred Reuse/Disposal Strategy
- Select Preferred Dewatering Alternative
- Select Preferred Equipment

10.2.2.3.1 Confirm FSL Volatile Reduction/Solids Accumulation

Due to limited data availability, assumptions were applied in this Master Plan with respect to volatile solids reduction in the FSLs. These assumptions directly influence the calculations used herein to determine the mass of solids that are currently accumulating in the FSLs, and thus the mass of solids that require alternative disposal. The assumptions about the volatile solids reduction rate also affect a decision about the timing of when the dewatering project needs to be implemented. If volatile solids reduction rates are higher than assumed, the sizing of the biosolids dewatering facilities could be decreased and implementation of the project delayed. If volatile solids reduction rates are lower than assumed, the capacity of the biosolids dewatering facilities would need to be larger and the project timing would be sooner.

A recommended initial step is to collect additional volatile solids data for the solids being applied to the DLD to better assess the extent of volatile solids reduction in the FSLs. This data collection effort could also include measurements to assess the extent to which solids have accumulated in the lagoons.

10.2.2.3.2 Identify Preferred Disposal/Reuse Strategy

A critical first step needed if a District-only project is preferred is to conduct a market research study to assess possible offsite disposal/reuse options for the dewatered biosolids and confirm hauling and tipping fees. The lifecycle cost estimates in Chapter 8 assume a hauling and tipping fee of $50 per wet ton. However, costs could be significantly lower, depending on the ultimate disposal location.

Long-term disposal of the dewatered biosolids in a landfill (even as alternative daily cover) is not likely an option due to the requirements for reducing organics being directed to landfills. Therefore, other offsite disposal options will need to be considered, many of which would involve reuse of the biosolids. Options that may be available include agricultural reuse/composting, use as a raw product for a fertilizer manufacturing or use as a raw product in an energy generation facility. The District could also consider installing a reuse technology option at the WWTP to further processing the solids onsite to produce a more marketable product.

A list of potential options for biosolids nutrient/energy recovery that could be considered as part of this effort is provided in Table 10-4. Note that this list includes the SCFI alternative currently being considered for the WWTP site, and it is expected that this technology will be compared to other potential options.

Table 10-4. Biosolids Nutrient Removal/Recovery Technologies

Technology (Manufacturer)	Technology Status[(a)]	Type of Use/Product	Pros	Cons
Composting (various)	Proven	Compost	Lower cost option	Offsite hauling
Hydrolysis (Lystek)	Emerging	Liquid fertilizer	Increased solids destruction and gas production	Odors, storage for product needed
Hydrolysis (CAMBI)	Proven	Class A biosolids	Increased digester loading and gas production	2 dewatering steps needed, Odors
Hydrothermal (Genifuel)	Emerging	Alternative fuel, biocrude and natural gas	Reduce undigested or digested solids	Heat energy required
Thermal Drying (various)	Proven	Alternative fuel, soil amendment	Reduce hauling costs, more options for use	Heat energy required for drying
Thermal Drying (Bioforcetech)	Emerging	Alternative fuel, soil amendment	Reduced hauling costs, more options for use	New technology
Drying (Wright Tech)	Emerging	Alternative fuel, soil amendment	Reduced hauling costs, more options for use	Large footprint
Supercritical Oxidation (SCFI)	Demonstration	Ash	Large volume reduction, resource recovery	Liquid oxygen storage needed, new tech

[(a)] Technology Status progresses from "Demonstration" to "Emerging" and then finally to "Proven".

10.2.2.3.3 Select Preferred Alternative

As discussed in the Recommended Facilities Plan section of this Chapter, it has been assumed for purposes of space and budget planning in this Master Plan that biosolids dewatering Alternative 2 will be implemented. This alternative involves dewatering solids from the digesters. However, significant savings could potentially be realized if Alternative 3 (dewatering from the FSLs) is implemented. The potential for such savings will need to be re-evaluated once dewaterability and odor control requirements are better defined.

To assess the dewaterability and odor control requirements, it is recommended that the District retain the services of a third party to complete a demonstration study. The demonstration study would specifically include the following three tasks:

- Dewaterability: Both digester solids and FSL solids should be dewatered for a few weeks to demonstrate the amount of chemical and staff time needed for dewatering solids from each source.
- Odor Potential: District staff could complete odor monitoring during the demonstration study to assess the odor potential of both dewatering operations. The odor evaluation could also include assessment of odor potential from spreading the dewatered biosolids on the DLD site.
- Beneficial Reuse: Another factor that should also be considered is whether the value of the biosolids as an energy or nutrient resource is different if they have been further digested in the FSLs. To address this, the characteristics of each biosolids product with respect to volatile solids and nutrients should be measured.

The information from this effort would be used to complete an updated lifecycle cost analysis of both alternatives by taking into account more certain estimates of: odor control requirements, staff and chemical use costs, opportunities for onsite disposal (on the DLD site), and offsite disposal costs/revenues.

10.2.2.3.4 Select Preferred Equipment

Once the preferred dewatering approach is defined (i.e., dewatering from the digesters or from the FSLs), the District should evaluate and pilot test different dewatering technologies. The evaluation would include visits to facilities to observe operations of different types of dewatering equipment. If more than one potential equipment type is identified, the District could pilot test each type to identify the equipment that would be the most appropriate for the District's needs. Important factors to consider when selecting dewatering equipment include the ease of operations, operating costs (power and chemical usage), operating range, and the level of automation and staffing requirements.

10.2.2.4 Supercritical Oxidation Financial Analysis

As noted previously, the District should assess whether adding a supercritical oxidation process at the WWTP would be financially beneficial. The following factors should be considered:

- As shown in Table 8-22, the potential capital savings for the District's dewatering project if digester solids are sent to the supercritical oxidation process is estimated to be between $100,000 and $400,000. These savings should be re-evaluated once the sizing and layout of the dewatering system is defined.
- As shown in Table 8-23, the cost to construct the conveyance facilities to deliver dewatered solids to the supercritical oxidation process is estimated to be approximately $200,000. Unless SCFI covers the costs associated with the digested solids conveyance facilities, there may be limited benefit from a capital cost standpoint for implementing a supercritical oxidation process at the WWTP.

- Assuming digested solids are used as a feedstock, the supercritical oxidation process would use approximately 180 dt/yr of the District's digested biosolids. This would reduce the mass of solids that require alternative disposal by approximately 17 percent. Thus, the chemical and hauling/disposal costs shown in Table 8-10 would be reduced by this percentage, which equates to an operating costs savings of $85,000 and $100,000 per year.
- Any savings related to biosolids disposal would need to be compared to the increased operational costs that would be associated with handling the return flows from the supercritical oxidation process.
- It may be possible that SCFI would charge a tipping fee for disposal of dewatered solids that is less than the cost to haul the solids to another site for disposal. Savings associated with this potential operation should be defined concurrent with the market assessment discussed above. As noted previously, this analysis should consider whether the value of the biosolids as an energy resource for supercritical oxidation decreases if they have been further digested in the FSLs.

10.2.2.5 Energy Demand Reduction Projects

The District should continue to evaluate the opportunities for energy demand reductions at the WWTP that were presented in Chapter 9 during each phase of implementing the CIP. A specific project that should be considered in the near-term is replacement of barscreen building exhaust fans.

In addition, the District should observe some energy savings from the primary treatment capacity expansion project, as the loads to the aeration basins will be reduced and volatile solids loading to the digesters will be increased. The alum addition project, which provides for year-round anaerobic selector operations, should provide some energy savings. Finally, replacement of the gas treatment system with a more efficient process will provide reductions.

10.2.2.6 Odor Control Improvement/Expansion Plan

Two technology options have been identified for replacement of the ORT: sodium hypochlorite packed tower or activated carbon filter. Additional analysis of these options is needed to determine which is the most cost-effective long-term strategy.

The sodium hypochlorite packed tower option is potentially a lower cost option than an activated carbon filter from both a capital and operating costs standpoint. However, the cost of the sodium hypochlorite packed tower option is contingent on the cost to convey sodium hypochlorite from the District's existing chemical storage facilities to the ORT site. Given the existing sodium hypochlorite storage facility is located on the opposite side of Holding Basin 1, piping from the storage facility would likely need to be routed around this basin. In addition, an assessment of whether the existing sodium hypochlorite storage is adequately sized to accommodate both the WWTP odor control and disinfection needs would be required. Finally, operating costs for both options would need to be defined based on site-specific information from local suppliers.

The space requirements for either odor control option should also be confirmed. Specifically, a layout of the facilities required for each option would need to be developed for the expanded system. This analysis would confirm that either option would reasonably fit within the area taken up by the existing ORT footprint, and not impinge upon other existing or planned facilities/operations at the WWTP.

