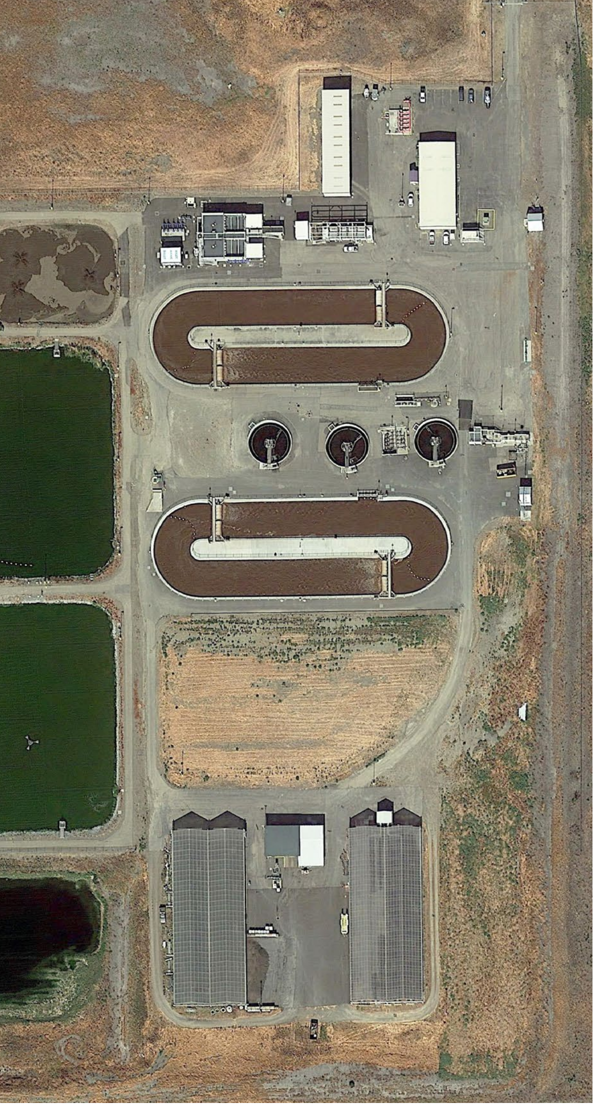

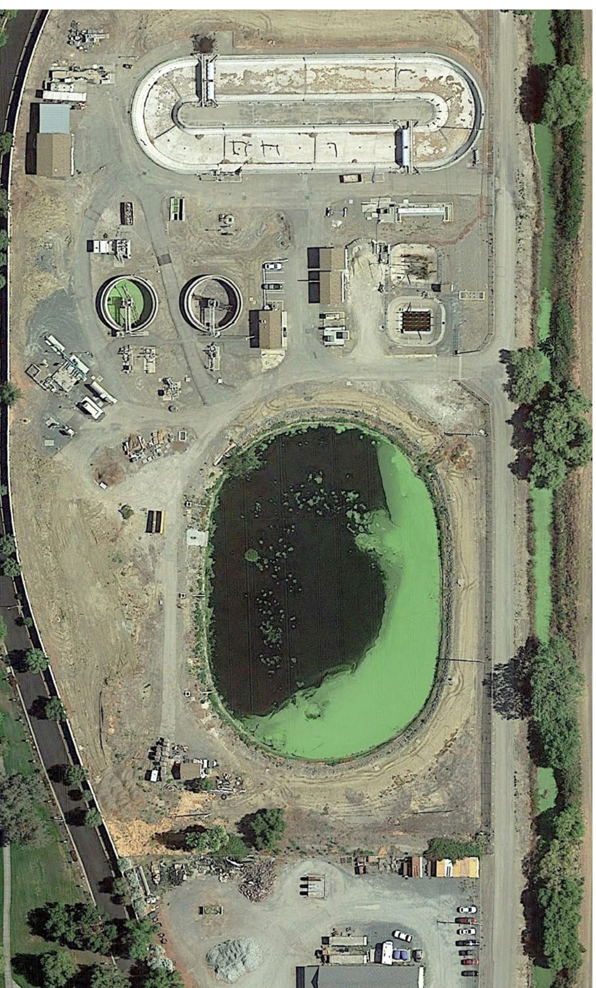

Stantec

The Town of Discovery Bay
Community Services District

Wastewater Treatment Plant
Master Plan Update

Final
November 2019

Prepared for
The Town of Discovery Bay
Community Services District

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Prepared by ______________________
(signature)

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11/14/2019

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1.0 INTRODUCTION

The Town of Discovery Bay Community Services District (TDBCSD) owns wastewater collection, treatment and disposal facilities that serve the community of Discovery Bay. These facilities are currently permitted to treat and discharge to Old River an average flow of 2.35 million gallons per day (Mgal/d). The overall wastewater treatment system includes interdependent facilities at two sites: Plant 1 and Plant 2.

The District completed a Wastewater Treatment Plant Master Plan, dated February 2013 (including Amendment 1), which included detailed evaluations of all components of the wastewater treatment system and resulted in a prioritized list of recommended improvements. Amendment 2 (April 2015) and Amendment 2 Update (September 2015) to the Master Plan were subsequently developed to investigate methods for meeting new and more stringent requirements for nitrogen removal. Amendment 3 (March 2016) was developed to investigate whether Plant 1 should be rehabilitated or replaced with new facilities at Plant 2.

Many of the improvements recommended in the previous Master Plan have been implemented through several major construction projects. However, the nitrogen removal improvements developed and recommended in Amendment 2 and Amendment 2 Update have not yet been constructed.

Since the preparation of the previous Master Plan, the District has experienced substantial reductions in wastewater flows, apparently resulting from water conservation. Because of these reductions and because of the high cost of the proposed improvements for nitrogen removal, the District authorized this Master Plan Update to re-evaluate needed improvements under the changed conditions.

This Master Plan Update is arranged in sections covering key aspects of the investigation and of the facilities as follows:

Section 1: Introduction.

Section 2: Executive Summary. This section includes a condensed version of the investigations and key findings developed throughout Sections 3 through 22.

Section 3: Existing and Future Land Use. The current level of development within the community is assessed and anticipated future development through buildout is evaluated so that incremental wastewater flows and loads from future development can be projected.

Section 4: Collection System Pump Stations. Recommendations and costs for improvements to collection system pumping stations are presented.

Section 5: Wastewater Flows and Loads. Recent plant data on flows and loads are evaluated to establish existing average wastewater characteristics and to assess the variability of those characteristics. Then the incremental flows and loads from future development are added to determine total projected flows and loads through buildout.

Section 6: Overview of Wastewater Treatment Plant. An overview of the existing wastewater treatment facilities is presented, including layout, types of treatment employed, process capacities and key design criteria, and performance.

Section 7: Plant Hydraulic Capacity Analysis. A computer model of all piping, pump systems, hydraulic structures, and other features that determine how much flow can be passed through the wastewater treatment facilities was developed and used to assess potential hydraulic bottlenecks under existing and future conditions.

Section 8: Compliance with Waste Discharge Requirements. The historical performance of the plant in meeting existing waste discharge requirements is reviewed. New requirements soon to be implemented and the need for plant improvements to meet those requirements are discussed.

Section 9: Influent Pump Station. The adequacy of this recently upgraded facility to meet revised future design requirements is assessed.

Section 10: Headworks. The headworks includes influent flow measurement, screening, and sampling features. Capacities, operational issues, and recommended improvements are presented.

Section 11: Secondary Treatment. The secondary treatment system is the heart of the wastewater treatment plant and is where most of the influent pollutants are removed. The improvements needed for nitrogen removal are evaluated and the capacities of these facilities (after upgrade) under various normal and abnormal operating conditions are assessed.

Section 12: Secondary Effluent Lift Station. The Secondary Effluent Lift Station is used to pump the effluent from the secondary treatment system to the downstream filtration and disinfection facilities. The adequacy of this pumping system for handling future design peak flows is assessed.

Section 13: Tertiary Filtration. A new filtration system has recently been constructed and is assessed to confirm its ability to meet future design flow requirements.

Section 14: UV Disinfection. Ultraviolet (UV) radiation is currently used for disinfection of the wastewater effluent. Testing procedures to confirm the capacity of this system are recommended.

Section 15: Effluent Pump Station, Pipeline, and Diffuser. The Effluent Pump Station is used to pump the treated effluent through the effluent pipeline and a diffuser system in Old River. The adequacy of these facilities for handling future peak design flows is assessed and needed repairs to the damaged outfall diffuser are discussed.

Section 16: Emergency Storage Return Pumping. An earthen basin is available emergency storage of influent wastewater at Plant 1. Recommended improvements for basin drainage pumping are considered.

Section 17: Effluent Disposal Alternatives. Storage and irrigation as well as percolation basins are considered as alternatives to river discharge.

Section 18: Solids Handling. This section includes an evaluation of the recently expanded facilities for the handling of residual solids (sludge or biosolids) developed within the wastewater treatment plant. Alternatives for disposal of dried biosolids are evaluated.

Section 19: SCADA System. Improvements to the supervisory control and data acquisition (SCADA) system are considered.

Section 20: Rehabilitation of Plant 1. The analyses and recommendations from the previous Master Plan Amendment 3 are summarized and additional improvements to Plant 1 are considered.

Section 21: Miscellaneous Improvements. Various improvements not covered in the foregoing sections are considered.

Section 22: Summary of Future Improvements. All of the improvements recommended in the preceding sections are summarized, together with costs, and recommended timing for implementation. A site layout with the recommended improvements is shown.

2.0 EXECUTIVE SUMMARY

Presented below is a section-by-section summary of the key investigations and findings included Sections 3 through 22 of this Master Plan report.

2.1 SECTION 3 - FUTURE LAND USE

Projections of future development in the Town of Discovery Bay Community Services District (TDBCSD) sewer service area were made so that flows and loads from future growth could be estimated (see Section 5 for flows and loads). Projected growth, based on land use, is summarized in Table 2-1.

Table 2-1 Projected Growth within TDBCSD After March 31, 2018

Development	Number
Homes and Condominiums to be Added After 7/31/2018	
Approved, But Not Yet Built	67
Undeveloped Lots (Discovery Bay Proper)	44
Pantages	300
Newport Point	70
Villages (Hoffman)	76
Golf Course	13
5-Acre Lots	5
Evans	19
Discovery Bay / Willow Lake Condominiums	80
Total	674
Homes and Condominiums Added 3/31/2018 through 7/31/2018	38
Equivalent for Conversion of 661 Vacation Homes to Primary Res.	496
Homes and Condominiums to be Added After 3/31/2018	**1,208**
Office and Business Park, Acres	
Bixler Business Park	7
Marsh Creek Office	1.2
Total	**8.2**
Commercial, Acres	
Highway 4	**5**

2.2 SECTION 4 - COLLECTION SYSTEM PUMP STATIONS

There are fifteen sewage pumping stations within the Discovery Bay sewage collection system. Most have undergone repairs in recent years. Four pump stations still require repairs and new coating systems for the concrete wet wells and are listed in Table 2-2, which shows to budgetary costs for these repairs.

2.3 SECTION 5 - WASTEWATER FLOWS AND LOADS

Existing and projected future flows and loads are shown in Table 2-3.

2.4 SECTION 6 – OVERVIEW OF EXISTING WASTEWATER TREATMENT PLANT

The TDBCSD wastewater treatment plant is a combination of two plants, referred to as Plant 1 and Plant 2. All influent sewage goes to the Influent Pump Station that is located within Plant 1, from which it is pumped to separate oxidation ditch secondary treatment systems at Plants 1 and 2. The secondary treatment effluents from the two plants are rejoined in Plant 2 for subsequent filtration, UV disinfection, and export pumping to Old River. Biosolids handling facilities for both plants are located at Plant 2 and include an aerobic digester, belt filter presses, active solar dryers, and sludge lagoons.

Site plans, flow schematics, and hydraulic profiles for the two plants are presented in Figures 6-1 through 6-5 in Section 6.

2.5 SECTION 7 – PLANT HYDRAULIC CAPACITY ANALYSIS

The hydraulic features within Plant 1 and Plant 2 (including the proposed anoxic basin additions) upstream of the UV disinfection system are able to handle the future peak hour flow of 4.89 Mgal/d whether or not Plant 1 is in service. However, due to limitations of the existing UV system and/or Export Pump Station, flows higher than 4.2 Mgal/d are accommodated with excess flow diversions to the sludge lagoons ahead of the effluent filters.

2.6 SECTION 8 – COMPLIANCE WITH WASTE DISCHARGE REQUIREMENTS

Effluent discharges from the TDBCSD WWTP to Old River are regulated under a National Pollution Discharge Elimination System (NPDES) permit issued by the State of California. The plant is generally compliant with all existing discharge requirements. New requirements for ammonia-nitrogen and nitrate+nitrite-nitrogen will take effect on December 31, 2023, and will require major improvements to the secondary treatment system (discussed in Section 11).

Table 2-2 Collection System Pump Stations Data and Required Improvements

Pump Station	Location	Type of Pumps	No. of Pumps	Capacity Each Pump, gpm	Horse-power Each Pump	Year Const.	Year Pumps Last Replaced	Year Pumps Last Rehabilitated	Required Improve-ments (a)	Budgetary Cost for Improvements, $ (b)
A	Discovery Point	Self Prime	2	225	3	70's	2008	-	1	40,000
C	Beaver Lane and Willow Lake Road	Self Prime	2	300	5	80,s	-	2009	1	40,000
D	Discovery Bay Blvd Near Beaver Lane	Self Prime	2	300	5	70's	2008	-	1	40,000
E	Discovery Bay Blvd and Cabrillo Point	Self Prime	2	680	10	80's	2008	-	1	60,000
Total Cost										180,000

(a) Required improvements according to code numbers as follows (not including SCADA improvements, which are covered in Section 19):
  1 Rehabiliatate and recoat concrete wet wells (cost $ 40,000 for small wet wells / $ 60,000 for large wet wells)

(b) Mid-2019 cost level. ENR 20-Cities CCI = 11,300.

Table 2-3 Existing and Future Flows and Loads

Parameter (a)	Existing (b)	Increment (c)	Baseline Future (d)	Alternate Future (e)	Previous Master Plan Future (f)
Flow Ratios					
ADWF/AAF	1.0	1.0	1.0	1.0	0.97
ADMMF/AAF	1.2	1.2	1.2	1.3	1.1
PDF/AAF	2.1	2.1	2.1	2.8	2.0
PHF/AAF	3.0	3.0	3.0	4.3	3.0
Load Ratios					
ADMML/AAL	1.3	1.3	1.3	1.3	1.3
PDL/AAL	2.0	2.0	2.0	2.0	2.0
Flow, Mgal/d					
ADWF	1.32	0.31	1.63	0.98	2.35
AAF	1.32	0.31	1.63	0.98	2.42
ADMMF	1.58	0.37	1.96	1.30	2.66
PDF	2.77	0.65	3.42	2.77	4.84
PHF	3.96	0.93	4.89	4.24	7.26
Annual Average Load, lb/d					
BOD	3,027	711	3,738	3,738	4,037
TSS	3,027	711	3,738	3,738	4,037
TKN	605	142	748	748	807
Average Day Maximum Monthly Load, lb/d					
BOD	3,936	924	4,860	4,860	5,248
TSS	3,936	924	4,860	4,860	5,248
TKN	787	185	972	972	1,050
Average Constituent Concentrations, mg/L					
BOD	275	275	275	459	200
TSS	275	275	275	459	200
TKN	55	55	55	92	40
Constituent Concentrations with ADMMF and ADMML,					
BOD	298	298	298	448	236
TSS	298	298	298	448	236
TKN	60	60	60	90	47
Constituent Concentrations with AAF and ADMML, mg/L					
BOD	358	358	358	597	260
TSS	358	358	358	597	260
TKN	72	72	72	119	52

(a) ADWF = Average Dry Weather Flow, AAF = Annual Average Flow,
ADMMF = Average Day Maximum Monthly Flow,
PDF = Peak Day Flow, PHF = Peak Hour Flow
AAL = Annual Average Load, ADMML = Average Day Maximum Monthly Load

(b) Based on AAF = 1.32 Mgal/d as of March 31, 2018.

(c) Average incremental flow from Table 5-11.

(d) Baseline future presumes per capita flows remain same as existing (83.5 gal/d, average). Flow and load peaking factors assumed same as existing.

(e) Alternate Future presumes exteme water conservation with average per capita flow of 50 gal/d. Differences between average flows and peak flows assumed same as Baseline Future. Flow peaking factors adjusted per above. Loads assumed same as Baseline Future.

(f) Final Master Plan dated February 13, 2013, Including Amendment 1.

2.7 SECTION 9 – INFLUENT PUMP STATION

The hydraulic analysis developed in Section 7 showed that the Influent Pump Station is now capable of handling flows substantially higher than the design peak hour flow of 4.89 Mgal/d, whether pumping to Plant 2 only or to a combination of Plant 1 and Plant 2. No future improvements to this pump station are currently anticipated.

2.8 SECTION 10 – HEADWORKS

There are separate headworks systems at Plant 1 and at Plant 2. Each headworks includes flow metering, screening, and odor scrubbing facilities. Both headworks have adequate capacity for future design flows and do not need to be expanded.

The Plant 1 headworks are in need of some repairs and rehabilitation, which are considered in Section 20 of this document.

The Plant 2 headworks includes an automated sampler system for monitoring the plant influent (same for both plants). The existing sampler intake is upstream from the screening system and suffers from rag accumulations, resulting in non-representative samples. A new sampler intake downstream of the screening system is recommended.

2.9 SECTION 11 – SECONDARY TREATMENT FACILITIES

The existing secondary treatment system includes one oxidation ditch and two clarifiers at Plant 1 and two oxidation ditches and three clarifiers at Plant 2. These systems were not designed for nitrogen removal and are not capable of meeting the new discharge requirements that will take effect on December 31, 2023.

To allow nitrogen removal, anoxic basins must be constructed upstream of each of the three oxidation ditches. A cost estimate for the recommended improvements is shown in Table 2-4.

The proposed anoxic basin and oxidation ditch facilities have been evaluated based on limited and incomplete wastewater characterization data. The proposed design must be validated after routine and intensive monitoring data become available from the new sampling system recommended at the Plant 2 headworks.

With improved secondary treatment systems, the capacity of Plant 2 alone will not be adequate to handle future peak design flows and loads. For this reason and to allow shutdowns for repairs on the oxidation ditches at Plant 2, Plant 1 must be upgraded and maintained in an operable condition, even though it will not be necessary to operate Plant 1 all of the time.

The actual oxygen delivery capacities of the existing rotors in the oxidation ditches are not accurately known. Field oxygen transfer testing is required to confirm capacities under various operating conditions.

Based on the best information currently available, it is apparent that substantial supplemental aeration capacity will be needed to meet future peak oxygen demands. Evaluation, selection, and design of supplemental aeration systems must be completed after the capacities of the existing brush rotors are confirmed. A cost allowance of $800,000 is currently recommended for these improvements.

Table 2-4 Cost Estimate for Concrete Anoxic Basins and Related Facilities

	Cost, $ (a)			
Item	**Ditch 1 Anoxic**	**Ditch 2 Anoxic**	**Ditch 3 Anoxic**	**Total**
Dewatering	165,000	165,000	165,000	495,000
Shoring	0	243,000	121,500	364,500
Excavation and Backfill	189,000	115,500	152,250	456,750
Concrete Structure and Guardrails	689,880	689,880	689,880	2,069,640
Pumps and Mixers	110,000	110,000	110,000	330,000
Piping and Appurtenances	251,800	120,600	120,600	493,000
Sitework	60,000	60,000	60,000	180,000
Electrical and Instrumentation	280,000	280,000	280,000	840,000
Subtotal 1	1,745,680	1,783,980	1,699,230	5,228,890
Subtotal 1, Rounded	1,746,000	1,784,000	1,699,000	5,229,000
Contingencies @ 20%	349,000	357,000	340,000	1,046,000
Subtotal 2	2,095,000	2,141,000	2,039,000	6,275,000
Engineering, Admin, and Environmental @ 25%	524,000	535,000	510,000	1,569,000
Total	2,619,000	2,676,000	2,549,000	7,844,000

(a) Mid 2019 cost level, ENR 20-Cities CCI = 11,300.

2.10 SECTION 12 – SECONDARY EFFLUENT LIFT STATION

The secondary effluent flows from the two plants are combined into the Secondary Effluent Lift Station, which is located on the Plant 2 site and is used to pump the secondary effluent to the downstream filters, Parshall flume, and UV disinfection system.

As developed in Section 7, the reliable capacity of the pump station is approximately 5.6 Mgal/d, which exceeds the future design requirement of 5.13 Mgal/d (4.89 Mgal/d plus 5% recycle allowance).

No improvements to the Secondary Effluent Lift Station are needed.

2.11 SECTION 13 – TERTIARY FILTRATION

A new upflow sand filtration system was recently constructed at Plant 2 and has a reliable capacity of 4.74 Mgal/d. However, actual flows to the filters are limited to 4.2 Mgal/d due to limitations of the downstream UV disinfection system and/or Export Pump Station. Flows greater than 4.2 Mgal/d are expected to be rare and of limited duration. Flows in excess of 4.2 Mgal/d can be diverted to the sludge lagoons.

The existing filters are adequate for the buildout condition. Dedicated flow equalization ahead of the filters is not needed and DAF treatment of sludge lagoon return flows is not needed.

2.12 SECTION 14 – UV DISINFECTION

The Discovery Bay WWTP contains two UV channels containing Trojan UV3000Plus™ equipment that was designed to each deliver a 100 mJ/cm^2 UV dose at a flow of 4.8 MGD and a UVT of 65%.

The system was validated by Trojan and a spot-check bioassay was performed in 2017 by Moreland. The 2017 Moreland report concluded that four of eight tests performed equally or better than predicted. However, when Stantec performed the calculations using the Trojan 2012 Addendum RED prediction equation and Trojan provided factor values, six of eight tests performed equally or better than predicted.

As summarized in Section 14.1.5 and 14.1.5.1, a considerable percentage of UVT values measured from September 2016 to October 2019 were lower than the assumed design UVT of 65%. As additional UV disinfection capacity is required when UVT drops, there are a number of conditions under which two channels must operate to deliver the required dose. For most normal flows and UVT conditions, one channel with four banks online is sufficient to deliver the required UV dose. Two channels will likely be needed during periods of wet weather flows and for periods of low UVT. The existing system controls will have to be modified to allow for dual channel operation. For conditions where the 100 mJ/cm^2 is not met with six banks online, flow must be diverted to the sludge lagoons upstream of the tertiary filters. Currently, the Export Pump Station has a capacity of 4.2 MGD. Based on the Trojan RED prediction equation, at a flow of 4.2 MGD, one channel with four banks online could deliver the required UV dose for UVTs above 62.9% and two channels with three banks online per channel could deliver the required UV dose for UVTs above 56.4%.

To verify the UV system’s performance, Stantec recommends evaluating the following:

1. The hydraulic capacity of the channels,
2. Velocity profiles, including proper flow splitting between the two channels, and
3. The delivered UV dose at different flows and at different UVTs.

The total cost for the UV system testing and possible improvements is estimated to be $200,000. This includes $110,000 for hydraulic and system performance testing and an allowance of $90,000 for hydraulic system improvements and control system modifications. The additional investigations described above must be completed to confirm recommended improvements.

2.13 SECTION 15 – EFFLUENT PUMP STATION, PIPELINE, AND DIFFUSER

The existing Export Pump Station, together with the export pipeline and the outfall diffuser (in its original design condition), has a reliable capacity of about 4.2 Mgal/d. Since incremental flows higher than this limit can be diverted to the sludge lagoons ahead of the filters, no improvements or expansion of the Export Pump Station and pipeline are needed. However, the existing outfall diffuser has been compromised, resulting in decreased capacity for the combined export facilities.

Recent investigations have shown that the outfall diffuser in Old River is obstructed and damaged and must be partially replaced. The budgetary cost estimate for this work is $500,000. The District is currently proceeding with design and implementation of improvements to the outfall using existing funding sources, separate from proposed Master Plan projects.

2.14 SECTION 16 – RETURN PUMP STATION FOR EMERGENCY STORAGE BASIN

The Plant 1 site includes an earthen emergency storage basin with a volume of approximate 5 million gallons. During an emergency when Plant 1 and/or Plant 2 may not be able to handle the entire influent flow, a portion or all the influent flow can be diverted to the emergency storage basin for temporary holding until such time as the stored volume can be treated. At the present time, however, the only way to return stored wastewater is to use portable pumping equipment.

A 12-inch drainpipe from the emergency storage basin to Pump Station W is recommended. The budgetary cost is $75,000.

2.15 SECTION 17 – EFFLUENT DISPOSAL ALTERNATIVES

Two options for possible disposal/reuse of the wastewater effluent on land were considered. The potential advantage of such disposal/reuse would be to attain less stringent discharge requirements, resulting in lower plant improvement costs, as compared to continued river discharge.

An independent investigation by the District indicated a likely cost near $17 million to implement winter storage and subsequent reuse by crop irrigation. Since this far exceeds any potential treatment plant cost savings (up to about $8 million), 100 percent storage and reclamation reuse were eliminated from further analysis.

A conceptual evaluation of a potential percolation disposal system showed a minimum likely cost of about $14 million, again in excess of any potential savings for treatment plant improvements. Therefore, percolation disposal is not recommended.

2.16 SECTION 18 – SOLIDS HANDLING

Existing facilities include an aerobic digester, three belt filter presses, and four active solar dryers. Evaluations showed that all of these facilities have adequate reliable capacity under future design loading conditions and do not need to be expanded.

In addition to the above, there are two sludge lagoons that historically had been used to store solids when belt press and solar dryer capacity was inadequate (before recent improvements). Now there is adequate belt press and solar dryer capacity to remove the stored solids over several years, while still keeping up with ongoing solids production. The sludge dredge that is used to remove solids from the lagoons is worn out and needs to be replaced at an estimated cost of $125,000.

The sludge lagoons will remain useful to the plant in the future, even after all stored solids are removed. Existing and future possible uses of the sludge lagoons include the following:

- Emergency storage of solids in the event of a failure or other removal from service of key solids handling facilities (aerobic digester, belt presses, or active solar dryers).
- Peak flow trimming storage for secondary effluent to limit the flow to the filters and UV disinfection systems.
- Temporary storage of subpar effluent to avoid discharge violations.

Currently all dried biosolids produced at the plant are disposed of into a landfill. However, landfill disposal is being phased out by State regulations. After initial screening out of other alternatives, three alternatives for biosolids reuse were developed as follows:

1. Land application of all biosolids on District-owned land (requires additional land acquisition).
2. Maximize land application of biosolids on existing District property and contract with Synagro (or similar service) for hauling and land application of the remainder.
3. Hauling and land application of all biosolids by Synagro (or similar service).

Alternatives 1 and 3 require major capital expenses ($4.4 million and $2.4 million, respectively), while Alternative 3 does not require any capital expenses. Additionally, Alternative 3 has the lowest annual costs. Therefore, Alternative 3 is recommended.

The only recommended improvements to the solids handling facilities are as follows:

- New sludge dredge for sludge lagoons - $125,000
- Repair damaged solar dryer conduits - $55,000
- Total - $180,000

2.17 SECTION 19 – SCADA NETWORKING IMPROVEMENTS

Various improvements are required to improve the function of the supervisory control and data acquisition (SCADA) system that connects to all of the District's facilities. These improvements and associated costs are indicated in Table 2-5.

Table 2-5 Cost Estimate for SCADA Improvements

Item	Cost, $		
	Unit Price	**Qty**	**Total Price**
New SCADA Server Equipment and Configuration	40,000	1	40,000
System-wide Radio Study **(note 1)**	10,000	1	10,000
Fiber Optics Improvements	10,000	1	10,000
Network Rack and new UPS at Golf Course Valve Station	15,000	1	15,000
Install Air Conditioning at Valve Station	7,000	1	7,000
Replace Network Switches; Configure SCADA Screens	20,000	1	20,000
Video Cameras and Integration into SCADA	4,000	10	40,000
Subtotal			142,000
Contingencies @ 20%			28,000
Total			170,000

Note 1: If the radio study proves that ethernet radios are viable for additional deployments, the estimated cost of replacing the master ethernet radio and antenna at Plant 2 is $5,000. The estimated cost for ethernet radios and antennas at each remote site is $3,000. Having ethernet radios as an option for the upcoming lift station upgrade projects gives plant staff an alternative to cellular modems, which presently carry a monthly data plan cost of $15/month per site.

2.18 SECTION 20 – REHABILITATION AND IMPROVEMENT OF PLANT 1

Since Plant 1 must be maintained in operable condition and must be reliable when operated, various improvements to plant facilities are needed, including major repairs to the oxidation ditches and clarifiers, replacement of MCC-C, addition of standby power, and more. These improvements and the costs for them are itemized in Table 2-6 (later in this section).

2.19 SECTION 21 – MISCELLANEOUS IMPROVEMENTS

In addition to the improvements developed in previous sections, the following are recommended or can be considered for implementation by the District (budgetary costs shown):

- Decant Pump Station improvements to allow drainage discharges into the sludge lagoons when desired ($84,000).

- Clarifier launder covers to prevent algae growth and eliminate tedious manual efforts for cleaning the launders ($338,000 for all five clarifiers at both plants).
- Extension of a reclaimed water pipeline to allow reclamation reuse on the golf course ($1.37 million).
- Water filling station for construction use of reclaimed water ($198,000).

2.20 SECTION 22 – SUMMARY OF RECOMMENDED IMPROVEMENTS

A list of all the recommended improvements developed in this Master Plan is presented in Table 2-6. For each improvement, a reference is given to the Master Plan section where that improvement is discussed in more detail, a budgetary cost is given, and the timing or condition that would trigger the need for the improvement is indicated. Costs are indicated in three columns to distinguish those improvements that are considered to be essential, those that are non-essential (but still recommended when available budgets allow implementation), and those that are unlikely to be required.

Proposed site plans with recommended improvements are presented in Figures 2-1 and 2-2 for Plants 1 and 2, respectively.

Table 2-6 Recommended Improvements

Item	Plant	Description	Rept. Sect.	Reason for Improvement	Trigger for Implementation	Possible Timing (a)			Budgetary Cost, $1000s (b)		
						Begin Design	Begin Const.	Begin Operation	Essential	Non-Essential	Unlikely
1	1&2	Anoxic Basins and Related Improvements for Denitrification	11, 20	Compliance with New Discharge Requirements	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	7,844 (c)		
2	1&2	Supplemental Aeration in Oxidation Ditches	11	Existing Rotors Inadequate for Future Max Oxygen Demand	Before Actual Oxygen Demands Exceed Reliable Rotor Capacity	2019	2021	2023	800 (d)		
3	2	UV Disinfection Testing and Improvement	14	Improve Performance	Desired Now	2019	2021	2023	200		
4	NA	Repair Effluent Diffuser in Old River	15	Restore Diffuser Capacity	Desired Now	2019	2021	2023	500		
5	1	Emergency Storage Drain to Pump Sta. W	16	Avoid Inconvenient and Inefficient Use of Temporary Pump System to Drain Emergency Storage Basin	When Possible	2019	2021	2023		75	
6	2	Solids Handling Improvements	18	Replace Dredge, Conduits	When Desired	TBD	TBD	TBD		180	
7	1&2	SCADA Networking Improvements	19	SCADA Performance Problems	Desired Now	2019	2021	2023	170		
8	1	Influent Pump Station Grating	20	Safety Concern	Desired Now	2019	2021	2023	15		
9	1	Oxidation Ditch Structural Rehab and Guardrail Repair	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	831		
10	1	Clarifiers Structural Rehab	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	83		
11	1	Clarifiers Mechanical Replacement and Upgrade	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	897		
12	1	MCC-C Replacement	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	416		
13	1	MCC-C Standby Power	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	249		
14	1	Headworks Grating	20	Safety Concern	Desired Now	2019	2021	2023	42		
15	1	Clarifier 1 and 2 RAS Pumps and Check Valves Replacement	20	Replace Deteriorated Equipment	When Possible	TBD	TBD	TBD		299	
16	1	WAS Pumps and Check Valves Replacement	20	Replace Deteriorated Equipment	When Possible	TBD	TBD	TBD		107	
17	1	Storm Drainage Improvements	20	Prevent Flooding	Desired Now	2019	2021	2023	38		
18	1	Transfer Station Instrumentation and Controls	20	Existing Controls Failed	Desired Now	2019	2021	2023	38		
19	1	Demolish Existing Abandoned Facilities	20	Provide Clean and Safe Site	When Possible	TBD	TBD	TBD	167		
20	1	Extend Pump Sta. F Forcemain to Pump Sta. W Manhole	20	Allow Bypass of Influent Pump	Desired Now	2019	2021	2023	38		
21	1	Coat Electrical Cabinets at Influent Pump Sta.	20	White Paint to Prevent Overheat	Desired Now	2019	2021	2023	8		
22	1	Pump Sta. W Isolation Valve	20	Replace Existing Ruined Valve	Desired Now	2019	2021	2023	30		
23	1	Oxidation Ditch Rotor Frame Elect. and Struct. Rehab.	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	600		
24	2	Decant Pump Station Improvements	21	Allow Discharge to Lagoons	Desired Now	2019	2021	2023	84		
25	1&2	Clarifier Launder Covers	21	Eliminate Tedious Maintenance	When Possible	TBD	TBD	TBD	338		
26	2	Extend Reclaimed Water Pipeline to Golf Course	21	Allow Reuse on Golf Course	When Desired	TBD	TBD	TBD		1,370	
27	2	Water Filling Station for Reclaimed Water	21	Allow Easier Construction Reuse	When Desired	TBD	TBD	TBD		198	
28	NA	Collection System Pump Stations	4	Restore Wet Well Integrity	When Possible	TBD	TBD	TBD	180		
29	2	Reverse Osmosis Facilities	21	Reduce Effluent Salinity, Last Resort	If Required by Regulation -- Very Unlikely	TBD	TBD	TBD			20,000
Total by Category, Excluding Effluent Diffuser in Old River (e)									13,068 (e)	2,229	20,000
Total Essential and Non-Essential, Excluding Effluent Diffuser in Old River (e)									15,297		

(a) Approximate timing recommendations, where applicable. TBD = To Be Determined.
(b) Total capital cost, including construction, contingencies, engineering, administration and environmental documentation, as applicable. Mid-2019 cost level. ENR 20-Cities CCI = 11,300.
(c) Validation of process design required after routine and intensive influent monitoring data is available from relocated influent sampler.
(d) Actual cost of supplemental aeration must be verified after special field studies to confirm existing rotor capacity and investigation of supplemental aeration alternatives.
(e) Costs for repair of Old River outfall diffuser are excluded from total due to different funding than other essential Master Plan projects.

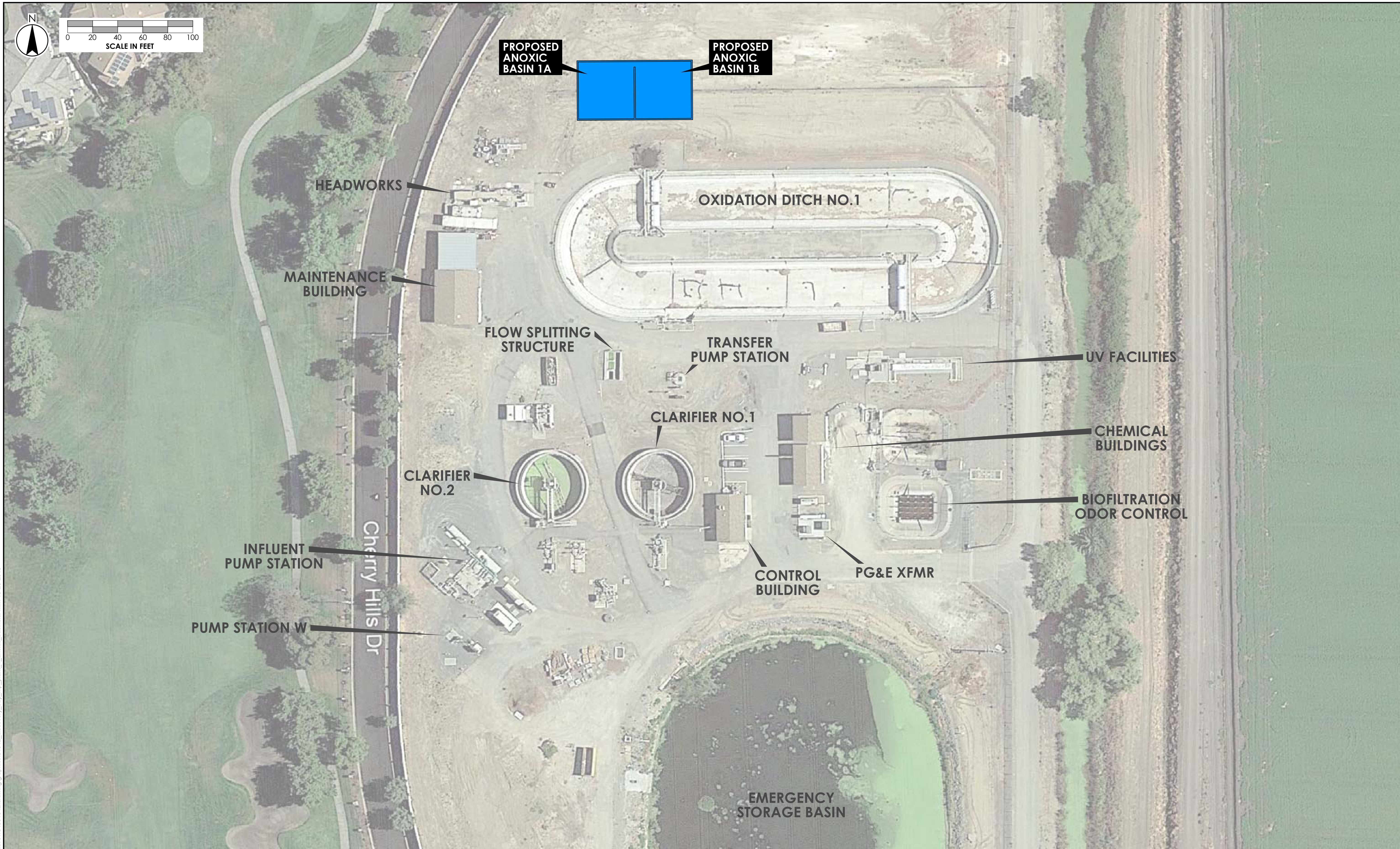


Figure 2-1
Plant 1 Site Plan with Proposed Improvements

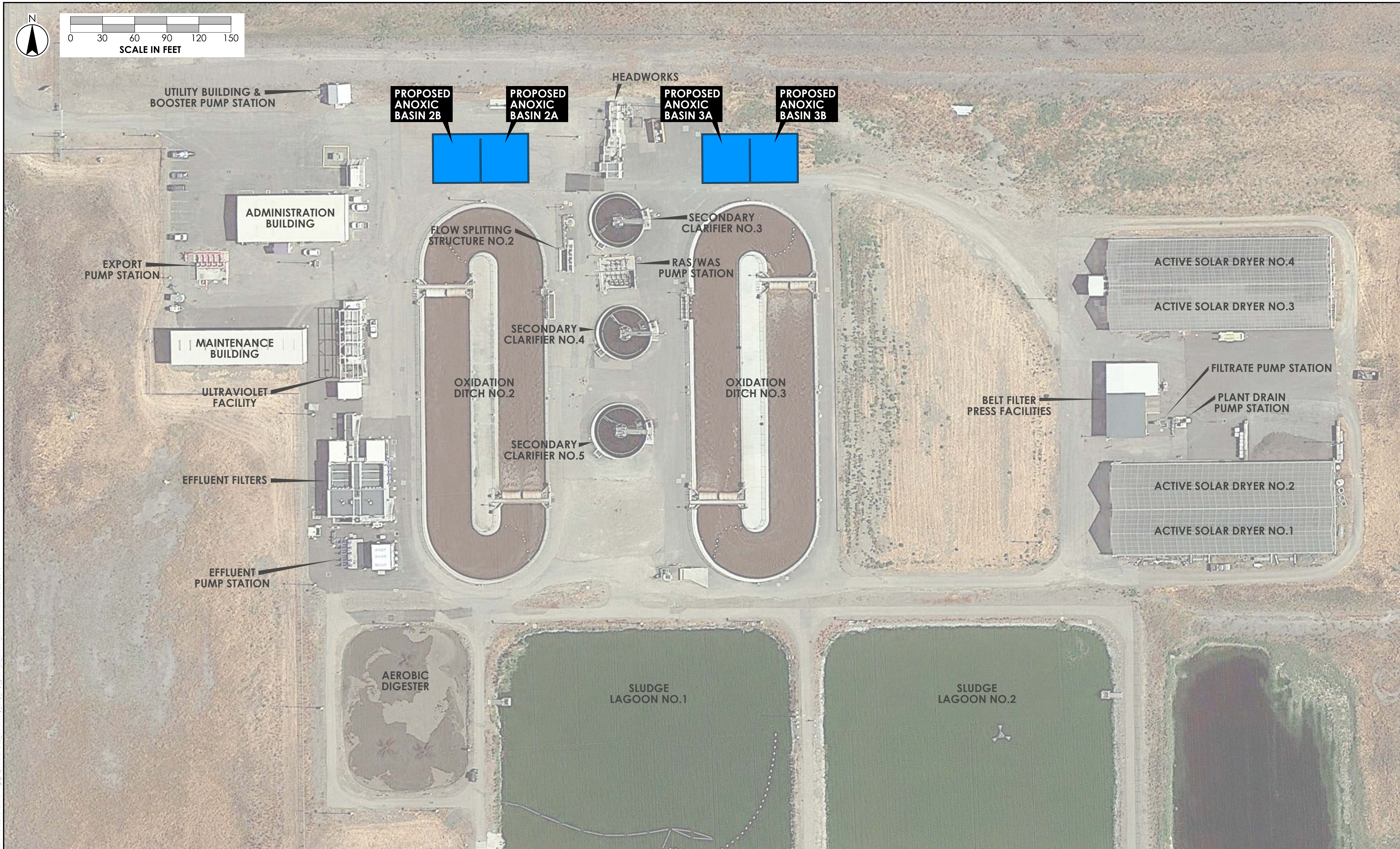


Figure 2-2
Plant 2 Site Plan with Proposed Improvements

3.0 EXISTING AND FUTURE LAND USE

In this section, existing and future land uses within the service area of the Town of Discovery Bay Community Services District Wastewater Treatment Plant (TDBCSD WWTP) are considered. The purpose for considering such land uses is to determine how much new development can be added so that potential increases in wastewater flows and loads can be estimated.

3.1 LAND USE MAP

A map showing existing and planned land uses within the TDBCSD service area is presented in Figure 3-1.

3.2 PROJECTED GROWTH WITHIN THE SERVICE AREA

Projected growth from March 31, 2018 through buildout within the TDBCSD service area includes both residential and non-residential developments. The specific development areas and the projected growth amounts were obtained from the District and are as shown in Table 3-1. The date of March 31, 2018 was selected as the starting point because that is the effective date of the latest annual average flow determination developed for this study (see Section 5). Growth after March 31, 2018 will result additional flow. Since growth projections provided by the District have a starting date of July 31, 2018, Table 3-1 includes an adjustment for actual growth between March 31 and July 31, 2018.

As indicated in Table 3-1, it is currently estimated by the District that there are 661 vacation homes within the District. It is estimated that current wastewater flows from these vacation homes is, on average, 25 percent of those from primary residences. However, it is projected that these vacation homes will be converted to primary residences before buildout, resulting in an effective addition of 496 primary residences (0.75 x 661).

Table 3-1 Projected Growth within TDBCSD After March 31, 2018

Development	Number
Homes and Condominiums to be Added After 7/31/2018	
Approved, But Not Yet Built	67
Undeveloped Lots (Discovery Bay Proper)	44
Pantages	300
Newport Point	70
Villages (Hoffman)	76
Golf Course	13
5-Acre Lots	5
Evans	19
Discovery Bay / Willow Lake Condominiums	80
Total	674
Homes and Condominiums Added 3/31/2018 through 7/31/2018	38
Equivalent for Conversion of 661 Vacation Homes to Primary Res.	496
Homes and Condominiums to be Added After 3/31/2018	**1,208**
Office and Business Park, Acres	
Bixler Business Park	7
Marsh Creek Office	1.2
Total	**8.2**
Commercial, Acres	
Highway 4	**5**

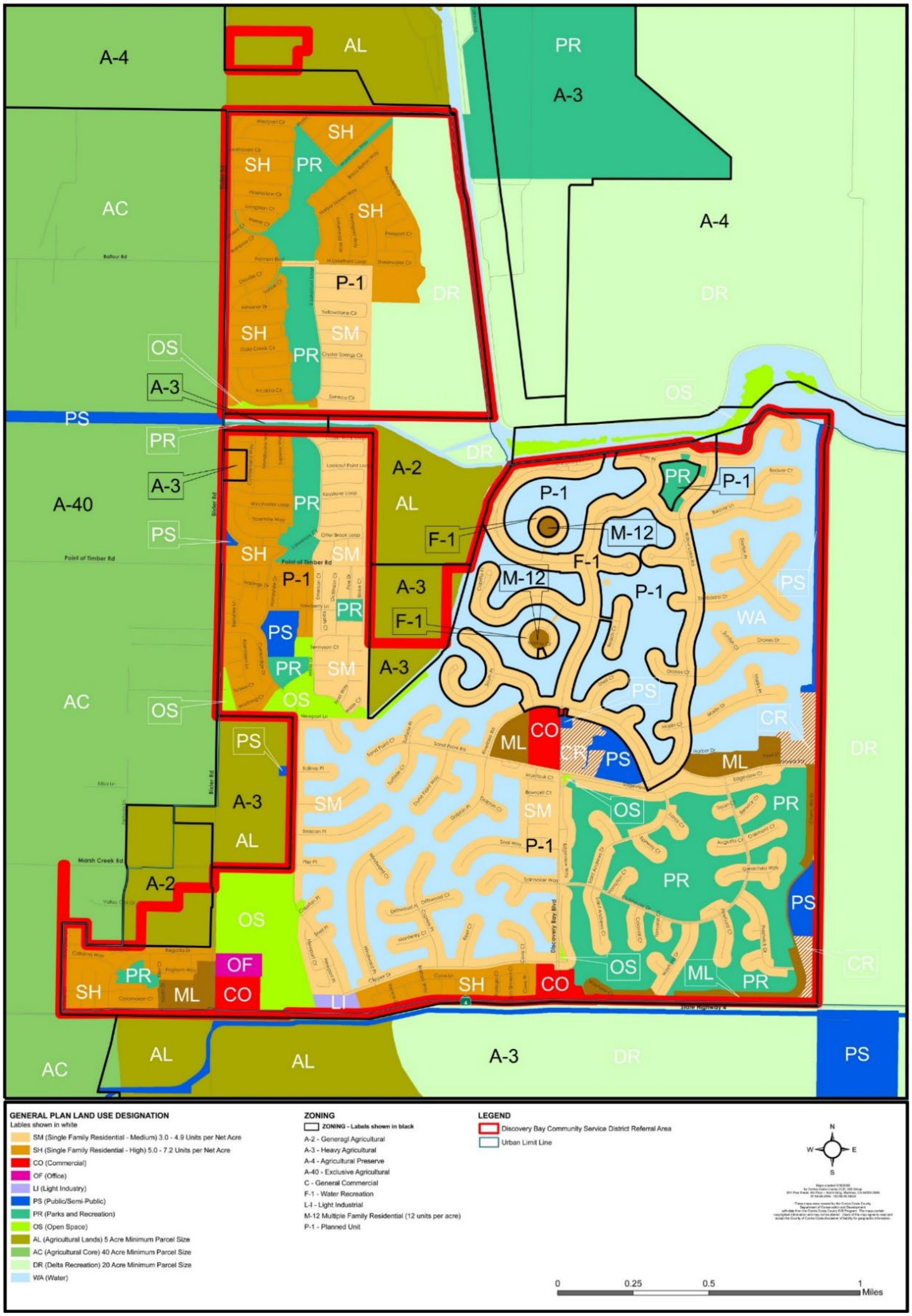


Figure 3-1 Discovery Bay Area Community Service District Referral Area

4.0 COLLECTION SYSTEM PUMP STATIONS

There are fifteen sewage pumping stations within the Discovery Bay sewage collection system. The previous Master Plan, dated February 2013, provided information on required improvements for each of these pump stations. Except as noted below, these improvements have been completed or are planned to be completed with ongoing maintenance activities. Four pump stations still require repairs and new coating systems for the concrete wet wells and are listed in Table 4-1.