Finally, if it is determined that the headworks facility biofilter requires significant additional improvements or replacement, the District should consider a third "whole plant" option. Under this third option, the District would install a deep bed Biosorbens® biofilter where the existing headworks biofilter is currently located. This filter would initially be sized to accommodate the headworks area, the existing ORT sources, and would be expandable to accommodate the primary clarifiers and aeration basin inlet channels. Sizing estimates presented in the Odor Control Plan indicate that such a facility would have a smaller footprint than the existing headworks biofilter, and would be the most sustainable, long term approach. However, as noted in the Odor Control Plan, the largest obstacle for this option would be the routing of large ducts from the ORT, primary sedimentation basins, and aeration basin inlet channel to the headworks biofilter location. The costs and feasibility of installing this ducting would need to be carefully evaluated.

10.2.3 Near-Term Monitoring Activities

There are several potential future constraints and opportunities for the WWTP that will not impact near-term decisions, but will impact the District's wastewater treatment decisions in later phases. During the near-term period, these items should be monitored for developments that should be considered in later phases as future projects are implemented. The following specific emerging regulations and opportunities should be monitored:

- Nutrient Regulations
- Nutrient Treatment Technologies
- Raw Water and Treated Water Augmentation Regulations
- Co-digestion Opportunities

10.2.3.1 Nutrient Regulations

The District should keep abreast of emerging regulations for nutrient treatment for San Francisco Bay discharges. It is expected that information regarding potential regulations will start to emerge after the next Nutrients Watershed Permit is adopted in 2019. As discussed previously in this Chapter, an understanding of what the regulations may be could inform decisions regarding the desired potable reuse pretreatment facilities.

In addition, if nutrient requirements are implemented on a seasonal basis, the District may be able to avoid surface water discharge during the irrigation season when all influent can be reused as recycled water. In that case, no additional nutrient removal would be required. Therefore, if seasonal load-based limits are identified as a likely outcome in the 2024 permit, the District may determine that expansion of the tertiary reuse opportunities should be more aggressively pursued in the near and mid-term periods.

10.2.3.2 Nutrient Treatment Technologies

Activated sludge treatment using anammox bacteria in a full-scale wastewater treatment plant is emerging as a viable technology for nitrogen removal. If this treatment approach can be shown to provide reliable treatment, wastewater treatment facilities could achieve significant nitrogen reductions with less space and using less energy than conventional approaches. During the near-term period, the District should monitor the developments in this very promising technology.

10.2.3.3 Raw Water and Treated Water Augmentation Regulations

In the near term, it is not expected that raw water and treated water augmentation requirements will be defined adequately in California to allow the District to proceed with such a project. However, because the District has defined raw water and treated water augmentation as a likely long-term strategy, it will be important to keep abreast of emerging regulations for these types of projects. Areas of interest would be the requirements being defined for treatment, monitoring, process control, and staff training/experience. If it appears that the requirements for raw water or treated water augmentation will be too onerous, the District may decide to reduce or eliminate the resources being directed toward this potential opportunity.

Another related topic that should also be monitored are studies currently being developed to evaluate whether ozonation plus BAC is an acceptable replacement for the RO membrane process in the FAT treatment train. A treatment train that does not include RO would have significantly lower capital and operating costs, and would eliminate the need for a brine discharge. Since the analysis presented in this Master Plan assumes that RO treatment and a brine discharge are a necessary component of any potable reuse project, the evaluations and conclusions presented in this Master Plan would need to be revisited.

10.2.3.4 Co-Digestion Opportunities

There are several wastewater treatment facilities in the Bay Area that have implemented, or are in the process of implementing, organic hauled waste receiving programs. In the near-term period, the District should monitor the information being reported back from these facilities with respect to benefits and impacts observed. Specific factors of interest include:

- Impacts on treatment performance
- Impacts on maintenance
- Increased solids production rates
- Increased biogas production rates
- Odor generation potential

The District should also remain open to discussions regarding potential sources of feedstock. However, it is not recommended that a preferred strategy be defined until some of the factors listed above are better understood.

10.3 MID-TERM IMPLEMENTATION STRATEGY (FY 21/22 TO FY 25/26)

The Master Plan implementation strategy for the mid-term period (2021 to 2026) is presented below. Recommendations are presented for facilities that should be designed and constructed, studies or evaluations that should be completed, and emerging issues and topics that should be monitored. Figure 10-3 provides a schedule overview of the recommended implementation plan for the entire 20-year planning period.

10.3.1 Mid-Term Design/Construct

In the mid-term period, the District will need to complete construction of the partial dewatering facilities. The timing of other improvements to be completed in this time frame will depend on the outcome of studies completed in the near-term period. The projects anticipated in the mid-term are summarized in Table 10-5. The anticipated years when design and construction would be initiated are also shown.

Table 10-5. Recommended Mid-Term Construction Projects

Project	Recommended Timing	Begin Design/ Environmental Review	Begin Construction
Potable Reuse			
Rehabilitation of CWR facilities	Because a significant number of projects are planned in the near-term period, the CWR rehabilitation project is recommended as a mid-term project. This timeline also provides for further evolution of current and new regulations.	FY 23/24	FY 25/26
Solids Treatment			
Partial biosolids dewatering	The specific timing will be determined under evaluations completed in 2017 and 2018. Implementation at the beginning of the mid-term period is expected.	FY 20/21	FY 21/22
Energy Management			
Hauled organics waste receiving facility	Programs currently being implemented by other agencies in the area should be evaluated prior to a commitment. The project should be completed prior to the 2025 deadline for landfill organics reduction.	FY 22/23	FY 23/24
Odor Control			
Odor control expansion	The timing/need for this project will be contingent on the magnitude and extent of odor complaints received for the WWTP. This project could get pushed into the long-term implementation period.	FY 23/24	FY 24/25

10.3.2 Mid-Term Evaluations

During the Mid-Term Implementation Period, the District is recommended to further evaluate the following items:

- Nutrient Treatment Facilities Plan
- Increased Co-Digestion Facilities Plan and Rate Study
- Energy Demand Reduction Projects

10.3.2.1 Nutrient Treatment Facilities Plan

As discussed previously in this Chapter, additional analysis will be necessary to further define the specific facilities that will be necessary (if any) to comply with the 2024 Nutrient Watershed Permit. Key studies needed to develop a facilities plan include:

- Confirm Treatment Requirements
- Select Preferred Treatment Technology
- Evaluate Return Stream Treatment Needs

10.3.2.1.1 Confirm Treatment Requirements

The facilities that will ultimately be required to meet nutrient requirements for discharges to the San Francisco Bay cannot be defined until more information is available regarding how the nutrient limitations will be applied in the 2024 Nutrients Watershed Permit. The key factors that could have a significant impact on the planning costs provided in this Master Plan are as follows:

- **Constituents of Concern:** The treatment facilities required will be dictated by what specific water quality parameters must be addressed (i.e., the constituents of concern). The analysis presented in this Master Plan assumes that the nutrient limitations adopted in 2024 will include requirements for ammonia, nitrate, and phosphorus. As such, the project includes expanded aerations basins and blower capacity (to address ammonia), installation of an anoxic basin upstream of the aeration basins and an internal mixed liquor recycled system (to address nitrate), and installation of an anaerobic basin upstream of the anoxic basins. However, if one or more of these parameters is not identified as a constituent of concern, then the treatment footprint (and cost) would be lower than the estimates provided herein.

- **Load-Based Versus Concentration-Based Limits.** The facilities layout and costs will also be impacted by whether the limitations are applied as load-based limitations or concentration-based limitations. The analysis presented in this Master Plan assumes that concentration-based limits will be required, thus requiring a high level of conservatism in the design because the facilities will need to consistently meet an effluent target at a wide range of flow and temperature conditions. However, if load-based limits are applied, the WWTP can be designed with less conservatism. Under this scenario, the design approach could allow for higher nutrient effluent concentrations during high flow events or extremely cold temperatures. In addition, depending on the load reductions that are required, expanded reuse or treatment of high-strength return streams from the biosolids dewatering systems could be viable compliance strategies.
- **Seasonal Versus Year-Round Limits.** If the nutrient limitations are applied only in the summer months (i.e., May to September), the District may not be required to build any improvements to remove nutrients. As discussed in the Wastewater Storage Assessment (TM No. 1, which is included as Appendix C), the volume of wastewater discharges between May and September are expected to be minimal by 2035 due to recycled water demands (see Table 16 of TM No. 1). Therefore, if load-based seasonal limitations are applied, then the District may be able to comply with the limitations simply by implementing the planned reuse facility expansion projects. On the other hand, if concentration-based seasonal limitations are applied, the most cost-effective compliance strategy might involve constructing some storage to hold flows that cannot be reused during the periods when limitations are applied.