As indicated in Table 4-1, the total budgetary cost for the listed pump stations combined is $180,000, assuming that all work will be coordinated by District Staff with only minor consultation with the District Engineer. It is recommended that the District establish appropriate priorities for this work and then budget to accomplish the work accordingly.

Table 4-1: Collection System Pump Stations Data and Required Improvements

Pump Station	Location	Type of Pumps	No. of Pumps	Capacity Each Pump, gpm	Horse-power Each Pump	Year Const.	Year Pumps Last Replaced	Year Pumps Last Rehabilitated	Required Improve-ments (a)	Budgetary Cost for Improvements, $ (b)
A	Discovery Point	Self Prime	2	225	3	70's	2008	-	1	40,000
C	Beaver Lane and Willow Lake Road	Self Prime	2	300	5	80,s	-	2009	1	40,000
D	Discovery Bay Blvd Near Beaver Lane	Self Prime	2	300	5	70's	2008	-	1	40,000
E	Discovery Bay Blvd and Cabrillo Point	Self Prime	2	680	10	80's	2008	-	1	60,000
Total Cost										180,000

(a) Required improvements according to code numbers as follows (not including SCADA improvements, which are covered in Section 19):
  1 Rehabiliatate and recoat concrete wet wells (cost $ 40,000 for small wet wells / $ 60,000 for large wet wells)

(b) Mid-2019 cost level. ENR 20-Cities CCI = 11,300.

5.0 WASTEWATER FLOWS AND LOADS

The purpose of this section is to establish the wastewater flows and loads that comprise the foundation of this Master Plan Update. Recent historical plant influent data are evaluated together with the results of special influent monitoring studies to establish existing conditions, which are used as the basis for projecting buildout conditions in Discovery Bay.

5.1 ANALYSIS OF RECENT PLANT INFLUENT DATA

Influent wastewater flows and characteristics from January 2013 through September 2018 were received from TDBCSD and have been analyzed as described below. Graphs showing influent flows, influent BOD loads, influent BOD concentrations, and ratios of TSS and Ammonia-N concentrations to BOD concentrations for the period of study are provided. Where 30-day and 365-day average values are shown, they are centered averages based on data extending one-half the averaging period before and after the date indicated.

5.1.1 Evaluation of Historical Flows

Historical influent flows for the period of record indicated above are shown in Figure 5-1. Although there was a slight decrease in the 365-day average flow (annual average flows or AAF) for the entire period from July 2013 to March 2018 (the first and last times that centered 365-day average values were available), the actual minimum AAF may have occurred in mid-2016 and flows have been rising slightly since then. The AAF as of March 31, 2018 (includes six months before and after) was 1.32 Mgal/d.

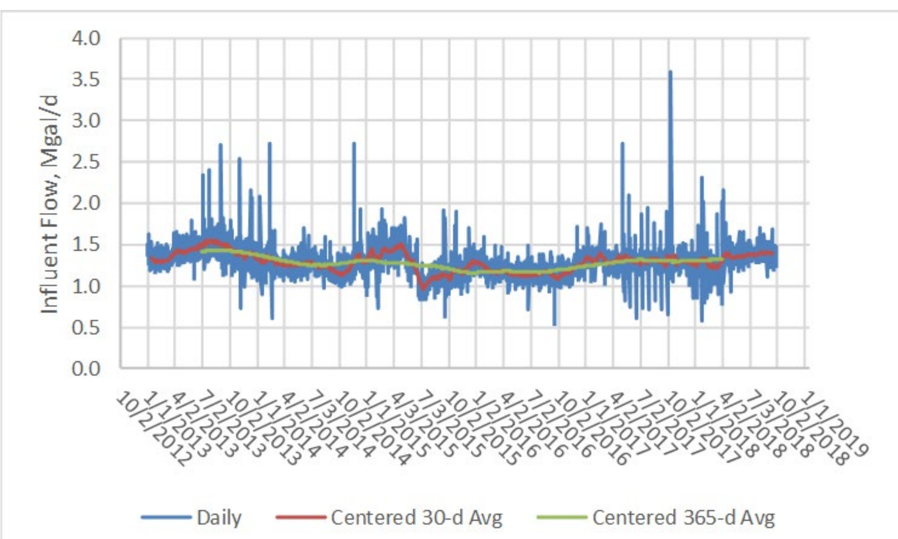


Figure 5-1 Influent Flows

Ratios of daily and 30-day average flows to then current 365-day average flows are shown in Figure 5-2. The maximum ratios shown in Figure 5-2 are compared to values adopted in the previous Master Plan in Table 5-1, which also includes recommended values for this Master Plan.

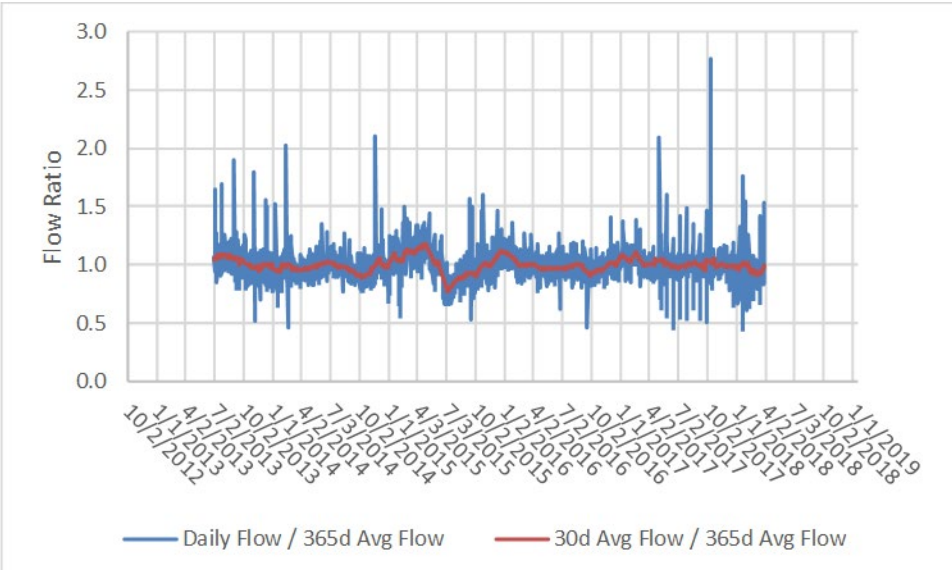


Figure 5-2 Influent Flow Ratio to Annual Average Flow

Table 5-1 Flow Ratios (Peaking Factors)

Flow Ratio	2013-2018 Data	Previous Master Plan Value	Value for This Master Plan
Max. 30d Avg / 365d Avg	1.18	1.1	1.2
Max. Daily / 365d Avg	2.10	2.0	2.1

Average dry weather flows (ADWFs) were evaluated as the average flow during the months of July through September. For the period of record considered herein the ratio of ADWF/AAF ranged from 0.87 to 1.06. For all practical purposes, the ADWF and AAF can be considered equal (the previous Master Plan ADWF/AAF ratio was determined to be 0.98).

In the Town of Discovery Bay CSD Preliminary System Evaluation and Capacity Assurance Plan (SECAP) completed by Stantec in June 2012, the peak hour flow for the collection system was determined for a 10-year frequency 6-hour storm event to be 4.35 Mgal/d. At the time, the average dry weather flow (and approximate annual average flow) was 1.59 Mgal/d, resulting in a peaking factor of 2.74. To be conservative and to allow for an increasing peaking factor with decreasing base flows, the ratio of the peak hour flow (PHF) to the AAF is established at 3.0.

5.1.2 Evaluation of Annual Average BOD Loads

Daily, 30-d average and 365-d average BOD loads are shown in Figure 5-3. Also shown in the figure is a linear regression analysis of the 365-d average data. This figure indicates an ongoing downward trend in

BOD load for the five-year period evaluated. The slope of the trendline indicates the BOD load is decreasing at the rate of about 50 lb/d per year. The apparent downward trend in BOD load is peculiar and would not be expected with continued development and while the population within the District has been increasing slightly. The BOD load data are considered to be unreliable – this topic is discussed further later in this memorandum in connection with special monitoring studies.

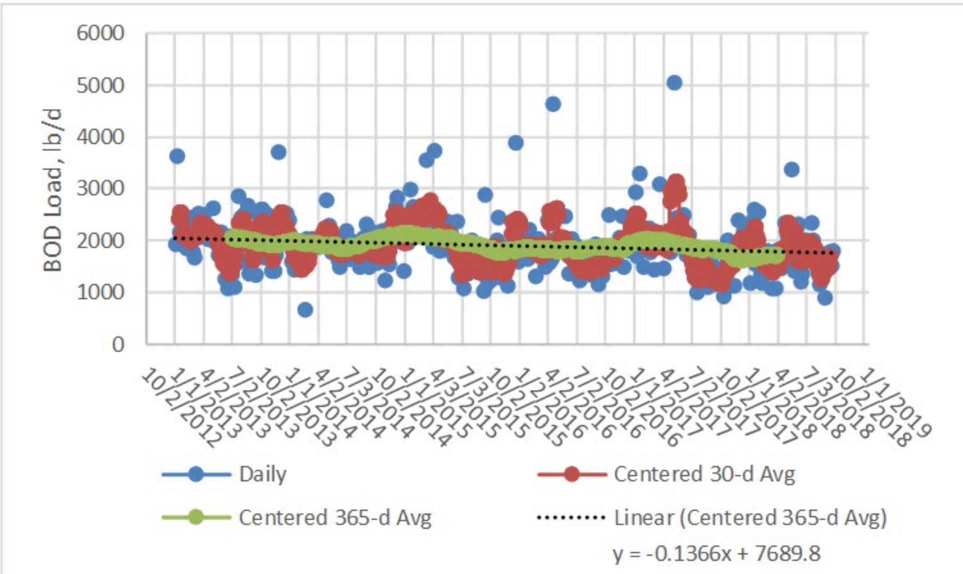


Figure 5-3 Influent BOD Loads

Ratios of daily and 30-day average BOD loads to then current 365-day average BOD loads (i.e., annual average loads, AALs) are shown in Figure 5-4. The maximum ratios shown in Figure 5-4 are compared to values adopted in the previous Master Plan in Table 5-2, which also shows recommended values for this Master Plan. As indicated in the table, the recommended values for this Master Plan are lower than the maximum values shown in Figure 5-4. Reasons for adopting the lower values are as follows:

- The historical data are based on once-per-week sampling. This is inadequate for developing reliable monthly average values, as there are only four data entries per month and a single unusual value can skew the monthly average.
- The historical BOD values are believed to be erroneous as discussed later in this section in connection with special monitoring studies.

Typical textbook peaking factor values are recommended to establish the average day maximum monthly load (ADMML) and the peak day load (PDL) for BOD. Accordingly, the following peaking factors are recommended for this Master Plan. They are the same as adopted for the previous Master Plan and for the same reasons.

- Ratio ADMML/AAL = 1.3
- Ratio PDL/AAL = 2.0

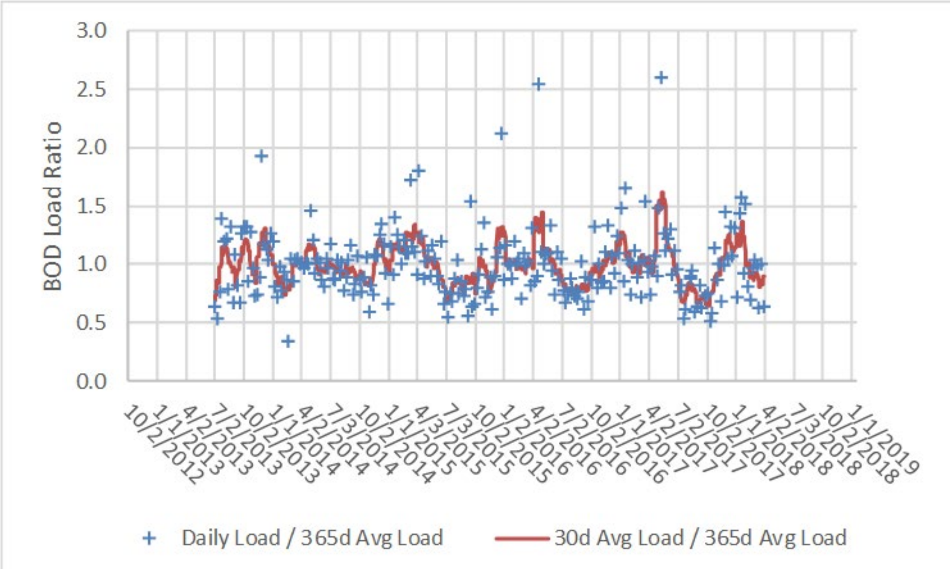


Figure 5-4 Influent BOD Load Ratios

Table 5-2 BOD Load Ratios (Peaking Factors)

BOD Load Ratio	2013-2018 Data (a)	Previous Master Plan Value	Value for This Master Plan
Max. 30d Avg / 365d Avg (ADMML/AAL)	1.6	1.3	1.3
Max. Daily / 365d Avg (PDL/AAL)	2.6	2.0	2.0

(a) Data considered to be unreliable as discussed in text.

5.1.3 Evaluation of Annual Average BOD Concentrations

Daily, 30-d average and 365-d average influent BOD concentrations are shown in Figure 5-5. From the graph, it appears that, although there is substantial scattering of data, the recorded average BOD concentration remained relatively constant for 2013 through mid-2017 and then dropped rather suddenly to a new lower tendency in the remainder of 2017 and throughout 2018. This apparent sudden decrease is peculiar. Possible explanations for the decrease could include a sudden increase in infiltration and inflow or a change in sampling or analysis methods. Although no probable cause for the decrease has been investigated, problems with the historical BOD data are discussed later in this memorandum in connection with special monitoring studies.

5.1.4 Evaluation of Influent TSS/BOD Concentration Ratios

Ratios of TSS/BOD are shown in Figure 5-6. Key observations are listed below:

1. The TSS/BOD ratio has been highly variable, which makes it difficult to have confidence in the values.

2. The central tendency of the data has been relatively constant over the five-year period evaluated. The average TSS/BOD ratio over the five-year period was 0.75, which is extremely low for domestic sewage (a value near 1.0 would be expected), which causes concern about confidence in the values.

3. Problems with historical BOD and TSS data are discussed later in this section in connection with special monitoring studies.

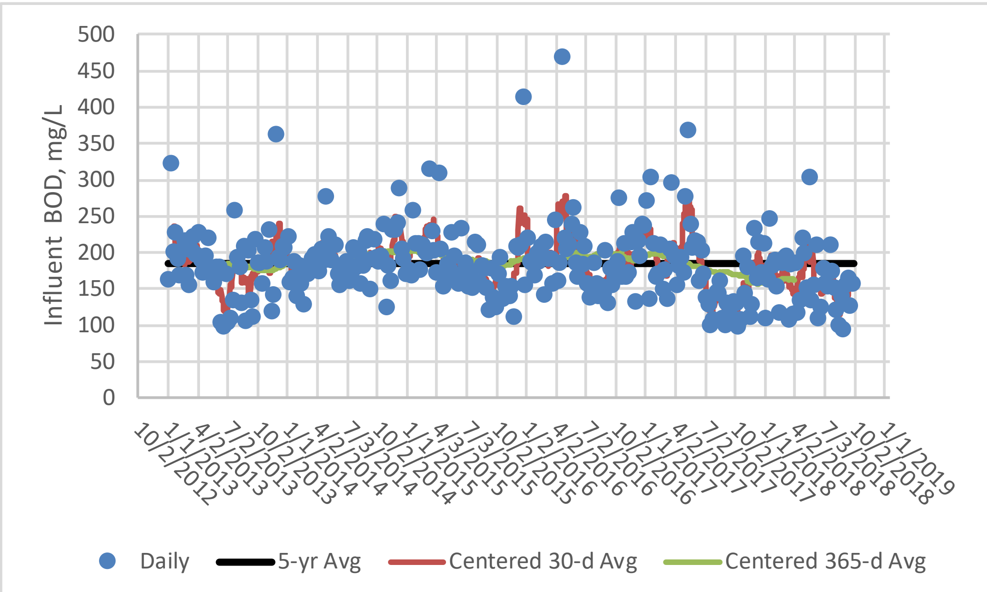


Figure 5-5 Influent BOD Concentrations

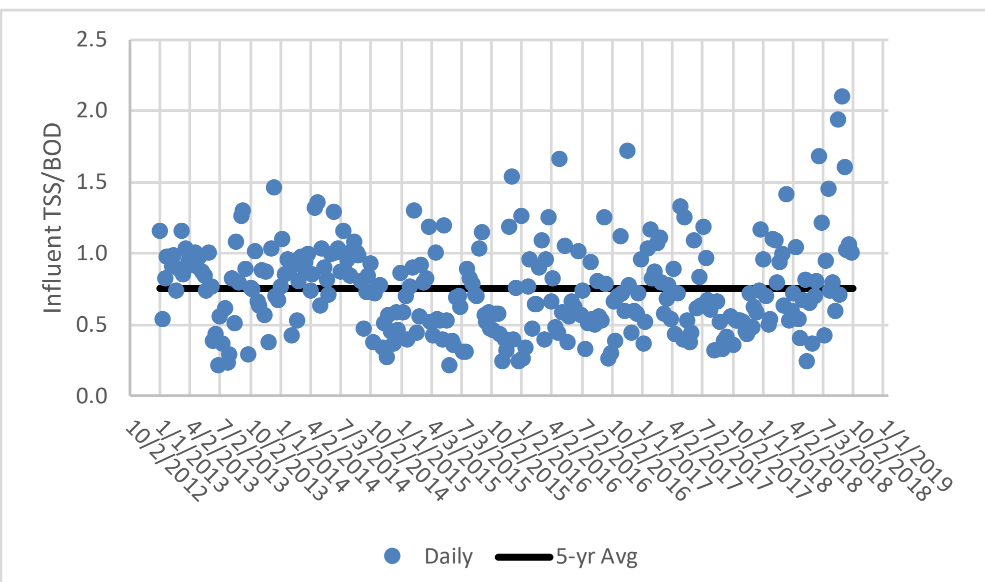


Figure 5-6 Influent TSS/BOD Ratio

5.1.5 Evaluation of Influent Ammonia-N Concentrations and Ammonia-N/BOD Concentration Ratios

Approximately two-years of influent ammonia-N concentration data were available from plant records. These data are shown graphically in Figure 5-7. As indicated in the figure, the concentrations were generally in the mid-30's at the beginning and end of the data period but were somewhat higher in the middle. The average of all the data shown is 36 mg/L.

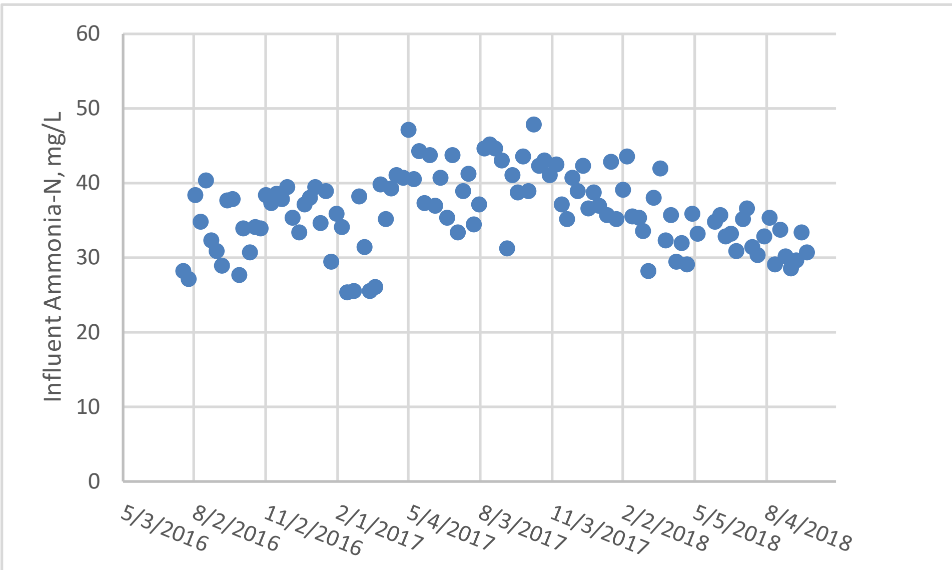


Figure 5-7 Influent Ammonia-N Concentrations

Ratios of Ammonia-N/BOD are shown in Figure 5-8. Key observations are listed below:

1. The Ammonia-N/BOD ratio has been highly variable, with values in late 2017 being substantially higher than those before and after. The reasons for such a trend are unknown, which makes it difficult to have confidence in the values.

2. The average Ammonia-N/BOD ratio for the period indicated was 0.22. This is considered to be extremely high. Normally, the influent TKN would be expected to be about 1.5 times the Ammonia-N, indicating a potential average TKN/BOD ratio near 0.33. For typical domestic wastewater, this value would be expected to be around 0.2. The apparent very high TKN/BOD ratio would adversely impact the ability of the secondary process to remove nitrogen as needed to meet the future Nitrate + Nitrite-Nitrogen limit of 10 mg/L, without supplemental carbon addition. Therefore, it is important that the TKN/BOD ratio be validated.

3. Problems with historical BOD and TSS data are discussed later in this section in connection with special monitoring studies.

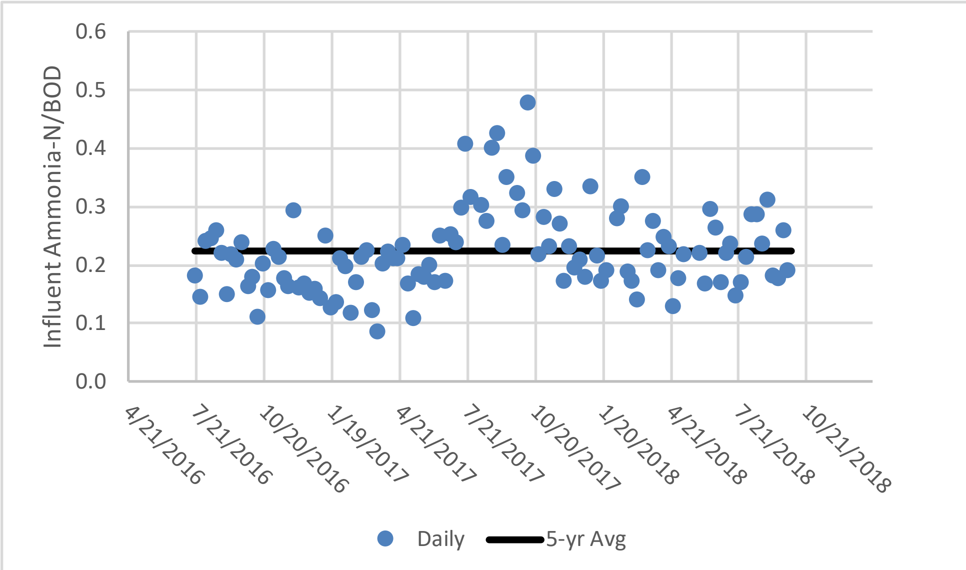


Figure 5-8 Influent Ammonia-N/BOD Ratio

5.1.6 Comparison of Recent Values to Previous Master Plan Values

A summary of recent average flows, BOD concentrations, and BOD loads for 2013 to 2018 taken from Figures 5-1 through 5-5 and the values contained in the previous Master Plan (February 2013 with updates through March 2016) is provided in Table 5-3.