10.3.2.1.2 Select Preferred Treatment Technology

As discussed in Chapter 6, a more conventional approach to nutrient removal that relies on current industry standards has been applied in this Master Plan. Relying on conventional industry standards for nutrient removal is the most conservative in terms of construction cost and space requirements, and thus provides a reasonable basis for long-term planning. Moreover, while there are some promising nutrient removal technologies emerging, projecting which of these technologies will be the most cost-effective in the timeline anticipated for the District's project is not possible.

Additional evaluation of the preferred nutrient removal approach is recommended once the limitations are more clearly defined. Several technologies that are currently emerging for wastewater treatment and nutrient removal will have time to further develop during this period. A list of promising technologies that should be considered as part of the future evaluations are provided in Table 10-6.

Table 10-6. Emerging Nutrient Removal/Recovery Technologies

Technology	Application at DSRSD	Pros	Cons
Anaerobic Membrane Bioreactor (AnMBR)	Nutrient removal and energy recovery	• Full-scale installations (outside the US) • Compact footprint • High water quality	• Lack of installations in the US • High capital costs
Membrane Aerated Biofilm Reactor (MABR)	Nutrient removal	• High O2 transfer efficiency • Compact footprint • Low loading on secondary clarifiers	• Unproven (lack of installations) • High unit cost
Granular Activated Sludge (i.e., Nereda®)	Nutrient removal	• Compact footprint • Energy efficient • Carbon efficient • Demonstrated to be a viable strategy outside the US • Low unit cost	• Emerging in the U.S. (no current installations) • Applied as a sequencing batch reactor processes, and conversion of conventional activated sludge processes has not been cost-effectively demonstrated • Proprietary process
Mainstream Deammonification	Nutrient removal	• Compact footprint • Energy efficient • Carbon efficient	• Emerging (few installations worldwide) • Complex controls • Low growth rate of anammox bacteria (long recovery if upset) • Could require side-stream treatment

Nereda® Technology is a patented and proprietary technology for biological treatment of wastewater with aerobic granular biomass, branded with the registered trade name Nereda® which is owned by HaskoningDHV Nederland B.V. The Nereda® Technology was first implemented as a full-scale process in 2006, and there are currently 30 plants worldwide that use this technology. While this is a very promising option, key issues that need to be addressed with this approach are how to apply this technology at plants that were designed for conventional activated sludge, not Sequencing Batch Reactors. A similar issue with converting the existing plant would apply for the AnMBR and MABR options.

One particularly promising technology for efficient nitrogen removal in a conventional activated sludge plant is mainstream deammonification. Deammonification is often referred to as Nitrogen Removal 3.0, whereas the conventional NDN approach applied in this Master Plan is considered Nitrogen Removal 1.0. Additional details regarding nitrogen removal pathways is provided as Appendix P.

In general, the oxygen and energy demand, carbon demand, solids yields, reactor volume requirements, and alkalinity demands are lower with the more advanced nitrogen removal processes. A comparison of key unit parameters is captured in Table 10-7.

Table 10-7. Comparison of Key Parameters for Nitrogen Removal 1.0, 2.0, and 3.0

Parameter	Units	Nitrogen Removal 1.0 NDN	Nitrogen Removal 2.0 Nitrite Shunt	Nitrogen Removal 3.0 Deammonification
Oxygen Required	lb O_2/ lb NH_4-N Removed	4.6	3.4	1.9
Carbon Required (if acetate used)	lb COD/ lb N Removed	6.6	4.5	0.0
Yield	lb VSS/ lb N Removed	1.9	1.5	0.1
Alkalinity Required	lb Alk as $CaCO_3$/ lb NH_4-N Removed	7.1	7.1	3.6
Solids Retention Time Required	days	5 – 7	Not Applicable	1 – 2

As shown in Table 10-7, Nitrogen Removal 3.0 (or deammonification) is an extremely efficient process, requiring less aeration than conventional nitrification/denitrification. In addition, deammonification does not have a carbon requirement. Therefore, a larger portion of the influent carbon can be directed to the digesters (increasing biogas generation) without impacting nitrogen removal performance. Finally, the reduced solids retention times means the size of the existing aeration basins may not need to change.

Annamox bacteria, which were first discovered in the early 1990's, remove nitrogen via deammonification. However, annamox are very slow growing organisms and they are easily out-competed by the ammonia-oxidizing bacteria (AOB) and nitrite-oxidizing bacteria (NOB) that are prevalent in wastewater treatment plants. In recent years, approaches have been developed to allow for better "selection" of annamox bacteria in wastewater treatment facilities. These advancements have resulted in the development of several off-the-shelf, high-strength return stream treatment units that achieve stable deammonification using annamox bacteria. However, the success of these treatment units has depended on the fact that anammox grow best in warmer waters that have high ammonia concentrations.

Mainstream deammonification, which involves applying annamox bacteria selection techniques that are used in the side stream processes to a mainstream process, is currently being demonstrated at a handful of facilities throughout the world. One common feature of each of these facilities is that they are "seeded" with annamox bacteria that are grown in a high-strength return stream treatment unit. Therefore, as it currently stands, one key feature that is needed for reliable mainstream deammonification process is a high-strength, warm temperature return stream.

Given the timeline anticipated for implementing a nutrient removal project, it is unlikely that the District will have time to implement a pilot project to evaluate an emerging technology once the 2024 Nutrients Watershed Permit is adopted. Therefore, if an emerging technology approach is desired, it may be necessary for the District to implement a pilot project prior to the adoption of the 2024 permit. Another option would be to observe operation of a desired technology at another treatment plant that has conditions similar to those at the WWTP. This latter strategy may be possible if pilot studies currently being proposed by the BACWA are pursued.

10.3.2.1.3 Evaluate Return Stream Treatment Needs

As noted above, unless advances in deammonification treatment are made that eliminate the need to have an onsite source of annamox seed grown from a high-strength waste stream, an evaluation of mainstream deammonification feasibility at the District's WWTP must consider the facilities needed to reliably produce a high-strength return stream for producing the annamox bacteria seed. In addition, it is also possible that treatment of the high-strength return flows from the WWTP's solids dewatering process is a cost-effective approach for meeting the 2024 Nutrients Watershed Permit (either alone, or in addition to, other improvements).

The following factors should be considered when assessing return stream treatment needs:

- FSL return flows have lower ammonia concentrations, cooler temperatures, and more variable flow rates compared to typical return flows from mechanical dewatering systems. Because of these characteristics, deammonification treatment is not a viable treatment strategy for the FSL return flows. Instead, a conventional biological nitrification processes will be needed for treatment if the FSLs remain in operation.
- A nitrification treatment system will have higher operating costs than a deammonification treatment system; however, the nitrification treatment system will provide a high-nitrate concentration return stream that can provide odor control benefits at the WWTP.
- The FSL return flows are currently discharged to one of the WWTP's influent sewers near the southeastern corner of the FSL site. It would be difficult to construct a return stream treatment system at this location. Therefore, FSL return flows would likely need to be re-routed to a treatment system (likely located near the proposed mechanical dewatering building) and a new outfall pipeline between the treatment system and the influent sewer would need to be constructed.

- Deammonification treatment would be a viable approach for the high-strength return stream coming from a mechanical dewatering unit treating digested solids from the digesters or a recuperative thickening process. This is because these flow streams have very high concentrations of ammonia, are typically warm (due to heating in the digesters), and are relatively constant year-round.
- Additional analysis is needed to determine if the water quality conditions of FSL return flows combined with mechanical dewatering return flows are acceptable for deammonification treatment.
- If the main treatment facilities are designed for nutrient removal, returning FSL return flows off peak may be a viable means of providing the required treatment. Moreover, due to available storage capacity, modest flow control improvements could make treatment of FSL return flow (either alone or combined with mechanical dewatering return flow) an economically favorable approach.

Table 10-8 provides a comparison of the key considerations that should be applied when assessing the treatment options for different high-strength return streams.