When comparing April 2018 values to 2010 values from the previous Master Plan, it apparent that there have been very significant decreases in flows (1.8 to 1.33 Mgal/d) and apparent BOD loads (3002 to 1712 lb/d) in the eight years involved. Although flows can decrease due to water conservation and elimination of infiltration and inflow, BOD loads would not be expected to decrease with a stable or increasing population. As mentioned previously, problems with historical BOD data are discussed later in this section in connection with special monitoring studies.

Table 5-3 Summary of Recent and Master Plan Average Flows, BOD Load, and BOD Concentrations

	July	April	%	Exist Master Plan	
Parameter	2013	2018	Change	2010	Buildout
Annual Average Flow, Mgal/d	1.42	1.33	-6.3	1.8	2.37
Annual Average BOD Load, lb/d	2058	1712	-16.8	3002	3953
Annual Average BOD, mg/L	184	163	-11.4	200	200

5.2 SPECIAL INFLUENT MONITORING STUDIES

As presented in the previous subsection, there are several questionable attributes of the historical plant data, including the following:

1. The influent TSS/BOD ratio has been quite variable and much lower than would be expected for typical domestic wastewater (0.75 actual average versus 1.0 expected).
2. The Ammonia-N/BOD ratio has been highly variable and the implied TKN/BOD ratio is extremely high (apparent value near 0.33 versus around 0.2 expected).
3. The apparent annual average BOD load decreased 17% (2058 lb/d to 1712 lb/d) from July 2013 through April 2018. Furthermore, the April 2018 value represents a 43% decrease from the 2010 value established in the previous Master Plan (1712 lb/d compared to 3002 lb/d). A decrease in BOD load would not be expected with a stable or increasing population.

It was hypothesized that influent sampling methods could be leading to non-representative samples, thus skewing the results. In this regard, it was noted that the influent sampler intake strainer was located inside a larger perforated pipe (see Figure 5-9). Within the larger perforated pipe, quiescent conditions could be created, leading to settling and removal of solids before entering the sampler. This could lead to erroneously low results for TSS in particular, but also for BOD (and other constituents with particulate components like COD and TKN, which are discussed in subsequent paragraphs). Rag accumulations on the perforated pipe also could be causing particulates to be excluded from samples.

5.2.1 Special Influent Monitoring Study 1

To investigate the hypothesis of non-representative sampling caused by the perforated pipe shown in Figure 5-9, it was decided to conduct a special monitoring program with two independent flow proportional composite samplers. The existing “fixed sampler” would continue to be used with its sample intake inside the perforated pipe in accordance with historical practices. A second “portable sampler” would be used with its sample intake hanging freely in the flow stream (not protected inside a perforated pipe).

Daily influent samples from each of the two samplers were collected for approximately four weeks beginning in late January 2019. The constituents analyzed and the results are shown in Table 5-4. As shown in the table, the average influent TSS resulting from the portable sampler was only 70 mg/L, compared to 138 mg/L for the fixed sampler. Apparently, more solids were being excluded from the portable sampler than from the fixed sampler. However, if this was the case, then BOD and COD values should also be lower for the portable sampler as compared to the fixed sampler, but they were somewhat higher. Another perplexing factor is that ammonia-N concentrations were nearly the same or higher than TKN concentrations for both samplers. Since ammonia-N and organic-N comprise TKN, it is impossible for ammonia-N to be higher than TKN. Also, for typical domestic wastewater, the ammonia-N should be about 2/3 of the TKN.

While investigating the discrepancies, it was discovered that the portable sample intake strainer had been strapped to the outside of the perforated pipe used to protect the fixed sampler intake strainer and was not free-hanging in the flow stream. It was determined that this arrangement could cause non-representative sampling.

Because of the issues discussed above, it was determined that the results from Special Influent Monitoring Study 1 were likely unreliable. Therefore, Special Influent Monitoring Study 2 was planned.

Figure 5-9 Perforated Pipe Surrounding Sampler Intake Strainer

Table 5-4 Results from Special Influent Monitoring Study 1

		Concentration, mg/L													
		Ammonia-N		TKN		BOD		COD		TSS		VSS		FSS	
Date	Day	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable
1/21/2019	Monday	30	31	31	30	161	227	279	399	97	73	87	66	10	8
1/22/2019	Tuesday	26	27	33	31	107	181	344	556	247	103	219	87	28	16
1/23/2019	Wednesday	27	28	25		148	164	344	376	198	88	157	74	41	14
1/24/2019	Thursday	27	30	27	31	132	141	366	341	158	185	135	154	23	31
1/25/2019	Friday	26	28	36	33	211	157	551	371	239	87	156	87	73	ND
1/26/2019	Saturday	29	30	26	25	112	169	378	356	80	75	70	68	9	8
1/27/2019	Sunday	31	33	28	34	99	159	258	366	66	126	66	117	ND	9
1/28/2019	Monday	29	32	37	28	165	262	443	436	106	74	92	74	11	ND
1/29/2019	Tuesday	28	30	38	25	431	205	662	399	454	89	405	89	49	8
1/30/2019	Wednesday	26	35	35	46	144	127		394	219	77	199	71	20	7
1/31/2019	Thursday	27	38	35	33	178	174	353	424	127	79	112	72	15	7
2/1/2019	Friday	24	35	23	26	146	159	298	338	165	54	151	46	14	8
2/2/2019	Saturday	27	27	26	25	98	130	228	293	76	42	76	42	ND	ND
2/3/2019	Sunday	27	33	28	32	166	158	378	368	82	62	82	62	ND	ND
2/4/2019	Monday	29	36	28	33	178	232	403	618	101	66	101	59	ND	7
2/5/2019	Tuesday	24	30	22	24	118	141	238	323	122	58	110	50	12	8
2/6/2019	Wednesday	26	34	29	29	98	121		283	68	55	57	55	11	ND
2/7/2019	Thursday	27	37	25	34	162	193	253	373	71	43	63	43	9	ND
2/8/2019	Friday	27	35	23	31	124	123	278	298	117	48	110	42	8	7
2/9/2019	Saturday	26	35	31	36	76	122	221	309	64	41	57	41	7	ND
2/10/2019	Sunday	27	36	22	31	89	147	266	319	95	48	95	48	ND	ND
2/11/2019	Monday	26	36	24	32	129	150	311	326	114	58	104	51	10	7
2/12/2019	Tuesday	25	35	26	28	139	50	349	294	112	54	89	54	23	ND
2/13/2019	Wednesday	28	32	33	34	107	162		319	245	45	226	45	19	ND
2/14/2019	Thursday	21	31	25	34	97	108	361	478	94	68	85	58	9	10
2/15/2019	Friday	22	30	24	29	154	192	264	339	189	53	172	43	17	10
2/16/2019	Saturday	24	31	24	27	59	95	226	246	44	99	44	99	ND	ND
2/17/2019	Sunday	26	35	32	28	106	132	326	321	149	41	137	41	12	ND
2/18/2019	Monday	28	35	30	35	195	151	588	513	111	48	96	41	15	7
Sampler Weekday Avg (a		26	32	28	31	142	146	336	356	154	72	135	66	22	11
Sampler Weekend Avg (b		28	34	29	31	143	180	361	407	102	66	96	62	12	8
Sampler All Days Avg		27	33	29	31	142	156	344	372	138	70	123	65	20	10
Overall Average		30		30		149		359		104		94		15	

(a) Samples ending on Tuesday through Saturday mornings; each sample most representative of the previous day.

(b) Samples ending on Sunday and Monday mornings; each sample most representative of the previous day.

5.2.2 Special Influent Monitoring Study 2

For Special Influent Monitoring Study 2, two separate hypotheses were investigated: 1) whether the sampler intake configuration was excluding particulates in the wastewater, and 2) whether there could be issues with laboratory errors.

To address the first issue, the two samplers previously described would again be used. This time, it would be assured that the portable sampler intake strainer would be freely hanging in a well-mixed channel location away from the perforated pipe used for the fixed sampler (initially, both sampler intakes would still be in the turbulent discharge area of the Parshall flume used for influent flow measurement). To address the second issue, all samples would be sent to three different laboratories for analysis. The laboratories were FGL (the laboratory historically and routinely used), Caltest, and McCampbell.

Special Influent Monitoring Study 2 was initiated on March 28, 2019, with the first composite samples becoming available on March 29, 2019. Samples were taken daily through April 11, 2019. Unfortunately, the flow-proportional functioning of the portable sampler failed before the commencement of the study, so all portable sampler samples were timed composites throughout Special Influent Monitoring Study 2.

When the first sample was taken on Friday March 29, the portable sampler intake strainer was pulled up out of the flow stream for inspection, mainly to confirm whether the sampler intake strainer had accumulated any rags that could impair representative sampling. Unfortunately, major ragging was discovered, as shown in Figures 5-10 and 5-11. The sampler intake was cleaned and re-installed for weekend sampling. However, on Monday morning April 1, 2019, the portable sampler intake was again inspected and found to be covered with rags (see Figure 5-12). It was then clear that the sampler intake location at the discharge of the Parshall flume, which is upstream of the influent screen, would not be acceptable. Although the outside of the perforated pipe that houses the fixed sampler intake strainer could not be inspected while submerged, it is highly likely that rag accumulation is (and always has been) an issue there also.

To avoid ragging issues, it is preferable to install the influent sampler downstream from the influent screen to avoid ragging of the sampler intake. This was known and efforts were made as part of the previous Master Plan monitoring programs to install a sampler with its intake downstream of the screen. Unfortunately, the configuration of the screen channel is not suitable for sampling for two reasons: 1) the flow at this location is not turbulent and well-mixed, and 2) there is possible contamination of the sample with return activated sludge (RAS) that is introduced to the channel just downstream.

To mitigate the two issues downstream of the screen, it was decided to temporarily add concrete blocks inside the channel to create a high velocity turbulent flow that would provide good mixing and also protect against back-mixing of RAS. A photograph of the concrete blocks and sampler intake as first installed on April 1, 2019 is shown in Figure 5-13. On April 2 and 4, additional concrete blocks were added to optimize the sampler intake. The final layout is shown in Figure 5-14.

Figure 5-10 Portable Sample Intake Strainer Being Pulled Out of Flow Stream on 3-29-19

Figure 5-11 Rags Attached to Portable Sampler Intake Strainer on 3-29-19

Figure 5-12 Rags Being Removed from Portable Sampler Intake Strainer on 4-1-2019

Figure 5-13 Initial Configuration of Concrete Blocks and Sampler Intake Tube in Screen Channel on 4-1-2019

Figure 5-14 Final Configuration of Concrete Blocks and Sampler Intake Tube in Screen Channel on 4-4-2019.

5.2.2.1 Special Influent Monitoring Study 2 Results Overview

In the paragraphs below, the monitoring results are evaluated without consideration of data quality issues resulting from sample handling and timed composite sampling, which are covered in the subsequent subsection.

Tabulated results from Special Influent Monitoring Study 2 are shown in Tables 5-5 through 5-8. In the tables, the "select averages" include only data from April 3 through April 11 when the portable sampler intake was located downstream from the influent screen and believed to be free from ragging. The other data for the portable sampler is considered to be unusable. To allow comparison of the portable and fixed sampler data, select averages for the fixed sampler are also calculated.

Tables 5-5 through 5-7 present data for all of the main constituents of interest for this study, namely BOD, COD, TSS, VSS, Ammonia, and TKN. Nitrate and nitrite data are shown in Table 5-8. Although nitrate and nitrite are not expected to be present in domestic sewage, they were added to the study because, if present, they could interfere with TKN analysis. As indicated in Table 5-8, these constituents were either non-detect or at trace concentrations in all samples. No further consideration of nitrate and nitrite is included in this section.

A summary of the select average data from all three labs for both fixed and portable samplers is presented in Table 5-9. From Table 5-9, it can be noted that the concentrations of TSS and VSS from the portable sampler were approximately two times as high as those from the fixed sampler. This is considered to be clear evidence that particulates are being excluded from the fixed sampler, likely due to rag accumulation on the perforated pipe that protects the sampler intake strainer and possibly due also to solids settling inside the perforated pipe. It is further noted that BOD, COD, and TKN (TKN to a lesser extent) include both soluble and particulate components. Therefore, the concentrations of these constituents were also higher in the portable sampler than in the fixed sampler, but to a lesser extent than TSS and VSS, which are entirely particulate by definition. Ammonia results for the fixed and portable samplers were only slightly different because ammonia is soluble and not removed with particulates.

Based on the results described above, it is believed that the entire historical database of wastewater constituent concentrations, which are based on the fixed sampler, are compromised. For example, as shown in Table 5-9, the select average BOD result for the portable sampler is almost 40% higher than that for the fixed sampler (248 mg/L vs 181 mg/L). This may provide a good indication as to the general magnitude by which historical plant BOD records, which are all based on the fixed sampler location, could be skewed low. Similarly, actual influent TSS concentrations could be perhaps double those recorded.

While the likely issues associated with the fixed sampler results were not fully revealed in the previous Master Plan, it was recognized in that plan that the low values indicated in plant records for BOD and TSS were problematic and questionable. Because of this, BOD and TSS concentrations substantially higher than those indicated in plant records were adopted as the basis for the Master Plan after consideration of the District population and expected per capita BOD contributions.

Table 5-5 Special Influent Monitoring Study 2 Results – BOD and COD

		BOD, mg/L						COD, mg/L (a)					
		FGL		CalTest		McCampbell		FGL		CalTest		McCampbell	
Date	Comment	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable
3/29/19	Portable Sampler at Flume, Ragged	163	184	273	380	97	250	543	366	874	1220	360	850
3/30/19	Portable Sampler at Flume, Presume Ragged	129	164	125	167	89	120	306	356	465	467	310	350
3/31/19	Portable Sampler at Flume, Presume Ragged	201	259	245	284	200	180	598	601	690	811	480	550
4/1/19	Portable Sampler at Flume, Ragged	168	214	207	245	200	210	491	506	888	891	490	570
4/2/19	Portable Sampler After Screen, Layout 1, Slight Rags	162	314	166	381	170	200	401	738	534	1030	330	550
4/3/19	Portable Sampler After Screen, Layout 2	158	285	134	268	150	160		603	553	525	160	290
4/4/19	Portable Sampler After Screen, Layout 2	169	233	127	297	150	210	384	566	616	753	380	510
4/5/19	Portable Sampler After Screen, Layout 2, No Rags	236	195	218	273	140	150	488	623	661	930	280	350
4/6/19	Portable Sampler After Screen, Layout 3	145	240			120	170	324	556			220	310
4/7/19	Portable Sampler After Screen, Layout 3	126	285	170	459	120	130	371	603	404	1270	330	530
4/8/19	Portable Sampler After Screen, Layout 3	375	186	184	294	220	280	558	461	529	1010	370	820
4/9/19	Portable Sampler After Screen, Layout 3	210	236	142	576	140	160	526	496	485	1360	320	260
4/10/19	Portable Sampler After Screen, Layout 3	328	270	231	186	180	220		571	607	506	410	420
4/11/19	Portable Sampler After Screen, Layout 3	205	234	154	301	180	140	663	671	578	831	500	540
Average		198	236	183	316	154	184	471	551	606	893	353	493
Select Average (b)		217	240	170	332	156	180	473	572	554	898	330	448
Select Avg. All Fixed		181						447					
Select Avg. All Portable		248						629					
Select Avg. Overall		214						542					

(a) Darker highlighted data for Caltest represents average of re-analysis results.

(b) Select average includes only non-ragging data from 4/3/19 to 4/11/19.

Table 5-6 Special Influent Monitoring Study 2 Results – TSS and VSS

		TSS, mg/L						VSS, mg/L					
		FGL		CalTest		McCampbell		FGL		CalTest		McCampbell	
Date	Comment	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable
3/29/19	Portable Sampler at Flume, Ragged	214	63	560	543	100	394	205	63	533	513	92	364
3/30/19	Portable Sampler at Flume, Presume Ragged	62	56	55	65	53	78	55	49	50	59	48	71
3/31/19	Portable Sampler at Flume, Presume Ragged	244	241	169	198	203	208	218	221	160	189	182	192
4/1/19	Portable Sampler at Flume, Ragged	194	186	228	249	222	242	177	168	212	228	207	230
4/2/19	Portable Sampler After Screen, Layout 1, Slight Rags	187	313	160	506	182	117	163	283	156	460	164	100
4/3/19	Portable Sampler After Screen, Layout 2	74	247	158	252	97	15	74	223	145	240	80	14
4/4/19	Portable Sampler After Screen, Layout 2	83	174	63	260	113	229	83	166	58	753	104	281
4/5/19	Portable Sampler After Screen, Layout 2, No Rags	310	231	213	717	86	102	283	208	207	354	76	94
4/6/19	Portable Sampler After Screen, Layout 3	83	163			51	143	55	163			46	135
4/7/19	Portable Sampler After Screen, Layout 3	71	283	80	560	64	157	62	265	64	503	59	142
4/8/19	Portable Sampler After Screen, Layout 3	281	119	100	480	90	413	266	110	90	447	80	380
4/9/19	Portable Sampler After Screen, Layout 3	115	196	124	820	101	111	108	184	110	784	96	107
4/10/19	Portable Sampler After Screen, Layout 3	288	283	260	163	382	258	262	263	250	150	353	243
4/11/19	Portable Sampler After Screen, Layout 3	223	379	115	440	165	108	207	355	113	410	159	105
Average		174	210	176	404	136	184	158	194	165	392	125	176
Select Average (a)		170	231	139	462	128	171	156	215	130	455	117	167
Select Avg. All Fixed		146						134					
Select Avg. All Portable		281						272					
Select Avg. Overall		213						203					

(a) Select average includes only non-ragging data from 4/3/19 to 4/11/19.

Table 5-7 Special Influent Monitoring Study 2 Results – Ammonia and TKN

Date	Comment	Ammonia as N, mg/L						TKN as N, mg/L (a)					
		FGL		CalTest		McCampbell		FGL		CalTest		McCampbell	
		Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable
3/29/19	Portable Sampler at Flume, Ragged	31	32	29	31	28	31	35	24	77	78	39	43
3/30/19	Portable Sampler at Flume, Presume Ragged	29	26	35	31	33	30	37	23	45	41	33	32
3/31/19	Portable Sampler at Flume, Presume Ragged	32	32	34	32	33	41	31	27	93	77	35	37
4/1/19	Portable Sampler at Flume, Ragged	31	38	33	41	32	30	32	49	75	74	38	34
4/2/19	Portable Sampler After Screen, Layout 1, Slight Rags	28	25	32	34	29	33	24	42	69	90	42	36
4/3/19	Portable Sampler After Screen, Layout 2	29	35	32	40	31	41	29	35	51	61	35	39
4/4/19	Portable Sampler After Screen, Layout 2	30	37	32	41	32	40	28	47	46	59	38	42
4/5/19	Portable Sampler After Screen, Layout 2, No Rags	29	36	32	41	29	38		42	54	62	33	45
4/6/19	Portable Sampler After Screen, Layout 3	31	32			37	36	33	40			29	33
4/7/19	Portable Sampler After Screen, Layout 3	32	31	34	36	38	33	46	39	50	95	46	39
4/8/19	Portable Sampler After Screen, Layout 3	34	38	35	41	36	41	34	44	49	66	54	69
4/9/19	Portable Sampler After Screen, Layout 3	32	27	33	31	31	35	35	21	51	130	41	53
4/10/19	Portable Sampler After Screen, Layout 3	32	38	33	40	32	39	32	36	56	57	45	54
4/11/19	Portable Sampler After Screen, Layout 3	28	34	31	37	32	38	43		45	92	34	45
Average		31	33	33	37	32	36	34	36	59	76	39	43
Select Average (b)		31	34	33	38	33	38	35	38	50	78	39	47
Select Avg. All Fixed		32						41					
Select Avg. All Portable		37						54					
Select Avg. Overall		34						48					

(a) Darker highlighted data for Caltest represents average of re-analysis results.

(b) Select average includes only non-ragging data from 4/3/19 to 4/11/19.

Table 5-8 Special Influent Monitoring Study 2 Results – Nitrate and Nitrite

		Nitrate as N, mg/L						Nitrite as N, mg/L					
		FGL		CalTest		McCampbell		FGL		CalTest		McCampbell	
Date	Comment	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable	Fixed	Portable
3/29/19	Portable Sampler at Flume, Ragged	0.20	0	0.13	ND	0.29	ND	0.14	ND	0.31	ND	0.43	ND
3/30/19	Portable Sampler at Flume, Presume Ragged	0.07	0.02	ND	ND	ND	ND	0.08	0.02	ND	ND	ND	ND
3/31/19	Portable Sampler at Flume, Presume Ragged	0.10	ND	ND	ND	ND	ND	0.02	0.03	ND	ND	ND	ND
4/1/19	Portable Sampler at Flume, Ragged	0.09	0.07	ND	ND	ND	ND	0.02	0.02	ND	ND	ND	ND
4/2/19	Portable Sampler After Screen, Layout 1, Slight Rags	0.60	ND	ND	ND	ND	ND	0.02	0.02	ND	ND	ND	ND
4/3/19	Portable Sampler After Screen, Layout 2	ND	ND	ND	ND	ND	ND	ND	0.02	ND	ND	ND	ND
4/4/19	Portable Sampler After Screen, Layout 2	ND	ND	ND	ND	ND	ND	0.02	0.02	ND	ND	ND	ND
4/5/19	Portable Sampler After Screen, Layout 2, No Rags	0.10	ND	ND	ND	ND	ND	0.08	ND	ND	ND	ND	ND
4/6/19	Portable Sampler After Screen, Layout 3	0.09	0.10			ND	ND	0.02	0.02			ND	ND
4/7/19	Portable Sampler After Screen, Layout 3	0.10	0.10	ND	ND	ND	ND	0.02	0.02	ND	ND	ND	ND
4/8/19	Portable Sampler After Screen, Layout 3	0.10	0.08	ND	ND	ND	ND	0.02	0.02	ND	ND	ND	ND
4/9/19	Portable Sampler After Screen, Layout 3	ND	0.02	ND	ND	ND	ND	ND	0.02	ND	ND	ND	ND
4/10/19	Portable Sampler After Screen, Layout 3	ND	ND	ND	ND	ND	ND	ND	0.02	ND	ND	ND	ND
4/11/19	Portable Sampler After Screen, Layout 3	0.08	0.08	ND	ND	ND	ND	0.01	0.02	ND	ND	ND	ND

Table 5-9 Special Influent Monitoring Study 2 Results – Summary

	Concentration, mg/L						Concentration Ratio				
Description	BOD	COD	TSS	VSS	Amm.-N	TKN	COD/BOD	TSS/BOD	VSS/TSS	TKN/BOD	Amm/TKN
Select Average, Fixed Sampler, All Labs	181	447	146	134	32	41	2.46	0.80	0.92	0.23	0.78
Select Average, Portable Sampler, All Labs	248	629	281	272	37	54	2.54	1.13	0.97	0.22	0.68
Ratio Portable/Fixed	1.37	1.41	1.93	2.03	1.14	1.30	1.03	1.41	1.05	0.95	0.88

5.2.2.2 Special Influent Monitoring Study 2 Data Quality Issues

While the results from Special Influent Monitoring Study 2 are highly significant and informative with regard to issues associated with the fixed sampler and while the select portable sampler results are believed to be much more reliable than the fixed sampler results, the portable sampler results are not considered to be fully reliable as a basis upon which to base the Master Plan Update. There are several issues as noted below:

1. Only the portable sampler results from April 3 to April 11 are considered to be useful. These nine days of data, even if accurate and representative of the actual influent wastewater characteristics on those nine days, comprise only a brief snapshot of the Discovery Bay wastewater and cannot be considered to be long-term averages. Furthermore, although general comparisons between fixed and portable sampler results have been presented, these comparisons do not provide an accurate basis for adjusting historical plant records.

2. The portable sampler was operated on a timed composite basis, rather than the desired flow-proportional composite basis. With timed composite samples, sample portions taken when flows and concentrations could be low (likely in the late night and early morning hours) are given equal weighting to sample portions taken when flows and concentrations are high (likely in the middle of the day and early evening). This could lead to erroneously low constituent concentrations.