If mainstream deammonification treatment is pursued, it will likely be necessary to grow and harvest the annamox bacteria using a return stream deammonification treatment process. Two key factors unique to the District's operations will increase overall cost and operational complexity of this approach:

- First, as noted above, the FSL return stream cannot be reliably treated in a deammonification treatment process. Therefore, if mainstream deammonification treatment is pursued *and* the Nutrients Watershed Permit requires treatment of all high-strength return flows (and off-peak returns of FSL flows are not viable), then the District will need to determine if abandoning the FSLs and providing mechanical dewatering of all digested solids is more cost effective than constructing and operating two different return stream treatment processes.
- Second, conveyance facilities will be needed to deliver the annamox bacteria from the return stream deammonification treatment process to the mainstream deammonification treatment. Moreover, a preferred strategy might be to provide return stream treatment at the main WWTP site to minimize the travel time of the annamox bacteria in the conveyance system, and a new conveyance system would be needed to deliver dewatering filtrate from the mechanical dewatering facilities to a treatment process at the main WWTP site. If the FSLs are eliminated, the FSL cap water pipeline could be used to convey either high-strength waste or annamox seed from the DLD site to the main WWTP site. Therefore, the analysis of mainstream deammonification should also consider whether abandoning the FSLs and providing mechanical dewatering of all digested solids is more cost effective than constructing new conveyance facilities to deliver high-strength return stream/annamox bacteria from the DLD site to the main WWTP site.

Table 10-8. Key Considerations for High-Strength Return Flow Treatment

Return Flow	Nitrogen Treatment/Management Options	Alum Addition Considerations
FSL Return	**Option 1:** Add nitrification treatment with a conventional aerobic process. FSL flows would need to be re-routed from their current discharge point to the new treatment system. Nitrate-rich effluent would be discharged to the collection system providing odor-control benefits. **Option 2:** Return nitrogen-rich flows off-peak. If this approach can be demonstrated to adequately mitigate nutrient load impacts on the WWTP, the FSL return facilities could be modified and/or operated differently to allow for off peak return of flows, thus avoiding additional treatment.	• Alum addition facilities installed in the near-term period (as discussed previously in this Chapter) would need to be relocated downstream of the return stream treatment system. • If loads can be mitigated by discharging off-peak, alum facilities installed in the near-term period can continue to be used.
Mechanical Dewatering Return	**Option 1:** Deammonification treatment. Will have a much lower operating cost compared to aerobic treatment; however, effluent from a deammonification treatment system will not provide the same odor control benefits. **Option 2:** Conventional aerobic treatment process.	• Alum addition facilities installed in the near-term period could be used, as is, if FSLs are abandoned or if only the mechanical dewatering return flow is treated. Under both scenarios, effluent from the mechanical dewatering treatment system would be discharged to a portion of the existing FSL return conveyance system. • New alum addition facilities may be needed if there are separate treatment systems for FSL return and mechanical dewatering return. • District could consider installing a phosphorus recovery system for the mechanical dewatering flow. (See discussion later in this Chapter.)
FSL and Mechanical Dewatering Return Combined	• Additional analysis needed to confirm whether deammonification treatment or conventional nitrification treatment is desired. • Regardless of treatment approach, the system would need to be located downstream of FSL and mechanical dewatering systems, thus the FSL flows would need to be re-routed.	• Alum addition facilities installed in the near-term period will need to be relocated downstream of the new return treatment system. • Depending on combined flow characterization, phosphorus recovery may be possible.
Recuperative Thickening Return	• Same treatment options as mechanical dewatering return stream treatment • Treatment facilities would likely need to be located within existing holding basins due to limited space at the WWTP.	• Install separate alum facilities for the recuperative thickening return. • Treatment facilities would likely need to be located within existing holding basins due to limited space at the WWTP.

The outcome of a deammonification return stream treatment analysis would be contingent on what (if any) dewatering options have been implemented at the time a deammonification treatment system is needed. Therefore, consideration of potential future deammonification treatment needs could be a consideration when selecting a preferred dewatering approach. The emphasis that should be placed on this consideration will depend on the timing of the dewatering project and whether mainstream deammonification treatment is demonstrated to be a viable approach for nitrogen removal within this timeframe.

Finally, another strategy that could also be considered for creating a high strength waste stream at the main WWTP site would be to install a recuperative thickening process on one or more of the digesters, which would effectively remove some of the digester liquid prior to sending the solids to the FSLs. This strategy has the added benefit of increasing the digestion capacity, a benefit that may be desired if organics waste management becomes a major feature of the WWTP.

Additional discussion of the various approaches for return stream treatment under a future mainstream deammonification treatment strategy is presented in Appendix Q.

10.3.2.2 Increased Co-Digestion Facilities Plan and Rate Study

The Master Plan includes the implementation of a hauled waste co-digestion program at the WWTP during the mid-term period. Prior to implementing this project, the District will need to complete a comprehensive evaluation to better define the project facilities and required tipping fee. The most important factor of this analysis will be the source of feedstock to be received, as this factor governs all other evaluation factors including:

- The amount of biogas that is expected per wet ton received
- The mass of additional biosolids that will be generated
- The type of facilities needed at the WWTP to pre-treat the hauled materials (if any)
- The facilities needed to control odors at the WWTP

Given the importance of feedstock source, the District will be well served by conducting independent research/marketing for desirable feedstock materials to identify the available feedstock(s) that would result in the greatest benefit to the District's operations. Potential sources include: solids waste management agencies, food processing industries and industrial waste haulers. Moreover, due to the WWTP location at the intersection of two major interstate freeways, it is not unreasonable to expect that several potential opportunities may be available.

Given the WWTP's size, and its unique odor control and biosolids management needs, careful evaluation of several potential hauled organic waste feedstock is recommended. This evaluation should consider for each potential feedstock type: the hauled waste receiving facility needs (including odor control), WWTP staff operational demands, potential WWTP process impacts, and potential for increases (or decreases) in biosolids production. This information can then be used to define the tipping fee required for each feedstock type to make hauled waste co-digestion financially sustainable for the District. A potential organic hauled waste stream would be considered viable if the tipping fees required by the District are reasonable for the hauler(s) and there is sufficient certainty in the long-term reliability of the feedstock source(s).

10.3.2.3 Energy Demand Reduction Projects

The District should continue to evaluate the opportunities for energy demand reductions at the WWTP that were presented in Chapter 9 during each phase of implementing the CIP. If not completed previously, operational changes that should be evaluated in the mid-term period are:

- Optimizing aeration blower header pressure; even a small reduction in pressure could reduce costs significantly.
- Reducing pressure of air compressor for utility and air scour air for the moving bed sand filters.
- Optimizing the 3W system to reduce the amount of total flow pumped. The most potential savings would be realized if the amount of flow used to remove waste heat from the cogeneration system could be reduced.
- Optimizing mixing energy for anaerobic digesters by cycling the mixers to the extent feasible.

10.3.3 Mid-Term Monitoring Activities

During the mid-term period, the District should continue to monitor some of the potential future constraints and opportunities for the WWTP that were discussed for the near-term implementation period. For the mid-term period, the following specific emerging regulations and opportunities are expected to be of interest:

- Raw Water and Treated Water Augmentation
- Energy Management Opportunities

10.3.3.1 Raw Water and Treated Water Augmentation

There are a number of issues and uncertainties regarding implementation of raw water and treated water augmentation in California. Nevertheless, regulations for raw and treated water augmentation are likely to be adopted within the next ten years, or by the beginning of the long-term implementation period. Assuming the regulations are adopted in this timeframe, the District's activities during the mid-term implementation period will involve monitoring the State's activities with respect to raw water/treated water augmentation regulation adoption, as well as the evolution of emerging technologies.

It is possible that raw water and treated water augmentation requirements will be defined in the mid-term period. If this were to occur, the District would have the information needed to fully evaluate a potential project during the mid-term. Additional discussion regarding the efforts required is provided in the Long-Term Evaluations section of this Chapter.

10.3.3.2 Energy Management Opportunities

It is anticipated that equipment and technologies available for processing and extracting energy from organic materials will continue to evolve rapidly over the next five to ten-year period. In addition, solar and battery technologies are expected to continue to evolve rapidly. Therefore, the District staff should continue to stay abreast of new technologies to be in the best position to take advantage of emerging opportunities in energy generation when an expansion of onsite generation capacity is desired or needed.

The preferred energy generation expansion strategy will depend heavily on the anticipated long-term WWTP energy demands. As discussed in Chapter 9, the WWTP energy demands are expected to increase significantly with the implementation of a CWR rehabilitation project and/or a nutrient removal project in the mid-term implementation period. However, the anticipated increases presented in Chapter 9 are based on current technology assumptions like conventional nitrification/denitrification for nutrient removal and RO for potable reuse treatment. If technologies like mainstream deammonification for nitrogen removal or ozone plus BAF for potable reuse treatment demands are determined to be viable, the ultimate WWTP demands could be lower than what is projected in this Master Plan.

The quantification of the anticipated long-term energy demands will evolve over the mid-term evaluation period. Therefore, it is recommended that the District wait until there is additional clarification regarding future treatment needs prior to making a significant investment in new generation capacity.