3. There were large discrepancies between the results developed by the three laboratories used for this study, indicating a likely problem of inadequate mixing during sample splitting.

Further discussion of Item 3 above is provided in the following paragraphs.

Comparisons of the analysis results from the three laboratories for the six key constituents are shown graphically in Figure 5-15, are summarized in Table 5-10, and are discussed below. Because portable sampler results are considered most relevant, only those results are shown in the figure. However, similar comparisons could be made for the fixed sampler results, which have been presented in a tabular format (Tables 5-5 through 5-7).

From the graphs shown in Figure 5-15 and from the summary data presented in Table 5-10, it can be noted that there are large discrepancies between the results obtained from each of the three laboratories. Ideally, all three labs would agree on the concentration of the same constituent in the same sample. In that case, the three data series shown in each graph would overlay each other. It is recognized that ideal is impossible and that there would be reasonable variations between the laboratories. However, the variations shown in Figure 5-15 are far more significant and troubling. Furthermore, similar to variations between fixed and portable samplers discussed previously, the variations shown in Figure 5-15 appear to be related to particulate content. For example, the variabilities in TSS and VSS, which are entirely comprised of particulate matter, are more substantial than those for BOD, which is partly soluble and partly particulate. The variability in ammonia, which is totally soluble, is the lowest. However, the variability seen in the TKN and COD data appears to be more pronounced than would be expected compared to the variability exhibited in data for BOD, TSS, and VSS (COD variability should be similar to BOD variability, while TKN variability should be lower because about 2/3 of TKN is soluble ammonia).

One possible explanation for the variability described above is that the samples may not have been adequately mixed while splitting portions out into sample bottles for shipment to each of the three laboratories. The main sample container was poured into two sample bottles for each of the three labs - one sample bottle for COD, TKN, and Ammonia, and one sample bottle for the remaining constituents - for a total of six sample bottles. Therefore, if the sample was not adequately mixed before and during sample splitting, it is possible for the COD/TKN/Ammonia sample to be impacted differently than the sample for the remaining constituents for a given lab and it is possible for the samples sent to the various labs to be impacted differently. If inadequate mixing occurred during any of the splits, then none of the three laboratory results for any of the analytes would be accurate. Results for constituents with particulate components (BOD, COD, TSS, VSS, TKN) would be skewed low in sample portions with less than average solids content, while results would be skewed high for sample portions with more than average solids content (i.e., the dregs of the sample bottle).

From the graphs shown in Figure 5-15, it can be seen that the results from FGL and McCampbell were generally in closest agreement, while those for Caltest were generally much higher. It is understood that the Caltest samples were poured last.

5.2.2.3 Special Influent Monitoring Study 2 Summary and Recommendations

Considering the data quality issues discussed above, and without the benefit of any new higher-quality data, it is difficult to determine reliable average constituent concentrations for existing conditions. However, for now, engineering judgement can be used to provide best estimates of values for use in the Master Plan. These suggested values are included in Table 5-10. The development of these values is discussed below.

<u>BOD.</u> The average BOD measured by the three laboratories ranges from 180 to 332 mg/L (average = 251 mg/L). These values could be skewed low by an unknown fraction (likely less than 10%) due to flow proportional sampling. Furthermore, the wastewater characteristics during the brief special monitoring effort do not necessarily represent average conditions.

Another estimate of the average BOD can be developed based on the District population and estimated per capita BOD load contributions, such as was done for the previous Master Plan. Based on the 2010 census and the number of new service connections added within the District since 2010, the estimated effective District population as of March 31, 2018 (the last date for which the average annual flow was calculated and shown in Figure 5-1), is approximately 15,500. Using an estimated per capita BOD load of 0.22 lb/d (from 10 States Standards for communities with in-sink grinders), the estimated total BOD load to the plant would be 3,410 lb/d. If this load occurred with the March 31, 2018 average annual flow of 1.32 Mgal/d, the BOD concentration would be 310 mg/L. Since this value is a rough estimate only and is much higher than the average value measured by the three labs (251 mg/L), the suggested value for the Master Plan is 275 mg/L (this equates to about 0.195 lb/d per person). It is reasonable to consider that the per capita BOD load for Discovery Bay could be somewhat lower than “typical” communities because many people in Discovery Bay work outside the community and contribute a portion of their daily BOD load elsewhere.

<u>COD.</u> The suggested average value for the COD concentration is 688 mg/L. This is based on a suggested COD/BOD ratio of 2.5, which is generally consistent with the average value determined from the three laboratories and is consistent with typical domestic wastewater (per Metcalf and Eddy/AECOM, Wastewater Engineering, Fifth Edition).

<u>TSS.</u> The suggested average value for TSS is based on a typical domestic wastewater TSS/BOD ratio of 1.0, which is generally consistent with the values indicated in Table 5-10. This gives an average TSS concentration of 275 mg/L.

<u>VSS.</u> A typical VSS/TSS ratio for domestic wastewater is around 0.80. However, the range indicated for the three labs in Table 5-10 is 0.93 to 0.99, with an average of 0.97. Tentatively, a value of 0.95 is suggested, but further evaluation of this parameter may occur during process analysis. Therefore, the initial estimated average VSS concentration is 261 mg/L.

<u>Ammonia-N.</u> Since ammonia is soluble, its concentration should not have been impacted by sample mixing and splitting operations. This is undoubtedly why the three laboratories were in reasonably close agreement regarding ammonia-N concentrations. Accordingly, it is appropriate to use the average value determined by the three laboratories, which is 37 mg/L. This is in close agreement with the average influent ammonia-N concentration of 36 mg/L recorded in plant records for the period from mid-2016 to mid-2018 (data shown in Figure 5-7).

<u>TKN.</u> For typical domestic wastewater, the ammonia-N/TKN ratio is around 0.66 (default value in BioWin process simulator). The range measured by the three laboratories and shown in Table 5-10 is 0.49 to 0.90, with an average of 0.68. This is an extremely important parameter for nitrification and denitrification design, so it is disconcerting to not have more certainty on its value. At this time, the suggested average TKN value is 55 mg/L, based on an ammonia-N/TKN ratio of 0.67. The resultant average TKN/BOD ratio is 0.20. The BioWin process simulator default value for this ratio is only 0.16, while a typical value indicated by Metcalf and Eddy/AECOM (Wastewater Engineering, Fifth Edition) is 0.18. Therefore, the suggested TKN/BOD ratio of 0.20 is somewhat higher than expected for typical domestic wastewater, but slightly lower than the average value of 0.22 measured by the three labs for this study.

The suggested average constituent concentration values indicated in Table 5-10 are believed to be reasonable current values to be used as the basis for projecting future flows and loads upon which the Master Plan will be based. However, it is highly recommended that the District proceed as soon as possible to institute permanent improvements that would allow reliable representative sampling downstream from the influent screen. Additionally, sample handling protocols should be reviewed and modified as needed. In particular, it is recommended that the large sample jug that comes from the automatic sampler be vigorously mechanically mixed while sample portions are transferred by pumping or are discharged from a spigot to be added near the bottom of the jug. Alternatively, the entire jug contents could be poured into another container better suited for mechanical mixing while withdrawing sample portions. Once the improvements and sample handling procedures are implemented, regular flow-proportional composite influent sampling should be completed on at least three days per week and samples should be analyzed for BOD, COD, TSS, VSS, Ammonia-N, and TKN until a reliable influent database can be developed. The reliable data should be used for final design of improvements.

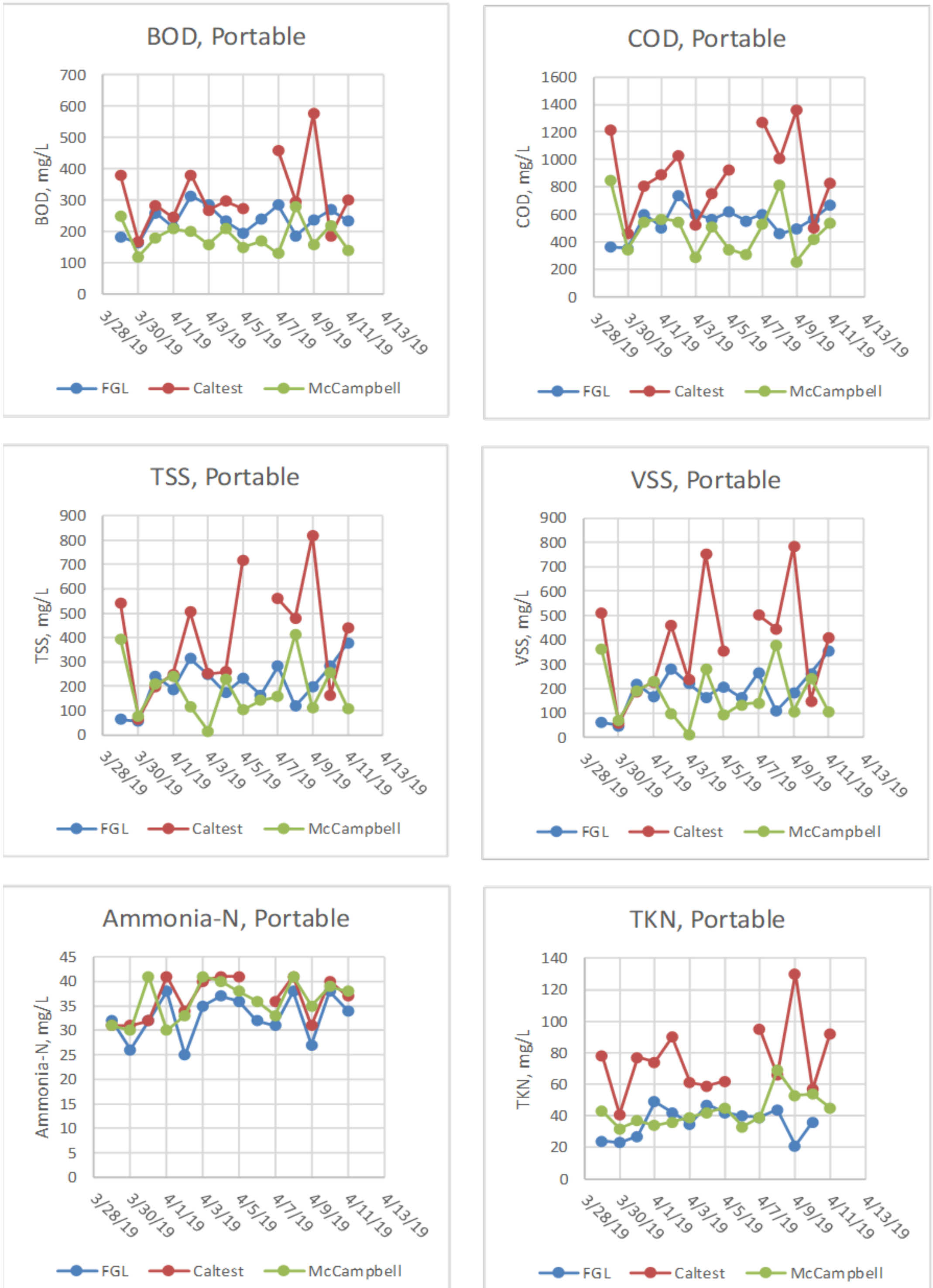


Figure 5-15 Comparison of Laboratory Results for the Six Main Constituents (Portable Sampler Only)

Table 5-10 Summary of Average Portable Sampler Constituent Concentrations and Suggested Values for Master Plan

	Concentration, mg/L						Concentration Ratio				
Description (a)	**BOD**	**COD**	**TSS**	**VSS**	**Amm.-N**	**TKN**	**COD/BOD**	**TSS/BOD**	**VSS/TSS**	**TKN/BOD**	**Amm/TKN**
Select Average, Portable Sampler, FGL	240	572	231	215	34	38	2.38	0.96	0.93	0.16	0.90
Select Average, Portable Sampler, McCampbell	180	448	171	167	38	47	2.49	0.95	0.98	0.26	0.81
Select Average, Portable Sampler, Caltest	332	898	462	455	38	78	2.71	1.39	0.99	0.24	0.49
Select Average, Portable Sampler, All Labs	251	639	288	279	37	54	2.55	1.15	0.97	0.22	0.68
Select Average, Portable Sampler, FGL & McC.	210	510	201	191	36	43	2.43	0.95	0.95	0.20	0.85
Suggested Value for Master Plan	275	688	275	261	37	55	2.50	1.00	0.95	0.20	0.67

(a) Select average includes only non-ragging data from 4/3/19 to 4/11/19.

The suggested average constituent concentration values indicated in Table 5-10 are approximately 38% higher than those developed for existing conditions in the previous Master Plan (e.g., BOD = 275 vs 200 mg/L and TKN = 55 vs 40 mg/L). The increased concentrations are due mostly to water conservation resulting in previously existing wastewater constituent loads being carried in less water. At the time of the previous Master Plan, the average annual flow was 1.8 Mgal/d, which is 36% higher than the current value of 1.32 Mgal/d (as of March 31, 2018). A secondary factor that has resulted in increased concentrations is that the District population has increased (resulting in higher constituent loads) even while the flows have been decreasing.

5.3 INCREMENTAL FLOWS FROM FUTURE GROWTH

Future residential and non-residential growth projections for TDBCSD are included in Section 3 and can be used as the basis of calculating incremental flows from future growth.

Flows from future residential connections can be estimated based on typical values for existing customers. Based on District records, there were 5497 equivalent primary residential households on March 31, 2018, when the annual average flow was 1.32 Mgal/d. Based on District water use records, it is estimated that approximately 98 percent of the District's sewage flow is residential, indicating an estimated annual average residential flow of approximately 1.29 Mgal/d on March 31, 2018. Therefore, the annual average sewage flow per equivalent primary residence is estimated to be 235 gpd.

Flows from future commercial and business park / office connections can be estimated using the City of Brentwood development standards of 1600 and 2000 gallons per acre per day, respectively (average annual flow).

Based on the above, incremental average annual flows from projected growth within TDBCSD are shown in Table 5-11.

Table 5-11 Average Annual Flows from Projected Growth

Development Type	Units	Number	Sewage Generation Rate, gpd/unit	Projected Flow, gpd
Residential	Homes	1208	235	283,880
Commercial	Acres	5	1,600	8,000
Business Park / Office	Acres	8.2	2,000	16,400
Total				308,280 round to 310,000

5.4 SUMMARY OF EXISTING AND FUTURE DESIGN FLOWS AND LOADS

Based on the existing flows and loads and the incremental flows from future growth established above, existing, future incremental and future total flows and loads are summarized in Table 5-12. For the Baseline Future condition shown in Table 5-12, it is presumed that per-capita flow rates will remain the same as existing ([235 gpd/home]/[2.816 people per home] = 83.5 gpd, average) and that wastewater constituent concentrations and flow and load variability for future growth will be the same as existing. An Alternate Future condition is shown based on the possibility of extreme water conservation and average per capita sewage flows decreasing to 50 gal/d. For the Alternate Future, constituent loads are assumed to be the same as the Baseline Future, resulting in much higher constituent concentrations.

Considering the discussion above, an alternative to considering plant capacity in terms of flow is to consider plant capacity is in terms of the population equivalents (PE) that can be served. Although the flows will vary with water conservation, loads will likely remain about the same. This is because a person, on average, contributes a fixed BOD load (e.g., 0.195 lb/d), regardless of how much water the person uses. Therefore, the average design BOD load of 3738 lb/d indicated in Table 5-12 represents a PE of approximately 19,000 at 0.195 lb/d per person.

In actuality, plant capacity depends both on peak flows and peak loads; therefore, neither flow nor load alone can be used to accurately represent capacity.

There are substantial plant capacity implications associated with using the Alternate Future scenario versus the Baseline Future scenario. These implications vary from process to process, depending on the extent to which the process is designed based on flow versus load and on whether the capacity is expressed on the basis of flow or on the basis of PE. For example, the oxidation ditches are sized based mostly on load (but also somewhat on flow due to their interrelationship with the clarifiers). Under the Alternate Future scenario, the load remains the same, but the flow is much lower than in the Baseline Future scenario; therefore, the oxidation ditches will have a much lower flow capacity but perhaps a slightly higher PE capacity under the Alternate Future scenario. On the other hand, pumping systems, the filters, and the UV system are designed based on flow; therefore, with decreasing flows such as in the Alternate Future scenario, the capacities of existing facilities in terms of PE would be much greater than under the Baseline Future scenario.

In general, for existing facilities or for a given set of improvements, it would be expected that the capacity of each unit process in terms of PE would be the same or higher under the Alternate Future scenario than under the Baseline Future scenario. Therefore, it should generally be conservative to base the Master Plan on the Baseline Future scenario. The number of houses and people that can be served by the plant would not be expected to decrease with water conservation. However, there might be specific instances where slight modifications in facilities and/or operations would be warranted.

Table 5-12 Existing and Future Flows and Loads

Parameter (a)	Existing (b)	Increment (c)	Baseline Future (d)	Alternate Future (e)	Previous Master Plan Future (f)
Flow Ratios					
ADWF/AAF	1.0	1.0	1.0	1.0	0.97
ADMMF/AAF	1.2	1.2	1.2	1.3	1.1
PDF/AAF	2.1	2.1	2.1	2.8	2.0
PHF/AAF	3.0	3.0	3.0	4.3	3.0
Load Ratios					
ADMML/AAL	1.3	1.3	1.3	1.3	1.3
PDL/AAL	2.0	2.0	2.0	2.0	2.0
Flow, Mgal/d					
ADWF	1.32	0.31	1.63	0.98	2.35
AAF	1.32	0.31	1.63	0.98	2.42
ADMMF	1.58	0.37	1.96	1.30	2.66
PDF	2.77	0.65	3.42	2.77	4.84
PHF	3.96	0.93	4.89	4.24	7.26
Annual Average Load, lb/d					
BOD	3,027	711	3,738	3,738	4,037
TSS	3,027	711	3,738	3,738	4,037
TKN	605	142	748	748	807
Average Day Maximum Monthly Load, lb/d					
BOD	3,936	924	4,860	4,860	5,248
TSS	3,936	924	4,860	4,860	5,248
TKN	787	185	972	972	1,050
Average Constituent Concentrations, mg/L					
BOD	275	275	275	459	200
TSS	275	275	275	459	200
TKN	55	55	55	92	40
Constituent Concentrations with ADMMF and ADMML,					
BOD	298	298	298	448	236
TSS	298	298	298	448	236
TKN	60	60	60	90	47
Constituent Concentrations with AAF and ADMML, mg/L					
BOD	358	358	358	597	260
TSS	358	358	358	597	260
TKN	72	72	72	119	52

(a) ADWF = Average Dry Weather Flow, AAF = Annual Average Flow,
ADMMF = Average Day Maximum Monthly Flow,
PDF = Peak Day Flow, PHF = Peak Hour Flow
AAL = Annual Average Load, ADMML = Average Day Maximum Monthly Load

(b) Based on AAF = 1.32 Mgal/d as of March 31, 2018.

(c) Average incremental flow from Table 5-11.

(d) Baseline future presumes per capita flows remain same as existing (83.5 gal/d, average). Flow and load peaking factors assumed same as existing.

(e) Alternate Future presumes exteme water conservation with average per capita flow of 50 gal/d. Differences between average flows and peak flows assumed same as Baseline Future. Flow peaking factors adjusted per above. Loads assumed same as Baseline Future.

(f) Final Master Plan dated February 13, 2013, Including Amendment 1.

6.0 OVERVIEW OF EXISTING WASTEWATER TREATMENT PLANT

In this section, the existing wastewater treatment plant is described and discussed, including presentation of flow schematics, hydraulic profiles, and key design criteria. Also discussed are known issues of concern.

6.1 DESCRIPTION OF EXISTING FACILITIES

The wastewater treatment plant currently includes an influent pump station, influent screening, secondary treatment facilities using oxidation ditches, tertiary filtration, and ultraviolet (UV) disinfection prior to export pumping for discharge into Old River. Waste sludge is aerobically digested, dewatered using belt filter presses, and dried in active solar drying units before landfill disposal.

The overall treatment system is arranged in two distinct areas, referred to as Plant 1 and Plant 2. Plant 1 is located about ¼ mile north of Highway 4 within the Discovery Bay Development area, while Plant 2 is located immediately south of Highway 4. The two plants are interconnected and are dependent upon each other for various functions. Plant 1 was the original plant, which was started as a pond treatment system. Over the years, Plant 1 was upgraded to its current configuration with an oxidation ditch for secondary treatment. Plant 2 was originally constructed in the years 2000 and 2001 and has undergone several upgrades since then. With the Secondary Improvements Project completed in 2016, Plant 2 now includes two oxidation ditches and three secondary clarifiers. This has allowed Plant 1 to be taken out of service under normal operations and with existing flows and loads (see Section 11 for further discussion on future use of Plant 1).

The influent pump station that serves both plants is located on the Plant 1 site. The discharge from the influent pump station can be split as needed between Plant 1 and Plant 2, depending on which facilities are in service within the two plants. Independent influent screening and secondary treatment facilities exist at both plants. The secondary effluent from both plants is combined within Plant 2 for tertiary filtration, UV disinfection, and export pumping for discharge to Old River. All of the sludge handling facilities for both plants are located at Plant 2.

Site plans for Plant 1 and Plant 2 are shown in Figures 6-1 and 6-2, respectively. Copies of Construction Drawings G-2 and G-3 from the Effluent Filtration Project, dated April 2015 (the most recent major liquid stream treatment project) are presented in Figures 6-3 through 6-5 to show plant flow schematics and hydraulic profiles. Design criteria for the various facilities are discussed in the corresponding sections of this Master Plan document.

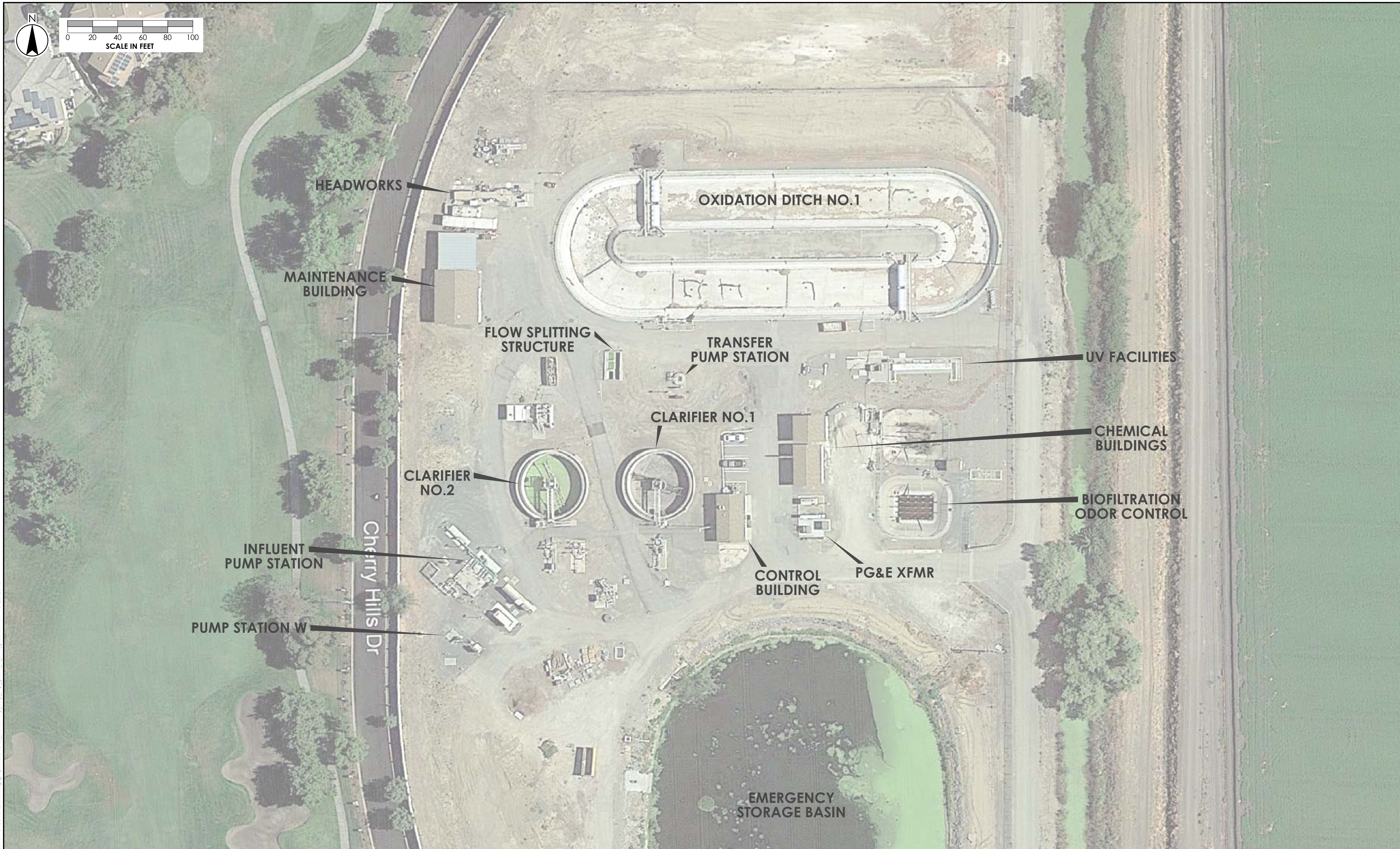
6.2 EXISTING PLANT PERFORMANCE

The existing wastewater treatment plant provides a tertiary level of treatment to meet key discharge requirements as follows:

Biochemical Oxygen Demand (BOD_5, average monthly) ≤ 10 mg/L
Total Suspended Solids (average monthly) ≤ 10 mg/L
Ammonia Nitrogen (maximum daily) ≤ 8.4 mg/L
Nitrate Plus Nitrite Nitrogen (maximum daily) ≤ 31 mg/L
Total Coliform Organisms (weekly median) ≤ 23 per100 mL Most Probable Number

The ammonia, nitrate plus nitrite, and total coliform permit requirements indicated above are interim effluent limitations, with more stringent requirements set to take effect by the end of 2022 for total coliform and by the end of 2023 for ammonia and nitrate plus nitrite. The tertiary filters needed to meet the more stringent total coliform requirement of 2.2 MPN/100 mL as a weekly median have already been constructed and are in operation. The facilities needed to meet the more stringent ammonia nitrogen (0.7 mg/L monthly average) and nitrate plus nitrite nitrogen (10 mg/L monthly average) requirements have not yet been designed or constructed and are discussed in detail in Section 11.

In general, the plant is successful in meeting the existing discharge requirements, as discussed further in Section 8.



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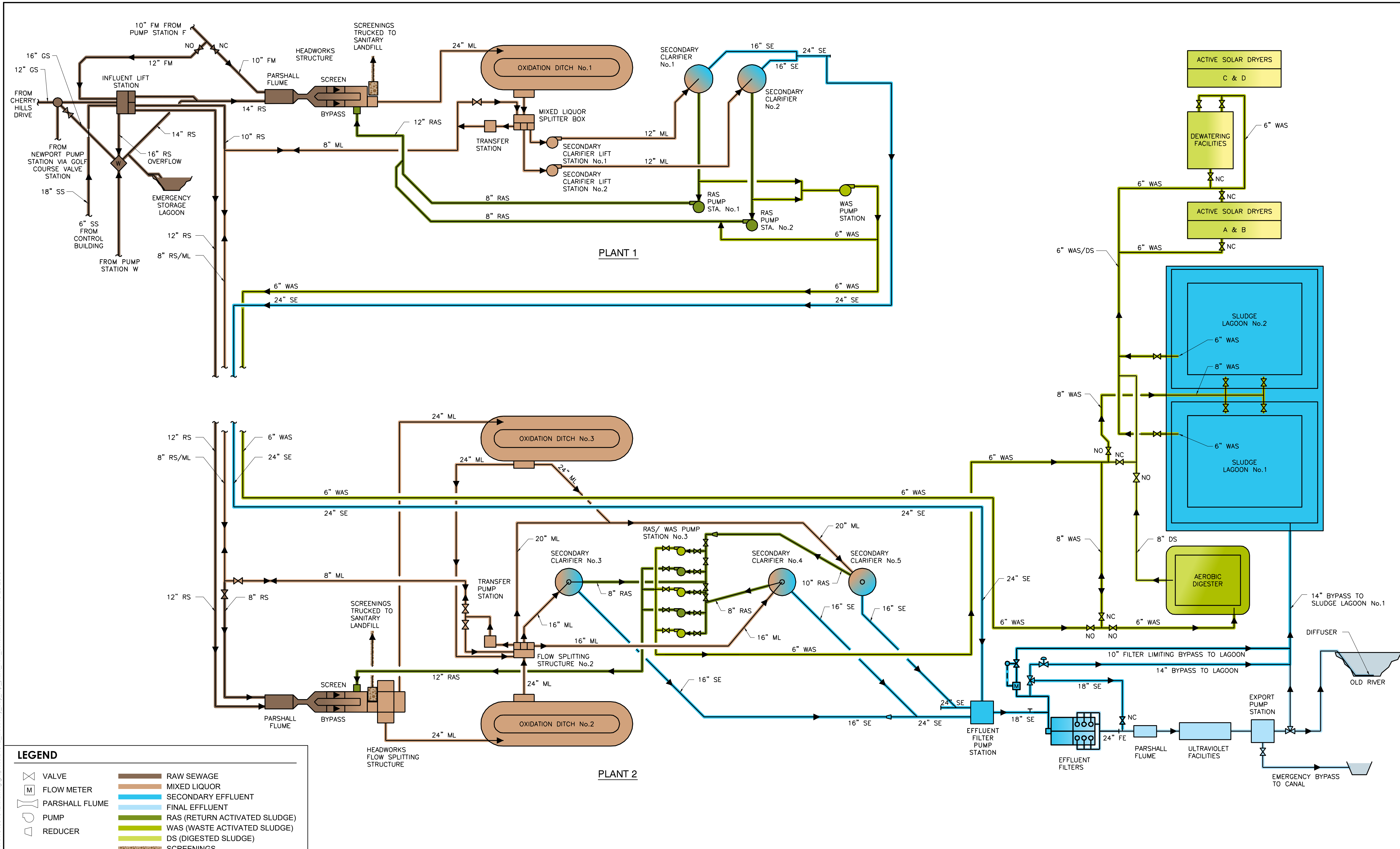
Town of Discovery Bay
Community Services District
Wastewater Master Plan Update

Figure 6-1
Existing Plant 1 Site Plan

Figure 6-2
Existing Plant 2 Site Plan



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Town of Discovery Bay
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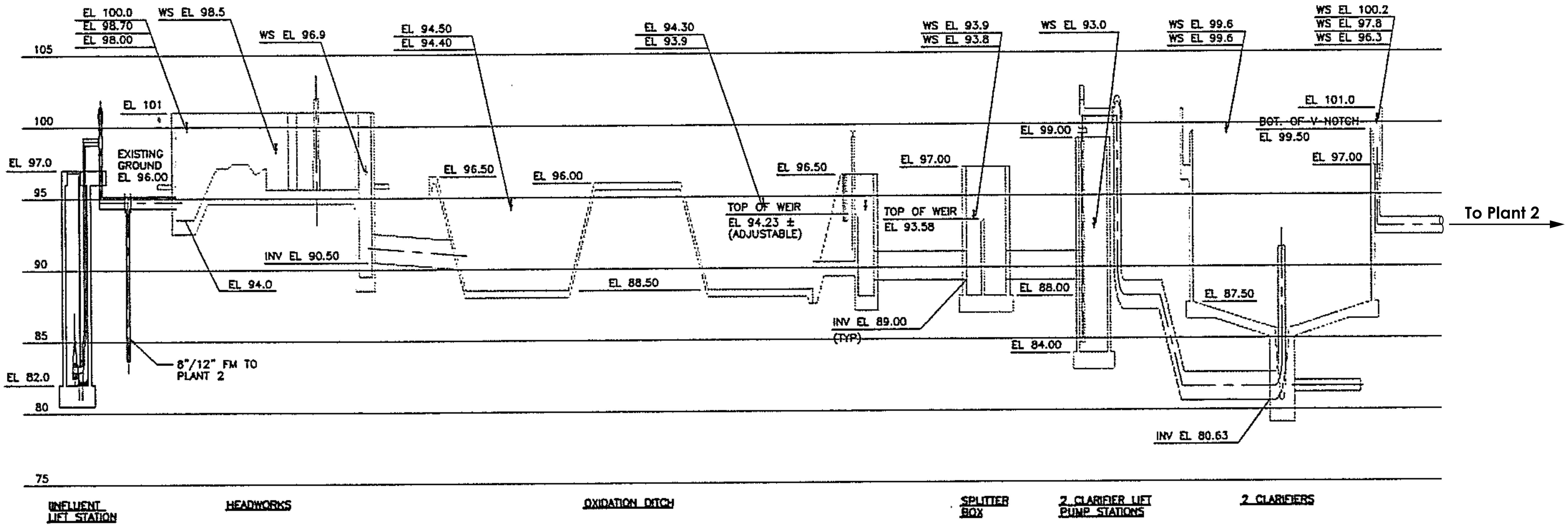
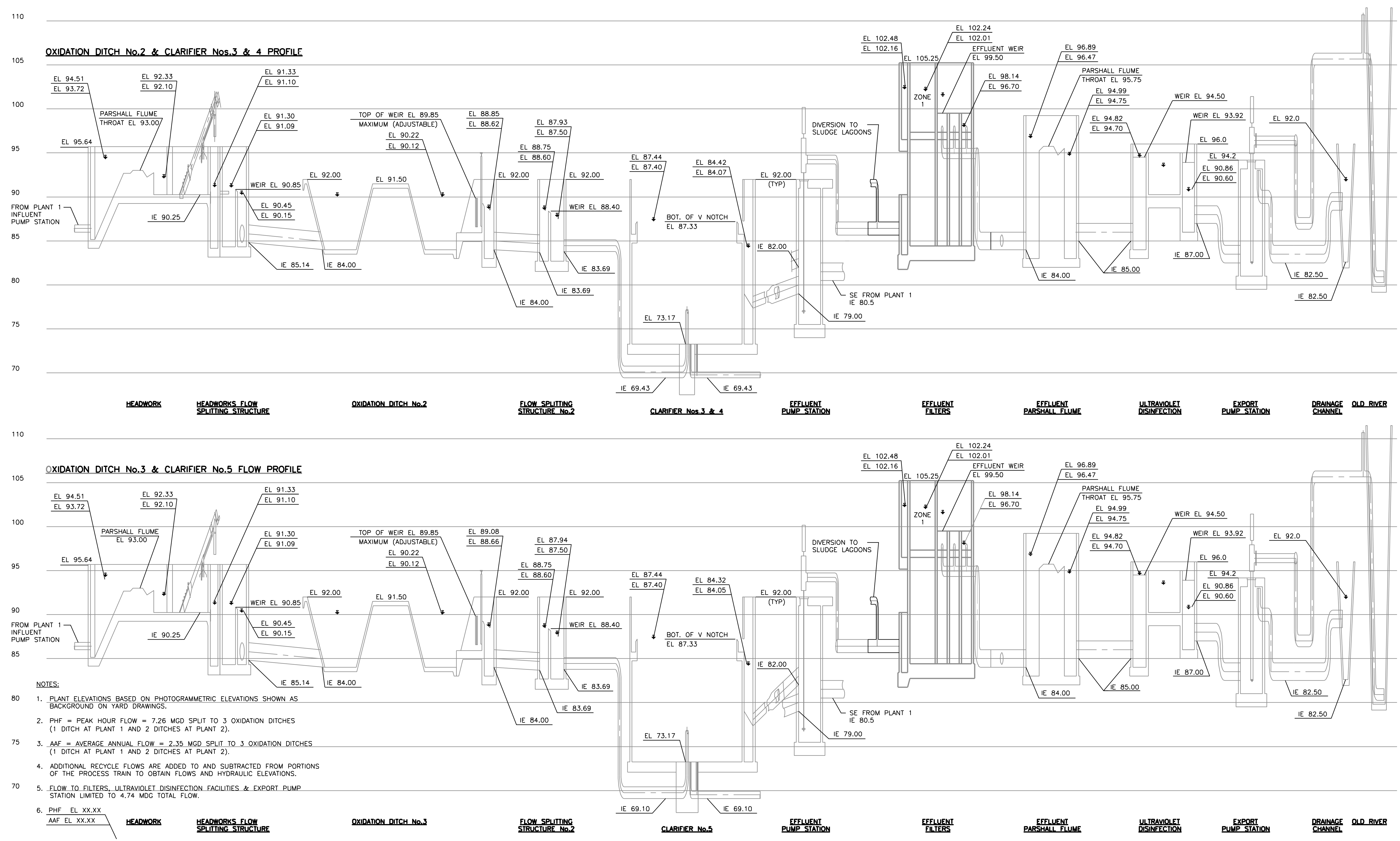
Wastewater Master Plan Update

Figure 6-3
Flow Schematics for Plant 1 and Plant 2



Note: See Figure 6-5 for Updated Plant 2 Hydraulic Profile

Figure 6-4
Hydraulic Profile for Plant 1



NOTES:

1. PLANT ELEVATIONS BASED ON PHOTOGRAMMETRIC ELEVATIONS SHOWN AS BACKGROUND ON YARD DRAWINGS.
2. PHF = PEAK HOUR FLOW = 7.26 MGD SPLIT TO 3 OXIDATION DITCHES (1 DITCH AT PLANT 1 AND 2 DITCHES AT PLANT 2).
3. AAF = AVERAGE ANNUAL FLOW = 2.35 MGD SPLIT TO 3 OXIDATION DITCHES (1 DITCH AT PLANT 1 AND 2 DITCHES AT PLANT 2).
4. ADDITIONAL RECYCLE FLOWS ARE ADDED TO AND SUBTRACTED FROM PORTIONS OF THE PROCESS TRAIN TO OBTAIN FLOWS AND HYDRAULIC ELEVATIONS.
5. FLOW TO FILTERS, ULTRAVIOLET DISINFECTION FACILITIES & EXPORT PUMP STATION LIMITED TO 4.74 MDG TOTAL FLOW.
6. PHF EL XX.XX
AAF EL XX.XX

7.0 PLANT HYDRAULIC CAPACITY ANALYSIS

To assess the ability of pumping and conveyance facilities to handle projected peak flows, a spreadsheet-based hydraulic model of the entire treatment plant (Plants 1 and 2) was used. All significant hydraulic features (structure elevations, pipe lengths and diameters, valves and fittings, weir configurations, etc.) of the liquid stream flow path from the Influent Pump Station through Plants 1 and 2 and through the Export Pump Station, pipeline and diffuser in Old River were included in the model. The hydraulic model is an updated version of the model that was first developed and used for the previous Master Plan dated February 2013 and was updated for Master Plan Amendment 2, dated April 2015.

The proposed additions of anoxic basins and related facilities at both plants were included in the updated hydraulic model used for this study. See Section 11 for a description of these facilities.

For the previous Master Plan efforts, the design peak hour flow was 7.11 Mgal/d (updated to 7.26 Mgal/d by Amendment 1). With the recent flow reductions that are discussed in detail in Section 5, the design peak hour flow for this Master Plan is only 4.89 Mgal/d. Several plant improvement projects were completed pursuant to the previous Master Plan (e.g., Influent Pump Station Improvements, Secondary Treatment Improvements, and Effluent Filtration Project) and were designed to accommodate flows higher than those currently projected for the buildout condition. Therefore, in these cases, hydraulic capacity is more than adequate for current projections. In some cases, improvements needed to accommodate the previous higher flow projections have not yet been completed and can be re-assessed under the new lower projections.

Two critical peak hour flow scenarios were evaluated for this study and are discussed below.

7.1 FUTURE PEAK HOUR FLOW SPLIT 1/3 TO PLANT 1 AND 2/3 TO PLANT 2

As developed in Section 11, under a future critical cold winter peak flow scenario, it will likely be necessary to operate both Plants 1 and 2. For the hydraulic analysis, the total influent flow analyzed was the future peak hour flow of 4.89 Mgal/d and the assumed flow split between the two plants was 1/3 to Plant 1 and 2/3 to Plant 2.

7.1.1 Influent Pump Station

The influent pump station was recently upgraded based on the previous Master Plan flow projections. Additionally, the pumps actually provided exceed the minimum design requirements. With four of the five existing pumps running, it is now estimated that as much as 5.6 Mgal/d could be pumped to Plant 2 at the same time as 2.9 Mgal/d is pumped to Plant 1 (8.5 Mgal/d total). These flows are much greater than required for the future peak hour flow of 4.89 Mgal/d split 1/3 to Plant 1 and 2/3 to Plant 2 (1.63 Mgal/d and 3.26 Mgal/d, respectively).

7.1.2 Plants 1 and 2 Headworks through Secondary Clarifiers

The facilities modeled were very similar to those considered in the previous Master Plan Amendment 2, with the primary differences being larger piping between the anoxic basins and oxidation ditches to accommodate higher internal mixed liquor recycle flows (see Section 11) and actual piping configurations built to suit Oxidation Ditch 3 and Clarifier 5.

The hydraulic analysis showed that the future design peak hour flow (plus associated plant recycle flows) can be accommodated without submerging the various process weirs. No hydraulic bottlenecks were noted.

7.1.3 Secondary Effluent Pump Station, Effluent Filters, and UV Disinfection

The Secondary Effluent Pump Station handles the combined secondary effluents from both Plant1 and Plant 2. The secondary effluent can be pumped into the effluent filtration system, or, after pumping, be diverted to the sludge lagoons. Therefore, the pump station must be able to handle the entire secondary effluent flow, whether or not any of the flow is diverted to the sludge lagoons. Since the future design peak hour influent flow is 4.89 Mgal/d, the Secondary Effluent Pump Station should be able to handle a flow at least 5 percent higher, or 5.13 Mgal/d, including recycle flows. The hydraulic model shows that this pump station has a reliable capacity of about 5.6 Mgal/d with two large and two small pumps running. Therefore, no improvements are needed.

The existing filters were designed for a maximum reliable capacity of 4.74 Mgal/d and can easily accommodate that flow. However, incremental flows greater than 4.2 Mgal/d are diverted to the sludge lagoons ahead of the filters (after pumping through the Secondary Effluent Pump Station), based on limitations of the downstream UV disinfection system and/or Export Pump Station.

The existing piping systems from the filters to the Export Pump Station are adequate to handle flows substantially higher than the 4.2 Mgal/d UV limitation and the 4.89 Mgal/d future peak hour flow. As part of the previous Master Plan, these piping systems were found to be adequate to handle the then projected peak hour flow of 7.11 Mgal/d (if the treatment facilities were upgraded to handle that flow).

7.1.4 Export Pump Station

Based on the hydraulic model, with four of the currently existing five pumps running, the Export Pump Station, working together with the export pipeline and effluent diffuser in Old River has a reliable capacity of about 4.2 Mgal/d. Although there is no apparent need to increase this capacity, options for modifying or replacing some or all of the export pumps could be evaluated if it is ever desired to do so.

The statement above regarding Export Pump Station capacity is based on the diffuser in Old River being in the original condition as designed and constructed. It is understood that the diffuser has recently been compromised and must be repaired to restore its original capacity.

7.2 FUTURE PEAK HOUR FLOW ROUTED TO PLANT 2 ONLY

Since it is desirable to run Plant 2 whenever possible and since it may be possible to run Plant 2 only during a future peak hour flow condition if other conditions are favorable (i.e., process temperature and SVI), the hydraulic model was used to assess hydraulic conditions with the entire future peak hour flow of 4.89 Mgal/d routed to Plant 2 (Plant 1 off-line). No hydraulic bottlenecks were revealed.

7.3 SUMMARY

The hydraulic features within Plant 1 and Plant 2 (including the proposed anoxic basin additions) upstream of the UV system are able to handle the future peak hour flow of 4.89 Mgal/d whether or not Plant 1 is in service. However, due to limitations of the UV system and/or Export Pump Station, flows higher than 4.2 Mgal/d are accommodated with excess flow diversions to the sludge lagoons ahead of the effluent filters.

8.0 COMPLIANCE WITH WASTE DISCHARGE REQUIREMENTS

The Town of Discovery Bay Wastewater Treatment Plant (WWTP) effluent is discharged to Old River at a location southeast of Plant 2. The discharge is currently regulated by Order R5-2014-0073-01 and National Pollutant Discharge Elimination System (NPDES) Permit No. CA0078590 adopted by the California Regional Water Quality Control Board, Central Valley Region.

An updated Report of Waste Discharge was submitted on January 23, 2019 to renew the NPDES permit. At the time of this report, only a draft of the new Order has been developed. This assessment includes evaluations based on compliance with the 2014 Order, requirements contained in the 2019 tentative Order, and potential future regulatory requirements.

8.1 COMPLIANCE ASSESSMENT

Review of monitoring reports submitted from January 2018 through August 2019 showed violations for electrical conductivity (EC), nitrate plus nitrite, turbidity, and mercury. Each of these constituents are discussed in this section. The reader is referred to the permit itself for complete coverage of all permit provisions.

In addition to effluent limitations, the permit contains receiving water limitations that govern the degree to which the plant effluent can impact the conditions in Older River. Included, for example, are limitations on bacteria, dissolved oxygen, pH, turbidity, etc. No receiving water limitation compliance issues are known to exist.

8.1.1 Electrical Conductivity

The electrical conductivity (EC) is a measure of the salinity associated with wastewater effluent and is primarily controlled by factors contributing salinity to the influent wastewater. In particular, the water quality of the potable water supply contributes significant salinity to the influent, and a large portion of the community softens water, which adds additional salinity.

The 2014 order included a limit on EC of 2,100 μmhos/cm as an annual average. This limit was exceeded in 2018 and will likely be exceeded in 2019. There is substantial dilution capacity in Old River to minimize any salinity impacts; however, limiting salinity discharges to reasonably obtainable levels are necessary to improve the overall quality of waters in the Delta. Future increases in effluent EC are possible as water conservation measures continue to be implemented in the community. The 2019 tentative Order addresses this occurrence by increasing the annual average EC limit to 2,400 μmhos/cm.

Source control is the most effective means for reducing the salinity of the wastewater. This may require implementation of District policies to limit the use of water softeners. However, since the effluent is in compliance with the updated salinity limit in the tentative Order, no wastewater treatment to reduce salinity is currently needed.

8.1.2 Nitrate Plus Nitrite

The permit includes strict limits on effluent ammonia-nitrogen (0.7 mg/L monthly average) and nitrite+nitrate-nitrogen (10 mg/L monthly average), which are scheduled to take effect on December 31, 2023. Currently, the District must meet interim limits for ammonia-nitrogen and nitrite+nitrate-nitrogen of 8.4 mg/L and 31 mg/L, respectively, both as daily maximums. The interim limit of 31 mg/L in the 2014 Order was exceeded twice, with a maximum effluent concentration of 34.7 mg/L. Understanding that these exceedances represent existing treatment process limitations, the interim limit has been increased to 39 mg/L in the tentative Order, which allows the effluent to remain in compliance. However, significant upgrades to the secondary treatment process are needed to comply with the ultimate nitrate plus nitrite limits, as defined further in Section 11.

The tentative Order has ammonia limits more stringent than those of the 2014 Order and does not contain a reopener clause to allow for studies to determine if mussels are present in the receiving water (in order to determine if the default 2013 Ammonia Criteria are applicable to the site). Further, available data indicate that dilution credits are available for both ammonia and nitrate plus nitrite, but they are not included as part of the final effluent limits. These items are being negotiated with the Board at this time. If the negotiations are unsuccessful, extensive treatment modifications are necessary to achieve the new limits, as described in Section 11.

8.1.3 Turbidity

The process limit for filtered effluent turbidity is 2 NTU as a daily average, measured upstream of ultraviolet (UV) disinfection. During a two-week period in January 2018 this limit was not achieved and has been attributed to the startup of the new effluent filtration system. After adjusting filtration operating parameters, the effluent has maintained compliance with this limit, other than a single exceedance in February 2018 (during process optimization).