10.4 LONG-TERM IMPLEMENTATION STRATEGY (FY 26/27 TO FY 35/36)

The Master Plan implementation strategy for the long-term period (2027 to 2037) is presented below. Recommendations are presented for facilities that should be designed and constructed, studies or evaluations that should be completed, and emerging issues and topics that should be monitored. Figure 10-3 provides a schedule overview of the recommended implementation plan for the entire 20-year planning period.

10.4.1 Long-Term Design/Construct

The primary activity that is expected to occur in the long-term period is a nutrient removal project to meet the requirements for discharge to the San Francisco Bay. The timing of other improvements to be completed in this time frame will depend on the outcome of studies completed in the near-term and mid-term periods, as well as factors outside the District's control, like new restrictions on the DLD or FSL facilities. The projects planned for the long-term implementation period are summarized in Table 10-9. The anticipated years when design and construction would be initiated are also shown.

Table 10-9. Anticipated Long-Term Construction Projects

Project	Recommended Timing	Begin Design/ Environmental Review	Begin Construction
Liquid Treatment			
Nutrient Removal	Driven by the 2024 Nutrient Watershed Permit	FY 26-27	FY 28-29
Potable Reuse			
Raw Water/Treated Water Augmentation	The timing of this project will depend on when regulations for treated water augmentation are adopted. At this time, it is anticipated that facilities would be in place by the end of the 20-year planning period.	FY 31-32	FY 33-34
Solids Treatment			
Full Biosolids Dewatering	Timing will depend on either 1) a regulatory demand to cease or alter use of FSLs or DLD site either due to odor, groundwater impacts, or greenhouse gas issues; or 2) increased favorability of biosolids market conditions. Timing shown herein assumes project will be implemented between the nutrient removal project and the treated water augmentation project.	FY 29-30	FY 30-31

10.4.2 Long-Term Evaluations

During the long-term implementation period, the District may need to further evaluate the following items:

- Potable Reuse Expansion
- Enhanced Primary Treatment
- Opportunities for Biosolids Reuse/Nutrient Recovery
- Energy Generation Expansion Plans
- Energy Demand Reduction Projects

10.4.2.1 Potable Reuse Expansion

As discussed previously, the District will decide in the near-term implementation period whether to proceed with a regional potable reuse project or with a District-only project at the WWTP. If a regional approach is pursued, the timing and implementation strategy will be defined by a potentially wide-range of regional factors that are beyond the scope of this Master Plan. Therefore, the discussion presented herein is based on an approach where the District decides to lead implementation of a potable reuse project at the WWTP site.

If a District-driven project is pursued, the initial phase would involve rehabilitation of the CWR facilities. Moreover, the rehabilitated CWR facilities would likely either be used as a demonstration/training facility or be used to produce water for augmentation of the Chain of Lakes, which are operated by Zone 7 Water Agency and are planned to supply the Del Valle WTP. The decision regarding the near-term strategy should also be based on the District's intended long-term goals to the extent they can be defined in the near-term period.

Potable reuse regulations and treatment technology strategies are developing rapidly. Therefore, decisions regarding long-term potable reuse will need to be revisited after the near-term project is implemented and more information becomes available regarding regulations and treatment options. Moreover, assuming the ultimate project involves upgrading the treatment system to provide for raw water or treated water augmentation, there could be significant risks for the District in moving forward with a raw water or treated water augmentation project ahead of the adoption of statewide regulations for these uses. Specifically, the raw water/treated water regulations will define both the level of treatment required as well as the operating requirements. These factors can have a significant impact on both construction and operating costs, so the District would risk making a premature decision regarding project feasibility. As noted previously, the timing of when raw water or treated water augmentation regulations are adopted is uncertain. At this time, it is expected that these regulations will be adopted within the next ten years, which would put adoption near the beginning of the long-term implementation period.

The District should also plan to operate the near-term facilities for a few years before making a final decision regarding the second phase of the project (if any). This operating experience will provide the information needed to better define the long-term costs for potable reuse. Given the significant investments required for the long-term potable reuse project(s), well defined operating costs are critical to understanding the long-term commitments being considered. Assuming the near-term project is implemented in the mid-term implementation period, it is expected that the first few years of the long-term implementation period will be used by the District to gain valuable operating experience with the facilities constructed in the initial phase.

Given the need for both statewide regulations to be defined and operating experience developed, a study to define the preferred long-term strategy should occur in FY 30/31 and/or FY 31/32. Additional discussion regarding factors to be evaluated as part of this effort were discussed in the near-term implementation period section of this Chapter.

10.4.2.2 Enhanced Primary Treatment

The current primary sedimentation basin improvements project is expected to achieve at least 65 percent TSS removal in the primary sedimentation basins. However, with additional improvements, it would be possible to achieve even better primary treatment performance. With enhanced performance, more influent solids would be sent to the digesters, thus increasing gas production and reducing energy consumption in the aeration basins. However, depending on the nutrient removal requirements needed in the future, the organics that would be removed in the primaries under an enhanced treatment scenario may be needed to support downstream biological denitrification.

As shown in the Recommended Plan Implementation Schedule provided as Figure 10-3, there is expected to be an approximate four-year period when the primary sedimentation expansion project is complete and the District will need to start evaluating nutrient removal. Given this relatively short timeframe, it is not recommended that the District further evaluate enhanced primary treatment until there is a more definitive understanding of the long-term nitrogen removal treatment requirements.

There are two conceivable nutrient removal scenarios under which enhanced primary treatment could be implemented:

1. Nutrient limitations are applied only for the summer months, and the District determines compliance is achievable without upgrading the WWTP.
2. A nitrogen removal strategy that relies on deammonification (or possibly some other emerging technology) is identified as viable, thus eliminating the need for carbon to support nitrogen removal.

If the District determines that enhanced primary treatment can be implemented without disrupting other treatment goals, an evaluation should be completed to assess potential options for achieving this objective. The evaluation should consider the following factors:

- Capital and operating costs associated with potential alternatives for enhanced primary treatment.
- Potential for increases in biosolids treatment, handling, and disposal costs if chemical addition is used to enhance performance.
- Cost savings associated with reduced carbon loadings to the aeration process.
- Comparison of biogas production benefits to benefits received from other potential organic feedstocks, as digester capacity is limited and additional primary solids may not provide the highest value use of the digestion capacity.

10.4.2.3 Opportunities for Biosolids Reuse/Nutrient Recovery

As discussed previously in this Chapter, the District will need to conduct market research to assess possible offsite disposal options for the dewatered biosolids, and a preferred approach for dewatered biosolids management will need to be selected when implementing the partial biosolids dewatering project in the mid-term. However, as technologies for recovering the energy and nutrient content from biosolids advance, new local opportunities for reuse of the WWTP biosolids could arise. Thus, it is recommended that the District continue to evaluate new and emerging opportunities for reducing biosolids handling costs by putting the WWTP biosolids to higher and better use(s). A list of technologies that are emerging for biosolids management/reuse that should be monitored was provided previously in Table 10-4.

A related opportunity of interest that should periodically be evaluated as part of these biosolids reuse evaluation efforts is phosphorus recovery. Phosphorus is a non-renewable resource that has an important value for humans with respect to growing food. As natural phosphorus resources are depleted or phosphorus extraction from mines becomes costlier, the value of phosphorus recovered from wastewater will increase. In addition, technologies for phosphorus recovery from wastewater are evolving, and it is likely that the costs for recovery will decrease over time.

As documented in Chapters 6 and 8, the economics of phosphorus recovery are not likely to be promising for the WWTP in the near or mid-term periods. The following specific topics related to this issue are presented:

- As discussed in Chapter 6, a WAS stripping process could be used to recover phosphorus from the combined primary solids/activated sludge flow stream prior to thickening in the DAFT. The WAS stripping process causes the phosphorus bound in the solids to solubilize, allowing for it to be separated from the solids in the DAFT. This high-strength liquid stream could then be directed to a phosphorus recovery process. However, for this approach to be viable, the WAS stripping and phosphorus recovery processes would need to be located at the main WWTP site, where available space is limited.

- As discussed in Chapter 8, phosphorus levels in the FSLs return flows are relatively low, and previous investigations completed by the District indicate concentrations are not favorable for economically-advantageous phosphorus recovery.
- While it is possible that the efficiency of phosphorus recovery systems will improve in the future, the economics are likely to be much more favorable if the WWTP included full dewatering from the digesters.
- Current technologies available for phosphorus recovery have been demonstrated to be economically attractive only for much larger facilities than the WWTP. The typical cut-off is at approximately 40 mgd ADWF permitted capacity plants.

After the mid-term period, the District will have the capability to dewater a portion of the biosolids (potentially from the digesters), thus making phosphorus recovery from the return stream more efficient. In addition, the cost of phosphorus recovery facilities is likely to decrease, and the price of phosphorus is likely to increases over the next ten years. Therefore, with these combined factors, it is possible that phosphorus recovery could become an opportunity for creating a positive revenue stream in the long-term period. The District therefore should continue to appraise the potential for implementing phosphorus recovery at the WWTP as part of ongoing assessment of biosolids reuse potential.

If a particularly compelling opportunity for biosolids reuse or phosphorus recovery arises that would result in a lower net cost (after considering revenue streams) than the current FSL/DLD operation, this could be a driver to move toward a full biosolids dewatering approach. On the other hand, if other factors drive the elimination of use of the DLDs or FSLs, the total mass, nutrient content, or volatile solids content of the dewatered biosolids could change. Any one of these factors could potentially open the door for new biosolids reuse/nutrient recovery opportunities that were not previously viable.

Despite the potential benefits of biosolids reuse or phosphorus recovery, costs are not included in the Master Plan CIP for implementing a biosolids reuse/phosphorus recovery project. It is also expected that capital costs invested to allow biosolids reuse or phosphorus recover could be recuperated over time through increased revenues and/or decreased operating costs.

For purposes of this Master Plan, it is assumed evaluations related to biosolids reuse/phosphorus recovery would be completed as part of the full dewatering project implementation. However, as discussed previously in this Chapter, it is possible that a full dewatering project will not be implemented within this Master Plan planning period. If this were the case, the District should still plan to evaluate biosolids reuse and/or phosphorus recovery during the long-term implementation period, as it is expected that there will be technological advancements (or other factors) that would make these approaches more economically justifiable.

10.4.2.4 Energy Generation Expansion Plan

The long-term WWTP energy demands should be adequately defined near the beginning of the long-term implementation period to support decisions regarding onsite energy generation expansion needs. In addition, technologies for onsite generation are expected to have evolved significantly by the beginning of the long-term implementation period. Therefore, it is recommended that early in the long-term implementation period the District re-evaluate the findings presented in Chapter 9 of this Master Plan regarding expansion of the onsite energy generation facilities to define a preferred strategy.

The District is currently installing a FOG receiving facility at the WWTP, and it is recommended that a hauled organics waste receiving facility be installed during the mid-term implementation period. These facilities will allow the District staff to evaluate during the near-term and mid-term periods whether the adverse impacts of a hauled waste receiving program can be reasonably mitigated. If District operations staff determine that the impacts from a hauled waste program can be mitigated, it may be desirable to expand the onsite biogas energy generation capacity by installing recuperative thickening (or possibly other strategies) for increasing digestion capacity and/or installing additional/replacement engines to increase cogeneration capacity.

A large solar power project is another energy generation strategy that should be considered if a full dewatering project is implemented, freeing the DLD site for use as a solar field. As discussed in Chapter 9, a significant portion of the potential WWTP energy demands could be met with a large solar project on the DLD site.

A third strategy of interest is the use of batteries to balance WWTP energy generation capacity with peak WWTP demands. Specifically, as large batteries become less expensive over time, it may become cost effective to use the cogeneration facilities to charge batteries during off-peak operating periods, so that the stored energy could be used for WWTP operations during peak electricity demand periods.

Ultimately, expansion of the onsite energy generating capacity should only move forward if the economics are determined to be favorable. However, as plant energy demands and electricity costs increase and co-digestion/co-generation technologies evolve, the economic favorably of onsite energy generation from digestion of organic feedstocks will likely improve.

10.4.2.5 Energy Demand Reduction Projects

As discussed previously, the District should continue to evaluate opportunities for energy reductions throughout the Master Plan planning period. One opportunity that should specifically be evaluated in the long-term implementation period is the potential replacement of the DAFT with a lower energy-use thickening process. This DAFT replacement evaluation should consider the following factors:

- Whether the WWTP solids thickening needs have changed due to changes in the secondary treatment process,
- The difference in cost between DAFT rehabilitation and replacement with other solids thickening technologies available,
- The space requirements for the DAFT versus other potential approaches,
- Benefits of providing system redundancy with an alternative (and smaller) thickening approach versus the current approach of providing redundancy using a portable thickening unit,
- Potential energy savings associated with alternative thickening options, and
- Differences in operating costs between the DAFT and other technologies, such as: chemical use, operations time, and maintenance needs.

A lifecycle cost analysis developed with this information will inform the District as to how to proceed with potential DAFT replacement.

An evaluation of the DAFT system should occur when DAFT equipment reaches the end of its useful life. The DAFT system was recently rehabilitated; therefore, it is expected to reliably serve the WWTP's solid thickening needs for the next 15 to 20 years. Thus, the recommended DAFT evaluation would occur around FY 30/31. Another possible driver for evaluating the DAFT is a change in thickening needs due to secondary process changes associated with required nutrient removal upgrades. As shown in Figure 10-3, evaluation of nutrient removal options is expected to begin in FY 25/26.

Finally, whether rehabilitation or replacement, a future DAFT project is considered a maintenance project and is thus included in the District's rehabilitation and replacement budget. Therefore, the cost of this project is not included in the Master Plan CIP.

10.4.3 Long-Term Monitoring Activities

During the long-term period, the District should continue to monitor potential future constraints and opportunities for the WWTP. The following specific emerging regulations and opportunities are expected to be of interest:

- Nutrient Regulations
- Nutrient Treatment Technologies
- Potable Reuse Expansion Opportunities

10.4.3.1 Nutrient Regulations

The Regional Board intended to adopt a revised Nutrient Watershed Permit in both 2029 and 2034. Thus, regulations for nutrient discharges to the San Francisco Bay will continue to evolve over the long-term implementation period, and District staff should continue to remain informed of ongoing Regional Board and/or BACWA activities regarding the Nutrient Watershed Permit throughout the long-term implementation period.

It is unlikely that the 2029 Nutrient Watershed Permit will include more stringent regulations than the 2024 permit, because it will take several years to assess whether improvements implemented due to the 2024 Nutrient Watershed Permit are effective in meeting the desired water quality goals for the San Francisco Bay. It is possible, however, that new requirements could be adopted in the 2034 Nutrient Watershed Permit.

10.4.3.2 Nutrient Treatment Technologies

As the potential for additional nutrient treatment requirements are clarified, the District should also consider options for using emerging treatment technologies to achieve the anticipated nutrient reduction requirements. If a technology is particularly promising, the District could also consider pilot testing at the WWTP to assess viability. The need for, and timing of, any additional evaluations would depend on several factors, including: changes in regulations that occur between 2024 and 2034, activities completed by the consortium BACWA agencies to evaluate emerging technologies, and the status of various studies being completed across the country to develop new and emerging technologies.

10.4.3.3 Potable Reuse Expansion Opportunities

As discussed previously in this Chapter, regulations for raw water and treated water augmentation are likely to be adopted by the beginning of the long-term implementation period, and the Master Plan CIP includes a project to expand the CWR facilities to provide for higher-level end uses. However, the CIP does not include expansion of the potable reuse treatment facilities beyond the 3.0 mgd capacity of the CWR facilities.

There are several factors that could change during the long-term implementation period that could make expansion of potable reuse treatment at the WWTP beyond the capacity of the CWR facilities more economically viable. These factors include:

- Availability of other summer-month water supplies sources for the potable reuse treatment system (e.g., dry-season runoff from urban areas, treated effluent from the City of Livermore wastewater plant, etc.)
- Reduction in current or anticipated water supplies due to increased drought conditions brought on by climate change
- Changes to Zone 7 for drinking water production costs due to increasing regulation or changes in supply water quality
- Reduced potable reuse treatment costs due to technological advancements, such as ozone plus BAC as a replacement for RO treatment

Therefore, the District's should continue to monitor each of the changes/opportunities listed above to determine if expanded potable reuse should be considered.

10.5 CAPITAL IMPROVEMENT PROGRAM

A CIP for the WWTP recommendations identified in this Master Plan is presented in Table 10-10. As reflected in this table, the CIP is based on project costs that were presented in the previous Master Plan chapters in combination with the implementation schedule shown on Figure 10-3.

The CIP presents the project costs separated into the efforts anticipated for further evaluations, for design and environmental review, and for construction. As discussed in Chapter 5, the non-construction costs (those costs not included in the construction contract and construction contingency), are predicted as 35 percent of the estimated construction costs. To develop the CIP, these non-construction costs were allocated to evaluation and design efforts in accordance to when the costs are expected to occur.