The receiving water limitations for turbidity include an allowable range for turbidity increases from background concentrations. On one occasion, June 5, 2019, monitored turbidity at RSW-001 was 4.1 NTU higher than that at RSW-002, which was a greater difference than the allowable ranges included in the permit. This exceedance was attributed to tidal influences on Old River and background sampling difficulties. All other monthly monitoring events between 2018 and August 2019 showed a difference of approximately 0.5 NTU between these locations and within the ranges included in the Orders. Based on the operational limit of 2 NTU for the effluent, no further wastewater treatment to reduce turbidity is considered necessary.

8.1.4 Mercury

The permit contains waste load limits of 0.37 grams of methylmercury per year (in accordance with the Basin Plan's Delta Mercury Control Program). The WWTP has exceeded this limit in the past and has implemented a pollution prevention program (PPP) to achieve load reduction. A TSO has been issued for the interim limit of 24 grams of total mercury per year, which has consistently been achieved.

Monitoring of methylmercury has shown the PPP is working and methyl mercury loading has been below the final limitation, with an annual average methylmercury load of less than 0.098 grams in 2017 and 0.064 grams in 2018. Continued implementation of the PPP and effluent monitoring will provide additional information on whether there is a need for treatment process improvements; however, at this time, no improvements are considered necessary for compliance with the final methylmercury effluent limit.

8.2 POSSIBLE FUTURE PERMIT REQUIREMENTS

The general trend in permitting is to become increasingly stringent over the years. With the exception of nitrogen constituents, which are being addressed, the treatment processes are anticipated to remain compliant with near-term water quality requirements. Long-term compliance is dependent on future permit requirements, which may include more stringent or added provisions for salinity, pesticides/herbicides/fungicides, and contaminants of emerging concern (CECs, including pharmaceuticals and endocrine disrupting compounds, such as hormones).

There have been significant increases in salinity restrictions in the Central Valley, upstream of the Delta, to minimize groundwater degradation and salt accumulation. These are not necessarily applicable to the current WWTP discharge (where salinity is significantly diluted and carried out to the ocean), but additional salinity load from the Central Valley has the potential to increase water salinity in the Delta and reduce dilution capacity in Old River. Future water conservation measures (water use restrictions) and recycling requirements may result in increased influent salinity, requiring additional treatment or source control. The Town should continue to participate in Central Valley salinity and water use planning programs to ensure their water quality needs are addressed. To minimize unreasonable degradation, the Town of Discovery Bay is required to maintain its Salinity Evaluation and Minimization Program.

The impacts of pesticides, herbicides, and fungicides on water quality and aquatic habitats will continue to drive implementation of control programs and development of discharge limits. This has resulted in the inclusion of chlorpyrifos and diazinon limits in the new (tentative) Order, even when there is no reasonable potential for impacts. Discharge limits for pyrethroid pesticides were adopted in the San Joaquin and Sacramento River Basin Plans and became effective on April 22, 2019. Although the Delta is not listed as impaired by pyrethroids, these limits are anticipated to be addressed in the next NPDES permit renewal cycle. The Delta is listed as impaired by Group A pesticides (including organochlorine pesticides), but a TMDL has not been developed and numerical limits are not included in the permit. Future developments in these types of chemicals will require additional analysis of effluent chemistry and potentially require additional treatment to comply.

CECs at variable concentrations have been detected in treated effluent from conventional wastewater treatment plants. However, numeric requirements for removal of CECs appear to be unlikely for the foreseeable future. If CEC removal becomes an issue with the current surface water discharge, use of advanced oxidation processes, such as ozonation, combined with biological activated carbon filtration can be considered.

9.0 INFLUENT PUMP STATION

The District's sewage collection system routes all flow to the Influent Pump Station, which is located on the Plant 1 site and is used to pump influent flows to both Plants 1 and 2.

The previous Master Plan, dated February 2013, included a detailed analysis of the Influent Pump Station and recommendations for replacing the pumps and related improvements to handle the future peak hour flow, which was then projected to be 7.1 Mgal/d (increased to 7.26 Mgal/d with Amendment 1). Pursuant to the Master Plan recommendations, the District completed the Influent Pump Station and Pump Station W Improvements Project in 2014. The actual pumps provided for that project exceeded the minimum specified requirements.

For the current Master Plan Update, the revised future peak hour design flow is 4.89 Mgal/d, based on actual reductions in wastewater flows experienced after the previous Master Plan was completed (see Section 5). The hydraulic analysis developed in Section 7 showed that the Influent Pump Station is now capable of handling flows substantially higher than 4.89 Mgal/d, whether pumping to Plant 2 only or to a combination of Plant 1 and Plant 2. No future improvements to this pump station are currently anticipated.

10.0 HEADWORKS

There are separate headworks systems at Plant 1 and at Plant 2. Each headworks includes a 12-inch Parshall flume for measuring the flow, a mechanical screening unit and a manual bypass bar screen unit. The channels of both headworks facilities are covered and vented through soil odor scrubber systems. At Plant 2, there is an automated sampler that is used to characterize the influent wastewater for both plants.

The screening system at each plant has a maximum design capacity of 6.2 Mgal/d, which exceeds the future peak hour design flow of 4.89 Mgal/d (see Section 5), whether this flow is pumped to Plant 2 only or is split between Plant 1 and Plant 2. Therefore, no modifications to increase the capacities of the screens are needed.

The previous Master Plan dated February 2013 recommended improvements to the headworks at Plant 2 to correct the problem of non-representative sampling caused by rag accumulations on the automatic sampler intake, which is located at the discharge of the Parshall flume and upstream of the screen. These improvements have not yet been completed and non-representative sampling remains to be an issue as documented in Section 5. At the time of writing this document, a plan to make minor modifications to the screen channel and to relocate the sampler intake downstream of the screen are proceeding. It is presumed that these improvements will be successful and that no further improvements to the Plant 2 headworks will be needed.

The Plant 1 headworks are in need of some repairs and rehabilitation, which are considered in Section 20 of this document.

11.0 SECONDARY TREATMENT FACILITIES

In this section, the existing secondary treatment system is described and methods to upgrade the system to meet new discharge requirements for ammonia-nitrogen and nitrate+nitrite-nitrogen at the future buildout flows and loads are evaluated. A recommended plan of improvements is developed.

11.1 EXISTING FACILITIES

The existing secondary treatment facilities are divided between Plant 1 and Plant 2 and consist of oxidation ditches, clarifiers and associated facilities. Plant 1 includes one oxidation ditch and two clarifiers, while Plant 2 includes two oxidation ditches and three clarifiers. At the present time, only the facilities at Plant 2 are being used. Facilities at Plant 1 remain available for use if units at Plant 2 need to be taken out of service for maintenance or repair. Additionally, Plant 1 can be restored to normal use if needed to serve future flows and loads, a topic that is evaluated in this section.

A flow diagram and key design criteria for these facilities are presented in Section 6. For ease of reference in this section, sizing and capacity data for the various components of the secondary treatment systems in Plant 1 and Plant 2 are listed in Tables 11-1 and 11-2, respectively.

The secondary treatment facilities at Plant 1 and Plant 2 comprise two separate activated sludge systems. The oxidation ditches are the reactor basins wherein mixed cultures of microorganisms are used to remove organic material and ammonia contained in the influent wastewater and produced within the process. Currently, no specific features are included for removal of nitrite or nitrate-nitrogen by denitrification, although limited removals can occur coincidentally.

The suspension of microorganisms and other wastewater solids in each oxidation ditch is referred to as mixed liquor. The microorganisms require oxygen, which is provided by four brush rotors in each ditch. The brush rotors also provide the motive force needed to keep the mixed liquor circulating around each ditch at a velocity that is adequate to keep the microorganisms and other solids in suspension.

The mixed liquor from the oxidation ditches flows to splitter boxes that are used to divide the flow equally to the secondary clarifiers within each plant. Within the secondary clarifiers, the microorganisms and other wastewater solids are settled to the bottom, while the clarified secondary effluent flows over weirs and into a collection channel arranged around the periphery of the clarifier before exiting the clarifier structure. The settled solids are collected by a rotating mechanism above the floor of the clarifier and are, for the most part, pumped back to the oxidation ditches using return activated sludge (RAS) pumps. A portion of the settled solids are wasted from the system and are pumped by waste activated sludge (WAS) pumps to the solids handling facilities.

In Plant 1, the clarifiers are at a higher elevation than the upstream splitter box; therefore, a clarifier lift pump station is used ahead of each clarifier.

Table 11-1 Secondary Treatment Facilities Component Sizing and Capacity Data – Plant 1

Component	Parameter	Value
Oxidation Ditch 1	Volume, Mgal	1.0
Oxidation Ditch 1	Number of Brush Rotors	4
Oxidation Ditch 1	Brush Rotor Horsepower, ea	30
Oxidation Ditch 1	Capacity per Brush Rotor, lb O_2 / d (Standard)	1,480 to 2,150 (a)
Clarifier Lift Pump Station 1 (Serves Clarifier 1)	No. Pumps	1 + 1 Standby
Clarifier Lift Pump Station 1 (Serves Clarifier 1)	Capacity per Pump, Mgal/d	1.6
Clarifier Lift Pump Station 2 (Serves Clarifier 2)	No. Pumps	1 + 1 Standby
Clarifier Lift Pump Station 2 (Serves Clarifier 2)	Capacity per Pump, Mgal/d	1.6
Clarifier 1	Diameter, ft	50
Clarifier 1	Depth, ft	10
Clarifier 2	Diameter, ft	50
Clarifier 2	Depth, ft	12
RAS Pump Station 1 (Serves Clarifier 1)	No. Pumps	1 + 1 Standby
RAS Pump Station 1 (Serves Clarifier 1)	Capacity per Pump, Mgal/d	0.80
RAS Pump Station 2 (Serves Clarifier 2)	No. Pumps	1 + 1 Standby
RAS Pump Station 2 (Serves Clarifier 2)	Capacity per Pump, Mgal/d	0.80
WAS Pump Station	No. Pumps	1 + 1 Standby
WAS Pump Station	Capacity per Pump, Mgal/d	0.58

(a) See text regarding apparent capacities of inside and outside rotors.

Table-11-2 Secondary Treatment Facilities Component Sizing and Capacity Data – Plant 2

Component	Parameter	Value
Oxidation Ditch 2 and 3	Volume, Each Ditch, Mgal	1.0
Oxidation Ditch 2 and 3	Number of Brush Rotors per Ditch	4
Oxidation Ditch 2 and 3	Brush Rotor Horsepower, Each Rotor	30
Oxidation Ditch 2 and 3	Capacity per Brush Rotor, lb O_2 / d (Standard)	1,480 to 2,150 (a)
Clarifier 3 - 5	Diameter, Each, ft	50
Clarifier 3 - 5	Depth, ft	14
RAS Pumps (Serving Clarifiers 3 - 5)	No. Pumps	3 + 1 Standby
RAS Pumps (Serving Clarifiers 3 -5)	Capacity per Pump, Mgal/d	1.1
WAS Pumps	No. Pumps	1 (b)
WAS Pumps	Capacity per Pump, Mgal/d	0.58

(a) See text regarding apparent capacities of inside and outside rotors.
(b) Standby RAS pump can also be used for WAS.

As noted in Tables 11-1 and 11-2, the clarifiers at Plant 2 are deeper than the clarifiers at Plant 1. Additionally, the clarifiers at Plant 2 have density baffles to mitigate the impacts of the sludge blanket rising up at the wall. This rise is caused by the introduction of the mixed liquor at the center of the clarifier. Since the mixed liquor has a higher bulk density than the clarified effluent in most of the clarifier volume, the mixed liquor tends to fall to the floor at the center and create a current that sweeps radially outward at the clarifier bottom and then up the wall. The density baffles in the Plant 2 clarifiers help to keep any rising solids away from the effluent weirs. Because of the clarifier depth and the density baffles, Plant 2 clarifiers are believed to provide a higher reliability of good performance, as compared to the Plant 1 clarifiers.

11.1.1 Rotor Capacity

Based on the manufacturer's submittal during construction, the rotors in Oxidation Ditch 3 (and presumed the same for Oxidation Ditches 1 and 2) should be operated at a maximum immersion of 13.25 inches, unless a higher immersion is approved by the factory. At this immersion, performance charts provided by the manufacturer indicate a power draw at the rotor shaft of 27.2 hp and a standard oxygen transfer rate (SOTR) of 2,133 lb/d. Due to losses in the belt and gear drives, the power draw at the motor could be around 6 to 11 percent higher than at the rotor shaft, or about 28.8 to 30.2 hp. Therefore, at 13.25 inches immersion, the 30 hp motors should be nearly fully loaded. At 30 hp full load, the motors are rated to draw 35.1 amps. Although the motors have a 1.25 service factor that could allow operation at higher immersion and power draw, it is typically desirable to avoid encroachment on the service factor, which should be considered as a safety margin.

Based on recent information provided by the Chief Engineer of Lakeside (the rotor manufacturer), the rotors could be operated at an immersion up to 13.9 inches, which would require 28.8 hp at the rotor shaft (perhaps around 30.5 to 32.0 hp at the motor shaft, which is about 2% to 7% above motor rating, but well within the 1.25 service factor). In this case the rotor oxygen delivery capacity would be 2,177 lb/d. If an SOTR of 2,133 lb/d is presumed to correspond to a current draw of 35.1 amps and to 27.2 hp at the rotor shaft and 30.0 hp at the motor shaft, then, based on rotor performance charts, 2,177 lb/d would be estimated to correspond to about 28.2 hp at the rotor shaft, 31.1 hp at the motor shaft, and a current draw of 36.4 amps.

Based on startup testing of the rotors, the District Engineer reported a current of 37 amps at the inside rotors (rotors closest to the center island in the ditch) with an immersion of approximately 13 inches (immersion estimated from water depth at the rotors when not running). Due to minor discrepancies in ditch floor elevation and rotor elevation as compared to the design values, it is possible that the actual immersion may have been higher than 13 inches. However, the current draw of 37 amps would correspond to a theoretical immersion of about 14 inches.

Based on the above, it is reasonable to say that rotor capacity should be in the range of 2,133 to 2,177 lb/d SOTR. Therefore, a value of 2150 lb/d is a reasonable assumption for this study.

In the same startup field testing mentioned above, the outside rotors, when operated at the same time as the inside rotors, had a current draw of only 24 amps. Since power delivery should be proportional to the current, the power draw at the outside rotors is estimated to be only 24/37 = 65 percent that of the inside rotors. Based on Lakeside rotor performance Charts, the corresponding SOTR of the outside rotor would be about 69 percent that of the inside rotor (SOTR is not directly proportional to power input). Thus, if an SOTR capacity of 2,150 lb/d is assumed for the inside rotors, then the outside rotors running at the same time would be estimated to have an SOTR of about 1480 lb/d. In that case, the average SOTR for all four rotors running at the same time would be 1,815 lb/d.

It is believed that the different performances of the inside and outside rotors are due to different hydrodynamic conditions (particularly ditch water velocities approaching the rotors). It is not known how the hydrodynamic conditions and the impacts on rotor current draw, power input, and SOTR would vary depending on which and how many inside and outside rotors run at the same time. Furthermore, accurate determinations of SOTR for the various conditions would require clean water oxygen transfer testing in at least one of the ditches. These types of analyses are beyond the scope of this Master Plan, but should be considered in the context of a preliminary design study. For this Master Plan, it is considered adequate to estimate the following SOTRs:

- All four rotors running: 2 x 2,150 + 2 x 1,480 = 7,260 lb/d
- Two inside and one outside rotor running: 2 x 2,150 + 1,480 = 5,780 lb/d
- Two outside and one inside rotor running: 2,150 + 2 x 1,480 = 5,110 lb/d

11.2 SECONDARY TREATMENT OBJECTIVES AND BACKGROUND

The existing secondary treatment system was designed to produce a secondary effluent with relatively low BOD and TSS concentrations (10 to 30 mg/L), with only minor coincidental removals of ammonia and nitrate-nitrogen. However, the District's National Pollutant Discharge Elimination System (NPDES) permit that was adopted on June 6, 2014 and the current draft NPDES permit renewal include strict limits on effluent ammonia-nitrogen (0.7 mg/L monthly average) and nitrite+nitrate-nitrogen (10 mg/L monthly average), which are scheduled to take effect on December 31, 2023. Currently, the District must meet interim limits for ammonia-nitrogen and nitrite+nitrate-nitrogen of 8.4 mg/L and 31 mg/L, respectively, both as daily maximums. The main purpose of this section is to determine how to meet the future permit limits most cost-effectively.

In the previous Wastewater Treatment Plant Master Plan, Amendment 2, dated July 2015, three key alternatives for the secondary treatment system were evaluated. In all cases, ammonia removal was to be accomplished in the oxidation ditches. The three alternatives were based around the methods to be used to remove nitrite+nitrate-nitrogen, as follows:

1. Simultaneous Nitrification and Denitrification (SND).
2. Anoxic Basins
3. Denitrification Filters

Simultaneous nitrification and denitrification was not recommended for two key reasons:

1. The cyclically low dissolved oxygen (DO) concentrations needed to meet the nitrite+nitrate-nitrogen limit would prevent reliable compliance with the ammonia-nitrogen limit, which would require consistently high DO.
2. Operation at low DO concentrations frequently leads to sludge bulking (failure of solids to settle well in the secondary clarifiers) and solids carryover from the secondary clarifiers.

Shortly before the start of this current Master Plan evaluation, there was some hope that the California Regional Water Quality Control Board, Central Valley Region, was going to review and relax the ammonia-nitrogen limit, which could have potentially made the SND alternative more attractive. However, it has since been determined that no significant relaxation of the ammonia-nitrogen limit is likely. Therefore, an SND alternative would have to be accompanied by additional treatment facilities for ammonia removal. This could be in the form of new aerobic suspended growth reactors after the oxidation ditches and before the clarifiers or new attached growth reactors (e.g., moving bed bioreactors) after the clarifiers and before the filters. However, even with additional ammonia removal facilities, the concern with SND sludge bulking would still exist. Also, SND design and performance is not precise and cannot be adequately validated without full-scale performance testing over more than a year, which would require significant modifications to the operation and control of the mechanical aeration systems in the oxidation ditches, with no guaranty of success. Based on all these factors, which apply regardless of the recent changes in flows and loads, the SND alternative is again not recommended.

The denitrification filter alternative was pilot tested at the Discovery Bay Wastewater Treatment Plant and was evaluated in detail in the previously mentioned Amendment 2 and was determined to be inferior to the anoxic basin alternative. Therefore, the District proceeded with construction of filters that are not structurally deep enough and do not have the chemical feed systems needed for denitrification.

Based on the above, the recommended method for denitrification is the addition of anoxic basins ahead of the existing oxidation ditches, which is consistent with the previous Master Plan, Amendment 2. However, because of recent changes in wastewater flows and loads, which are documented in Section 5, and because of reduced wastewater temperatures (discussed in the next subsection), it is necessary to re-evaluate the anoxic basin alternative and the capacities of Plant 1 and Plant 2 with these improvements.

11.3 WASTEWATER TEMPERATURE

Wastewater temperature has a large impact on microbiological activity and, therefore, on the rate of treatment in an activated sludge system. In particular, the slow growth rate of ammonia oxidizing bacteria (AOB) with cold temperatures in the winter months is the main limiter of oxidation ditch capacity.

Wastewater influent temperatures are measured weekly and effluent temperatures are measured twice per week at the Discovery Bay Wastewater Treatment Plant. Temperature data for the years of 2017, 2018 and a portion of 2019 are shown in Figure 11-1. Effluent temperatures are probably most indicative of temperatures in the activated sludge process. As indicated in the figure, however, influent and effluent temperatures were generally similar over the data period shown. For process design, the lowest seasonal temperatures that are sustained for a couple of weeks are most important (neglecting outlier data). Accordingly, from the data shown in Figure 11-1, a minimum process design temperature of 13°C is recommended.

In Figure 11-2, similar wastewater temperature data from the years 2004-2007, which were used as the basis of the previous Master Plan are shown. By comparing Figure 11-1 to Figure 11-2, it can be seen that minimum winter influent temperatures have decreased by about 7°C and effluent temperatures have decreased by about 2°C. The lower wastewater temperatures could be the result of lower flows and higher residence times in the sewer system and changed habits with regards to the use of hot water (e.g., shorter showers and more efficient use of hot water in appliances resulting from water and energy conservation). The lesser incremental change in effluent temperatures as compared to influent temperatures is likely due to the fact that the wastewater in the treatment basins was exposed to similar ambient temperatures in the earlier and later periods of record.

If all else remains equal, the 2°C decrease in effluent and process design temperature has the net effect of decreasing the capacity of the oxidation ditches by about 13 percent due to a similar decrease in AOB growth rate.