Table 10-10. Estimated Project Costs for Recommended CIP

Item	Master Plan Ref.	Project Costs, million $	Estimated Project Costs, million $																	
			Near-Term Implementation Period				Mid-Term Implementation Period					Long-Term Implementation Period								
			FY 17/18	FY 18/19	FY 19/20	FY 20/21	FY 21/22	FY 22/23	FY 23/24	FY 24/25	FY 25/26	FY 26/27	FY 27/28	FY 28/29	FY 29/30	FY 30/31	FY 31/32	FY 32/33	FY 33/34	FY 34/35	FY 35/36
LIQUID TREATMENT																				
Primary Effluent Diversion and Hydraulic Improvements	Tables 6-2 and 6-3	**0.35**																		
Design & Environmental Review		0.05	0.025	0.025																
Construct		0.30		0.30																
Primary Treatment Improvements/Expansion	Table 6-5	**9.7**																		
Design & Environmental Review		1.3	1.1	0.3																
Construct		8.4		4.2	4.2															
Alum Addition	Table 6-10	**0.75**																		
Evaluate		0.02			0.02															
Design & Environmental Review		0.10			0.05	0.05														
Construct		0.63				0.63														
Nutrient Removal Improvements[a]	Table 6-17	**42.8**																		
Evaluate		1.6								0.79	0.79									
Design & Environmental Review		4.8										2.4	2.4							
Construct		36.4												14.6	14.6	7.3				
POTABLE REUSE																				
Rehabilitation of CWR Facilities[b]	Table 7-5	**35.6**																		
Evaluate		1.1						0.55	0.55											
Design & Environmental Review		4.1							1.4	2.7										
Construct		30.4									15.2	15.2								
Treated Water Augmentation[c]	Table 7-11	**44.7**																		
Evaluate		1.5														1.0	0.49			
Design & Environmental Review		5.1															1.3	2.6	1.3	
Construct		38.1																	7.6	15.2	15.2
SOLIDS TREATMENT																				
Anaerobic Digester Expansion[d]	Table 8-3	**7.7**																		
Construct		6.7	5.4	1.3																
Partial Biosolids Dewatering[e]	Table 8-9	**19.0**																		
Evaluate		0.9		0.35	0.35	0.17														
Design & Environmental Review		2.1				0.7	1.4													
Construct		16.1						8.0	8.0											
Full Dewatering Expansion[f,g]	Table 8-15	**5.7**																		
Evaluate		0.2												0.20						
Design & Environmental Review		0.7													0.68					
Construct		4.8														4.8				

Table 10-10. Estimated Project Costs for Recommended CIP

Item	Master Plan Ref.	Project Costs, million $	Estimated Project Costs, million $																	
			Near-Term Implementation Period				Mid-Term Implementation Period					Long-Term Implementation Period								
			FY 17/18	FY 18/19	FY 19/20	FY 20/21	FY 21/22	FY 22/23	FY 23/24	FY 24/25	FY 25/26	FY 26/27	FY 27/28	FY 28/29	FY 29/30	FY 30/31	FY 31/32	FY 32/33	FY 33/34	FY 34/35	FY 35/36
ENERGY MANAGEMENT																				
FOG Receiving Facility[h]	Section 9.4.1.1	**1.2**																		
Construct		1.0	1.0																	
Biogas Treatment System[i]	Table 9-14	**4.0**																		
Design & Environmental Review		0.5	0.51																	
Construct		3.5		3.5																
Hauled Organic Waste Receiving Facility[j]	Section 9.4.2.1	Unknown																		
Evaluate							Unknown													
Design and Environmental Review								Unknown												
Construct									Unknown											
ODOR CONTROL																				
Existing Facility Improvements	Appendix D, Table 1	**1.0**																		
Evaluate, Design, and Environmental Review		0.12			0.12															
Construct		0.88				0.88														
Odor Control Expansion	Appendix D, Table 1	**5.3**																		
Evaluate, Design, and Environmental Review		0.6							0.6											
Construct		4.5								4.5										
Total			8.0	9.6	4.7	2.4	1.4	8.7	10.6	8.0	16.0	17.6	2.4	14.8	15.2	13.1	1.8	2.6	8.9	15.2	15.2

[a] Costs are based on BACWA Level 2, assuming the Alum Addition for Year-Round Anaerobic Selector Operation Progression.

[b] Costs are based on the Ozone + BAC pretreatment alternative.

[c] Costs are based on a 3.0 MGD Advanced Potable Reuse for Treated Water Augmentation System, assuming the Ozone + BAC pretreatment alternative for the 3.0 MGD Potable Reuse System was previously implemented. However, because the Full-Scale NDN facilities are expected to be constructed for Nutrient Removal Improvements, the cost of these facilities are not included in the potable reuse project cost. If Full-Scale NDN facilities are not constructed, then approximately $36.2 million should be added to this estimate.

[d] Fourth Digester was designed in the 2016-2017 fiscal year. Design costs not shown.

[e] Costs are based on the Dewater Portion of Digested Solids for Offsite Disposal Year Round alternative.

[f] Costs are based on the Full Dewatering from the Digesters alternative, assuming the Dewater a Portion of Digested Solids for Offsite Disposal Year Round alternative was previously implemented.

[g] Biosolids reuse and/or phosphorus recovery are assumed to be evaluated as part of the Full Dewatering Expansion project. If the Full Dewatering Expansion project is not implemented within the Master Plan period, the District should still evaluate biosolids reuse and/or phosphorus recovery during the long-term implementation period.

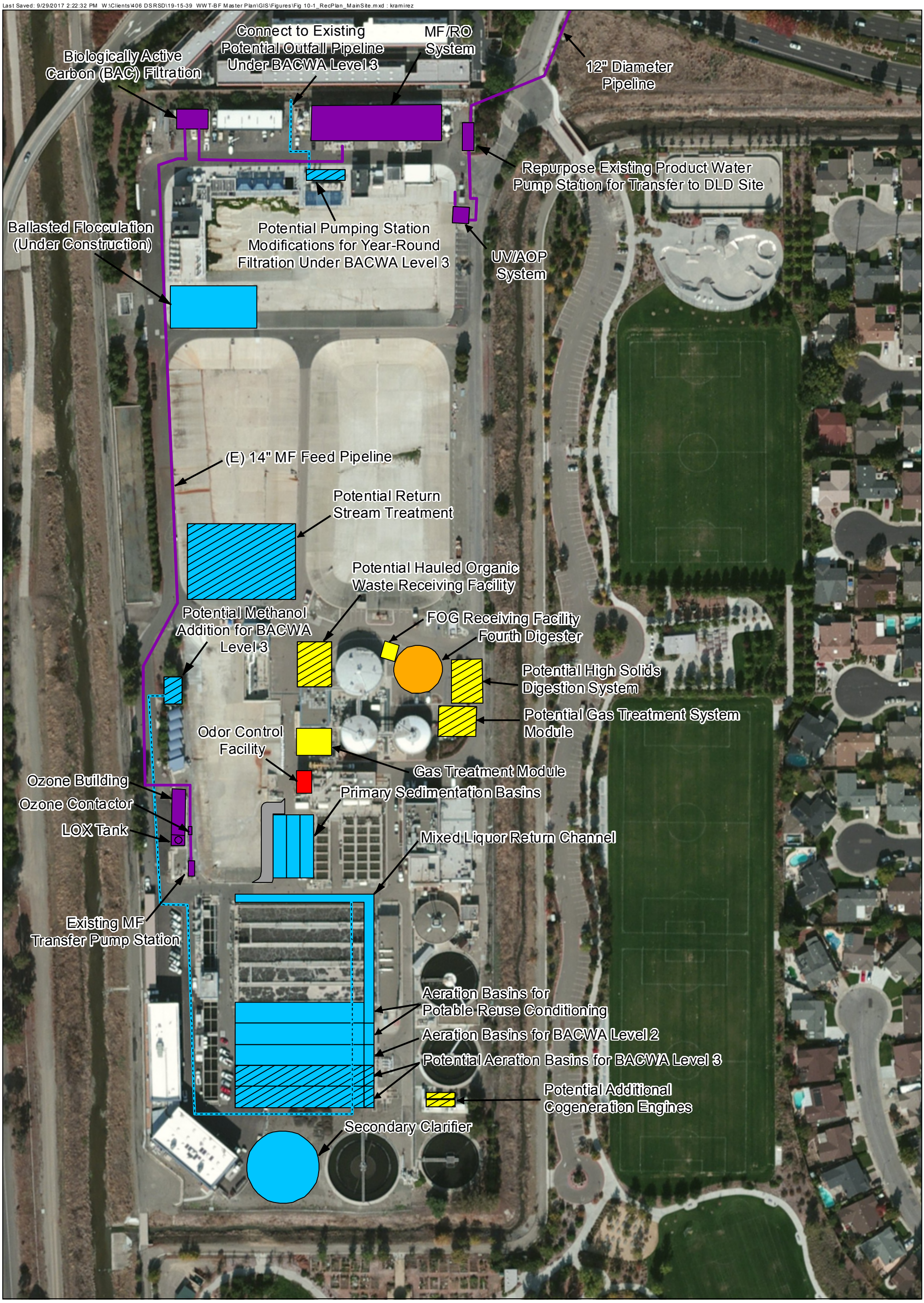
[h] FOG Receiving Station was designed in the 2016-2017 fiscal year. Design costs are not shown.

[i] Gas Treatment System costs include building one, 430 CFM module with two, 110 CFM air compressors.

[j] Costs for a hauled waste receiving facility cannot be reasonably defined until more is understood regarding the type of hauled waste anticipated and other financial terms for the project.

10.6 REFERENCE

City of San Diego, 2017, https://www.sandiego.gov/water/purewater/purewatersd, Accessed on June 15.



Symbology

- Liquid Treatment Facilities
- Potable Reuse Facilities
- Energy Management Facilities
- Solids Treatment Facilities
- Odor Control Facility
- Additional Paved Area

Note: Hatched areas and dashed pipelines represent potential facilities not included in the CIP.

N

0 70 140

Scale in Feet

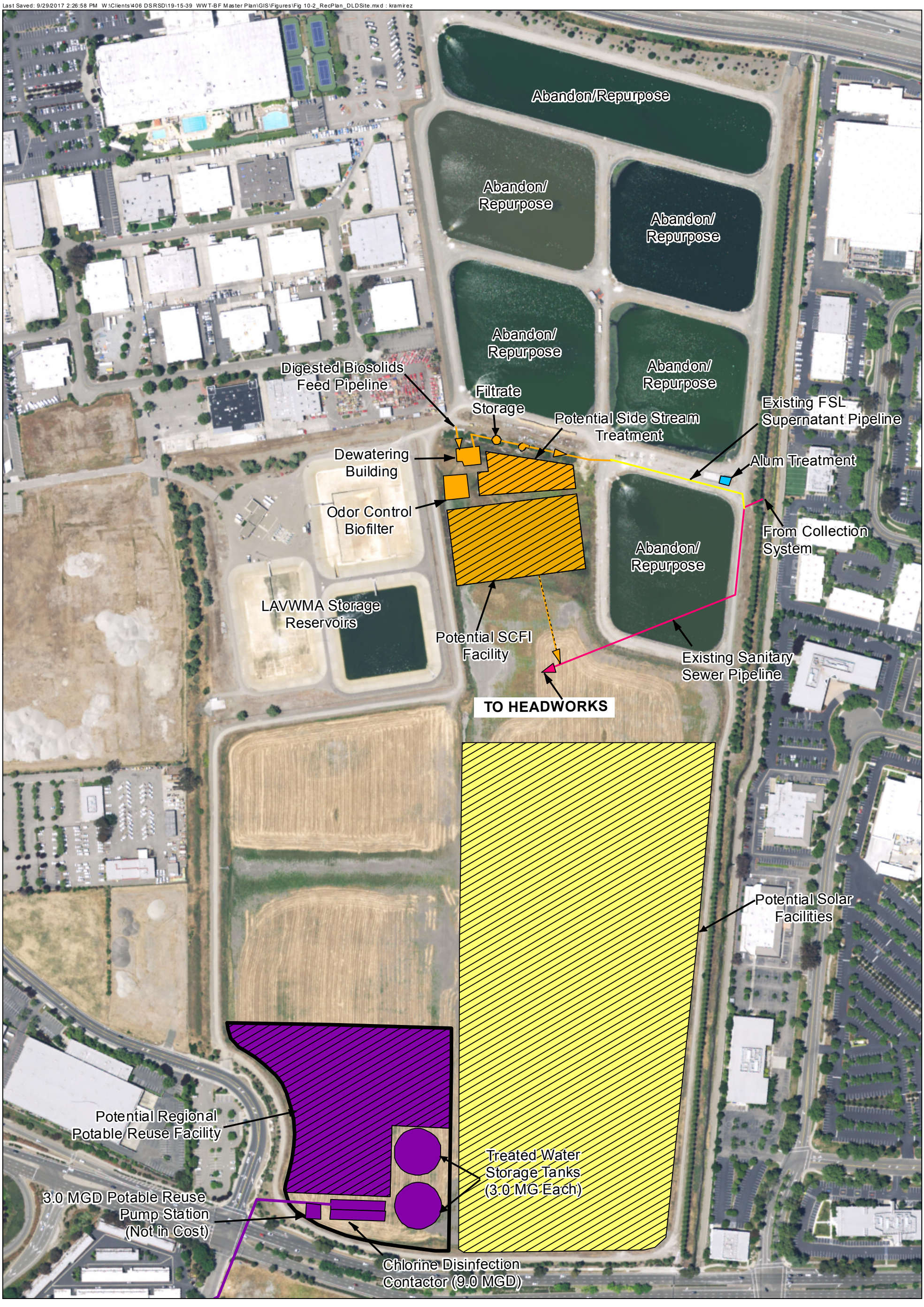
WEST YOST ASSOCIATES

Figure 10-1

Site Layout of CIP Facilities at Main WWTP Site

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

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Symbology

- Liquid Treatment Facilities
- Potable Reuse Facilities
- Solids Treatment Facilities
- Solar Facilities

Note: Hatched areas and dashed pipelines represent potential facilities not included in the CIP.

N

0 140 280

Scale in Feet

WEST YOST ASSOCIATES

Figure 10-2

Site Layout of CIP Facilities at DLD Site

Dublin San Ramon Services District
Wastewater Treatment Plant
Master Plan

Figure 10-3. Recommended Plan Implementation Schedule

Item	Near-Term Implementation Period												Mid-Term Implementation Period												Long-Term Implementation Period																							
	FY 17/18				FY 18/19				FY 19/20				FY 20/21				FY 21/22				FY 22/23				FY 23/24				FY 24/25				FY 25/26				FY 26/27				FY 27/28				FY 28/29				FY 29/30			
	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
LIQUID TREATMENT																																																		
Primary Effluent Diversion and Hydraulic Improvements																																																	
Design & Environmental Review																																																	
Construct																																																	
Primary Treatment Improvements/Expansion																																																	
Design & Environmental Review																																																	
Construct																																																	
Alum Addition																																																		
Evaluate																																																	
Design & Environmental Review																																																	
Construct																																																	
Nutrient Removal Improvements																																																	
Evaluate																																																	
Design & Environmental Review																																																	
Construct																																																	
POTABLE REUSE																																																	
Rehabilitation of CWR Facilities																																																	
Evaluate																																																	
Design & Environmental Review																																																	
Construct																																																	
Treated Water Augmentation																																																	
Evaluate																																																	
Design & Environmental Review																																																	
Construct																																																	
SOLIDS TREATMENT																																																	
Anaerobic Digester Expansion																																																	
Construct																																																	
Partial Biosolids Dewatering																																																	
Evaluate																																																	
Design & Environmental Review																																																	
Construct																																																	
Full Dewatering Expansion																																																	
Evaluate																																																	
Design & Environmental Review																																																	
Construct																																																	

Figure 10-3. Recommended Plan Implementation Schedule

Item	Near-Term Implementation Period												Mid-Term Implementation Period												Long-Term Implementation Period																							
	FY 17/18				FY 18/19				FY 19/20				FY 20/21				FY 21/22				FY 22/23				FY 23/24				FY 24/25				FY 25/26				FY 26/27				FY 27/28				FY 28/29				FY 29/30			
	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
ENERGY MANAGEMENT																																																		
FOG Receiving Facility																																																		
Construct																																																			
Biogas Treatment System																																																		
Design & Environmental Review																																																		
Construct																																																			
Hauled Organic Waste Receiving Facility																																																		
Monitor																																																		
Evaluate																																																		
Design and Environmental Review																																																		
Construct																																																			
ODOR CONTROL																																																		
Existing Facility Improvements																																																		
Evaluate																																																		
Design and Environmental Review																																																		
Construct																																																			
Odor Control Expansion																																																		
Evaluate																																																		
Design and Environmental Review																																																		
Construct																																																			

[a] DLD hydraulic loading limitation will be evaluated in the near term. Based on the need to address the DLD hydraulic loading limitation, the timing for design, environmental review, and construction of Partial Dewatering facilities may be adjusted.

[b] Biosolids reuse and/or phosphorus recovery are assumed to be evaluated as part of the Full Dewatering Expansion project. If the Full Dewatering Expansion project is not implemented within the Master Plan period, the District should still evaluate biosolids reuse and/or phosphorus recovery during the long-term implementation period.