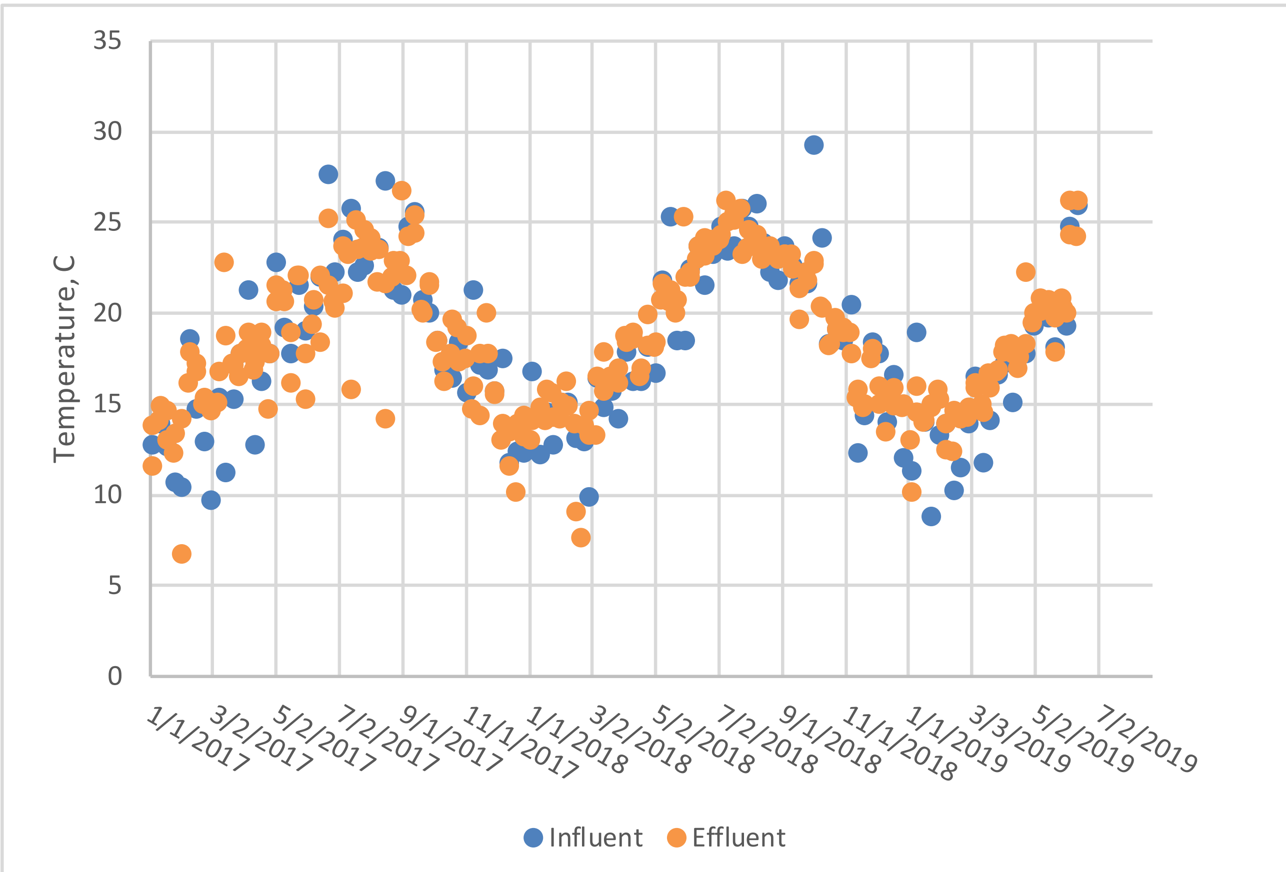


Figure 11-1 Wastewater Temperatures 2017-2019

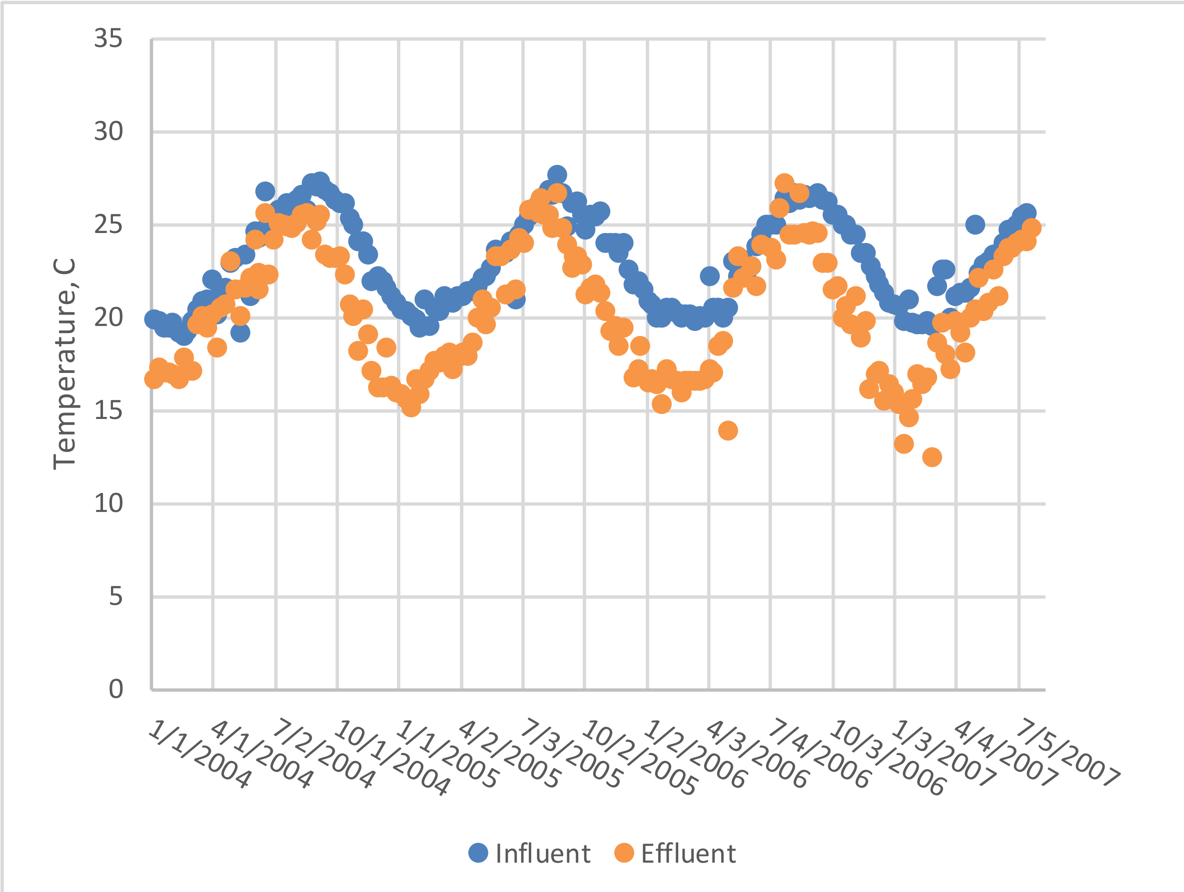


Figure 11-2 Wastewater Temperatures 2004-2007

11.4 RECYCLE FLOWS AND LOADS

In-plant recycle flows and loads can be significant and must be considered in the design and evaluation of the secondary treatment system. Recycle flows are created and handled within Plant 2 (no recycle flows within Plant 1) and include the following significant components:

- Filter backwash water
- Aerobic digester decant
- Sludge dewatering filtrate and spent belt press cleaning water

The filter backwash water is routed to the Decant Pump Station and is pumped to either Oxidation Ditch 2 or Oxidation Ditch 3. The aerobic digester decant and the sludge dewatering return flows are discharged into the sludge lagoons, which are occasionally decanted into the Decant Pump Station for pumping to either Oxidation Ditch 2 or Oxidation Ditch 3, together with the filter backwash water.

The return flows can be highly variable, depending on the number of belt presses in operation and whether decant is being returned from the sludge lagoons. The characteristics of the recycle flows from the sludge lagoons depend on algae growth and other conditions that can vary throughout the year. Based on solids balance calculations, it was determined that it is reasonably conservative to allow for total recycle flows to be 10 percent of the plant influent flow and for recycle loads to be 5 percent of the plant influent loads (BOD, TSS, and TKN). Therefore, these values were incorporated into the secondary process evaluations. All recycle flows and loads were assumed to be discharged directly into the oxidation ditches in Plant 2.

11.5 SECONDARY PROCESS ANALYSIS METHODS AND CRITERIA

Process design calculations were completed using both a spreadsheet-based model and using the BioWin process simulator. Each of these methods are discussed below, including key input criteria. In all cases, a critical design winter temperature of 13°C was used. Additionally, the critical design condition was based on average day maximum monthly loads occurring at the same time as average annual flows. This represents a reasonable worst case leading to high influent constituent concentrations (BOD and TSS at 358 mg/L and TKN at 72 mg/L; see Table 5-12 in Section 5).

The focus of the process analysis discussed below is on Plant 2. It is considered particularly important to maximize the capacity and use of Plant 2 and to use Plant 1 when necessary. All of the improvements and capacity determinations developed for Plant 2 are adapted to Plant 1 later in this Section.

Because the sizing of anoxic basins will impact the capacity and performance of the oxidation ditches, it is necessary to consider the anoxic basins and oxidation ditches in a combined analysis. In particular, increased sizing of the anoxic basins will generally improve denitrification performance and compliance with the effluent nitrate+nitrite-N permit limit of 10 mg/L. However, increasing anoxic volumes will result in a lower net growth rate of the microorganisms responsible for ammonia removal (nitrification). The objective of this analysis is to find the most efficient and cost-effective means of accomplishing both nitrification and denitrification as needed to meet effluent limitations for ammonia-N and nitrate+nitrite-N at the same time.

One of the most important design parameters used in the spreadsheet model and in BioWin simulations is the aerobic mean cell residence time (MCRT) needed to attain reliable nitrification. Therefore, this topic is considered first below.

11.5.1 Preliminary Evaluation of Mean Cell Residence Time Required for Reliable Nitrification

Nitrification, which is the biological conversion of ammonia to nitrite and nitrate, is the first step in nitrogen removal and is the rate-limiting step under low temperature conditions. Nitrification occurs under aerobic conditions (in the presence of dissolved oxygen), while the subsequent conversion of nitrate to nitrogen gas (denitrification) occurs under anoxic conditions (oxygen absent, but nitrate present). For Discovery Bay, nitrification will occur in the oxidation ditches and denitrification will occur in the anoxic basins.

Since the bacteria that accomplish nitrification grow only under aerobic conditions, it is necessary that the aerobic MCRT (total MCRT multiplied by the fraction of the total reactor basin volume that is aerobic; i.e., oxidation ditch volume divided by the total volume of the oxidation ditch and associated anoxic basin) be long enough so that the net growth rate is faster than the rate at which these bacteria are removed in waste activated sludge and so that an adequate population of nitrifiers can be sustained to attain the desired effluent ammonia-nitrogen concentration (ammonia-N<0.7 mg/L). The net growth rate is the rate of growth minus the rate of decay, noting that growth occurs only under aerobic conditions (in the oxidation ditches), but decay occurs under both aerobic and anoxic conditions (in the oxidation ditches and in the anoxic basins). Therefore, in the anoxic basins, the population of active nitrifiers will decrease. Theoretical aerobic MCRTs (with no safety factor) required to attain an effluent ammonia-nitrogen concentration of 0.7 mg/L are shown in Figure 11-3 as a function of the fraction of the total reactor basin volume that is under anoxic conditions and for various temperatures. For this study, anoxic basin volumes in the range of 0.2 to 0.4 Mgal at each oxidation ditch are considered. This range of anoxic volumes corresponds to anoxic volume fractions (anoxic volume divided by total reactor volume) of 0.17 to 0.29. For this range of anoxic volumes, and at the process design temperature of 13°C, the required aerobic MCRT ranges from approximately 10.7 days to 12.4 days (not including a safety factor). A modest safety factor of 1.25 would result in aerobic MCRTs from 13.4 to 15.5 days.

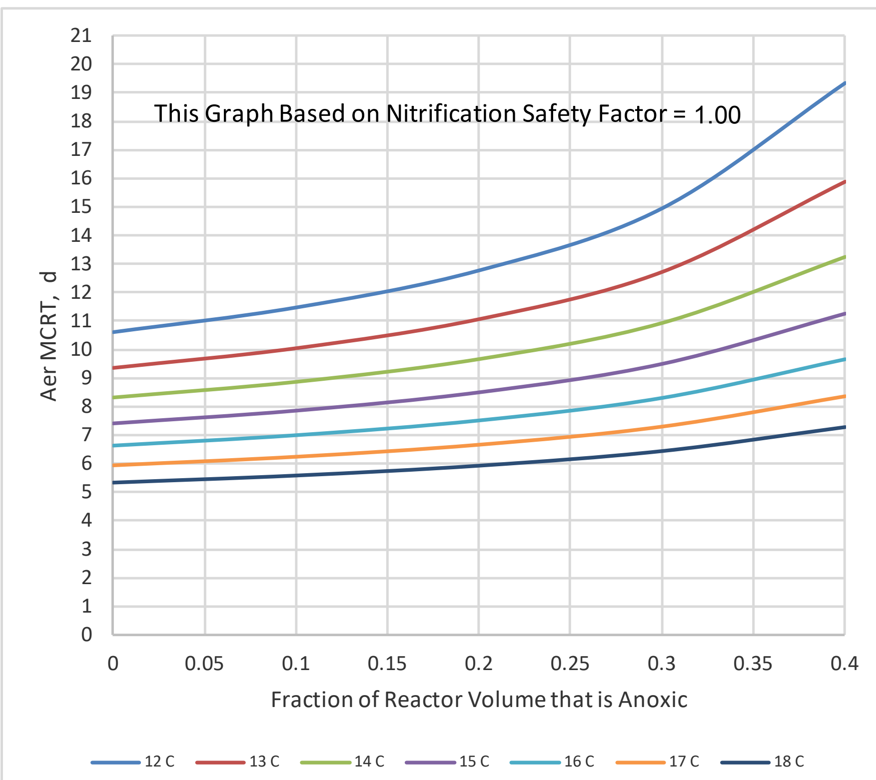


Figure 11-3 Aerobic MCRT for Nitrification vs Anoxic Volume Fraction and Temperature

The aerobic MCRTs shown in Figure 11-3 and discussed above are based on theoretical calculations that assume that the oxidation ditch is a completely mixed reactor in which the effluent ammonia-N concentration is 0.7 mg/L and the dissolved oxygen concentration is 2.0 mg/L everywhere throughout the volume. In reality, influent ammonia-N is introduced at one location in the oxidation ditch and is at that location immediately diluted by the flow of mixed liquor circulating around the ditch. As the mixed liquor continues its travel from the influent location to the effluent location in the ditch, the ammonia concentration is reduced. This means that the ammonia concentration at the influent location will be higher than the ammonia concentration at the effluent location. Since the rate of ammonia removal is higher with higher concentrations of ammonia, the average ammonia removal rate within the oxidation ditch will be higher than would occur at a constant ammonia-N concentration of 0.7 mg/L and the effluent ammonia-N will be lower than 0.7 mg/L. Similarly, dissolved oxygen concentrations are highest at the rotors and decrease downstream from the rotors, which also impacts the rate of ammonia removal. BioWin simulations are required to evaluate these impacts, as discussed later in this section.

11.5.2 Spreadsheet Model Description and Key Criteria

The capacity of the existing secondary treatment system at Plant 2 was assessed using a spreadsheet model to simultaneously solve biological process design equations for the oxidation ditches, secondary clarifiers and RAS pumping systems. In essence, the spreadsheet model is used to determine if the oxidation ditches are large enough to hold the biomass necessary for treatment and if the clarifiers are large enough to settle the mixed liquor solids flowing from the oxidation ditches, considering the settling characteristics of those solids. Although the spreadsheet model includes features for analysis of nitrification and denitrification, BioWin simulations are necessary to accurately evaluate performance with respect to ammonia-N and nitrate+nitrite-N concentrations.

Key parameter values used in the spreadsheet model, unless noted otherwise, are listed below:

- Average influent BOD = 275 mg/L
- Average influent TSS = 275 mg/L
- Average influent TKN = 55 mg/L
- Peak month BOD and TKN load = 1.3 x average annual BOD and TKN load
- Peak day BOD and TKN load = 2.0 x average annual BOD and TKN load
- Peak hour BOD and TKN load = 3.0 x average annual BOD and TKN load
- Peak day flow = 2.1 x average annual flow
- Peak hour flow = 3.0 x average annual flow
- Sludge yield based on Water Environment Federation Manual of Practice 8 (MOP8, Fourth Edition), Figure 11.7b, with mixed liquor solids 80% volatile
- Sludge Volume Index (SVI) = 175 mL/g

- Peak month recycle flow = 10% of influent flow
- Peak month recycle loads = 5% of influent loads

As noted above, sludge yields were based on values shown in Figure 11.7b of MOP8. For example, with a hypothetical 20-day total mean cell residence time (MCRT) and a temperature of 13°C, the sludge yield would be estimated to be about 0.93 pounds of total suspended solids (TSS) per pound of BOD removed. The MOP8 sludge yields are known to be conservatively high for most plants. Typical values would perhaps be around 80% of the MOP8 values. However, the MOP8 values are based on COD:BOD ratios of 1.9 to 2.2, while the ratio for Discovery Bay is estimated at 2.5 (see Section 5), and this would imply higher than typical sludge yields. Unfortunately, long-term reliable plant influent load data that would be needed to verify actual plant sludge yields are not available. Based on the uncertainty of actual sludge yields, the capacity assessments presented herein are approximate, but believed to be reasonably conservative.

The SVI of 175 mL/g assumed for this analysis is believed to be reasonably conservative (high) for the proposed system with an anoxic basin ahead of an aerobic basin when the aerobic basin is operated always with a relatively high dissolved oxygen concentration (2 mg/L) to assure reliable nitrification. Use of low dissolved oxygen concentrations are detrimental to nitrification and can cause sludge bulking (higher SVI).

11.5.3 Basis of BioWin Simulations

In addition to wastewater characteristics described for use in the spreadsheet model, BioWin requires more detailed characterization of the influent wastewater in terms of COD fractions. Key parameter values used in this study are summarized in Table 11-3. In addition to COD fractions, an SND switching function parameter is identified in Table 11-3 and discussed below because of its importance in the denitrification evaluations. BioWin default values were used for parameters not specifically mentioned below.

Because of the high recirculation rates around an oxidation ditch, the ditch is almost like a completely mixed reactor and is frequently modeled as such with adequate accuracy. However, as mentioned previously, some variations in process conditions do occur as the mixed liquor circulates around the oxidation ditch from the influent location to the effluent location. Most importantly for this study, and as previously mentioned, dissolved oxygen and ammonia concentrations vary (dissolved oxygen varies to a much greater extent than ammonia).

To provide a more precise evaluation of nitrification and denitrification performance, the oxidation ditch was modeled as six completely mixed reactor basins in series with a high recirculation flow rate representing the velocity of mixed liquor circulating around the oxidation ditch and with oxygen supply (rotors) only in the first and fourth reactor compartments. At a velocity of 1.0 ft/s, the mixed liquor circulating around each ditch is equivalent to a flow rate of about 135 Mgal/d. However, the two oxidation ditches and the three clarifiers at Plant 2 were combined into a single process train with total basin volumes and areas equivalent to the sum of the individual units. Therefore, in the model, a single 2 Mgal oxidation ditch with a recirculation flow rate of 270 Mgal/d was used. The BioWin flow diagram used to represent the Plant 2 secondary treatment system is show in Figure 11-4.

Table 11-3 COD Fractions Used in BioWin Simulations

Symbol	Description and Comments	BioWin Default	Value Used
F_{up}	Fraction of total COD that is unbiodegradable particulate. This value can vary significantly from plant to plant. Higher values are common with a high COD/BOD ratio. Theoretical calculations for conversions between BOD and COD were used to determine a value of 0.28.	0.13	0.28
F_{bs}	Fraction of total COD that is soluble and biodegradable (i.e., readily biodegradable COD or rbCOD). This parameter is very important in anoxic basin sizing. A value of 0.17 was determined in the previous Master Plan Amendment 2 and was used in this study.	0.16	0.17
F_{us}	Fraction of total COD that is soluble and unbiodegradable. A value of 0.07 was determined in the previous Master Plan Amendment 2 and was used in this study.	0.05	0.07
K	SND Switching Function Constant. This value determines the extent that denitrification can occur in a reactor with low dissolved oxygen concentrations. A higher value results in increased simultaneous denitrification in an aerobic reactor. When the previous Master Plan Amendment 2 was prepared, the BioWin default for this parameter was 0.05 mg/L. The current version of BioWin uses a default value of 0.15, which has the net effect of indicating improved denitrification and allowing smaller anoxic sizing. The lower BioWin default value was used in the previous Master Plan and the new higher default value was used for this study.	0.15	0.15

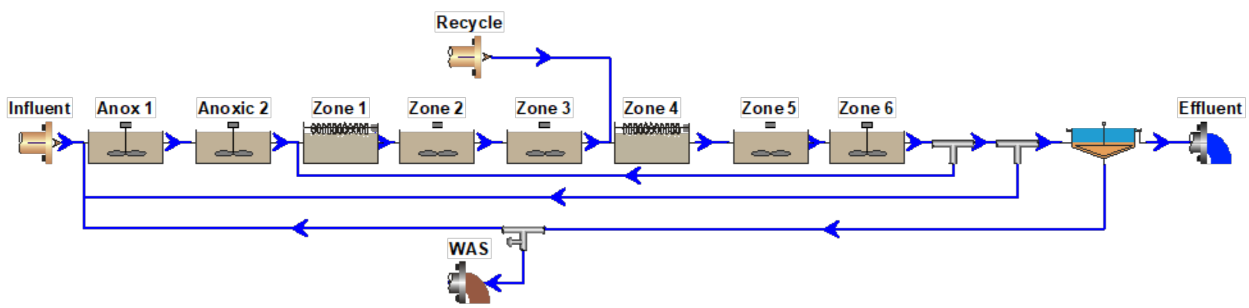


Figure 11-4 BioWin Flow Diagram for Plant 2 Secondary Treatment Facilities

As shown in Figure 11-4, the anoxic volume ahead of the ditch was modeled as two reactors in series, which is consistent with the design intent to compartmentalize these anoxic zones.

After some experimentation, it was determined that a dissolved oxygen setpoint concentration of 2.5 mg/L in Zones 1 and 4 (at the rotors), generally resulted in dissolved oxygen concentration of about 2.0 and 1.5 mg/L in the subsequent two zones, respectively, and in an average dissolved oxygen concentration of about 2.0 mg/L throughout the ditch.

As shown in the flow diagram, plant recycle streams were introduced between Zones 3 and 4, which represents the actual configuration in the field.

11.6 PLANT 2 CAPACITY EVALUATIONS USING THE SPREADSHEET MODEL

After preliminary evaluations, it was determined that process analyses should be accomplished over a range of aerobic MCRT values of 10 to 16 days and over a range of anoxic/aerobic volume ratios of 0.2 to 0.4. Accordingly, aerobic MCRT values of 10, 12, 14, and 16 days were evaluated at anoxic/aerobic volume ratios of 0.20, 0.25, 0.30, 0.35, and 0.40, resulting in 20 different combinations. The results of the 20 analyses are shown graphically in Figure 11-5, which shows the “potential capacity” of Plant 2 as a function of aerobic MCRT and the anoxic volume at each oxidation ditch. Since each oxidation ditch has a volume of 1.0 Mgal, the anoxic volume at each ditch in Mgal is numerically equivalent to the anoxic/aerobic volume ratio. The term “potential capacity” is used to indicate the capacity as limited by the volume of the ditches, the area of the clarifiers, and the RAS pumping rates. To realize the potential capacity, the nitrification and denitrification performance must be confirmed by BioWin simulations and the capacity of the oxygen delivery system (aeration rotors) must be adequate to support this capacity.

As shown in Figure 11-5, plant capacity is primarily a function of the aerobic MCRT and is only slightly impacted by the anoxic volume. The desired MCRT and anoxic volume are investigated further below.

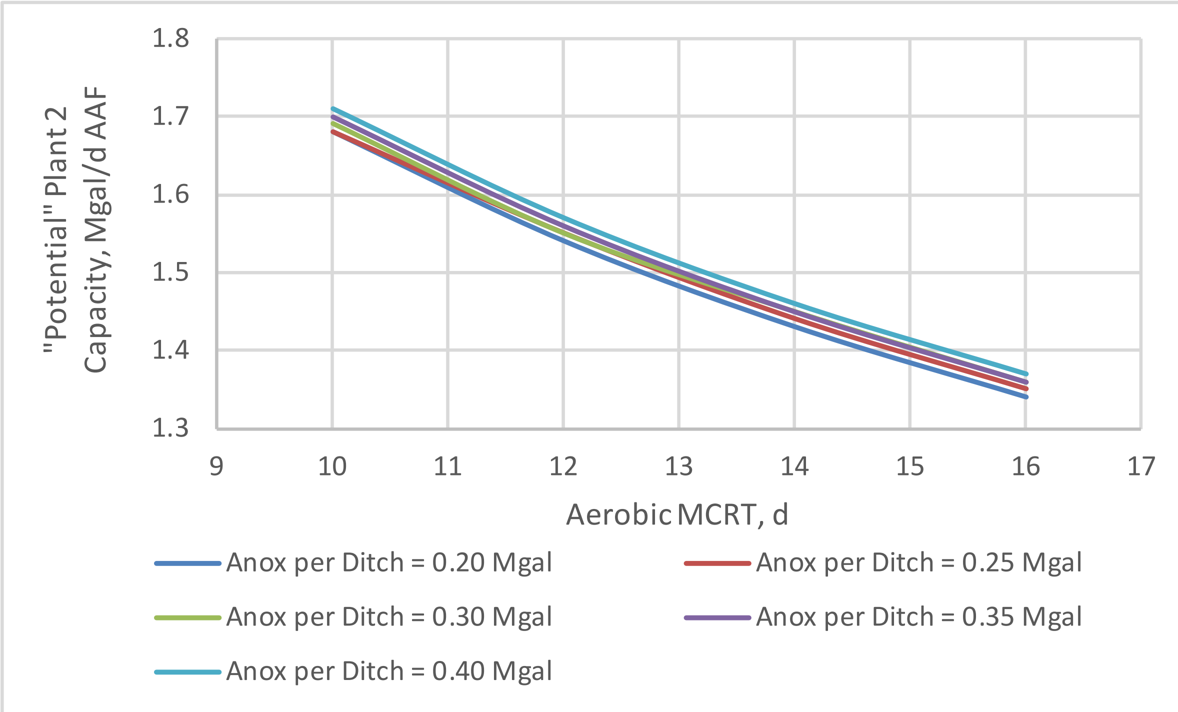


Figure 11-5 Plant 2 “Potential Capacity” Determined by Spreadsheet Model

11.7 PLANT 2 NITRIFICATION AND DENITRIFICATION PERFORMANCE DETERMINED FROM BIOWIN SIMULATIONS

Nitrification and denitrification performance was evaluated first by a series of steady state simulations and then refined by dynamic simulation as discussed below.

11.7.1 Steady State BioWin Simulations

A separate steady state BioWin simulation was performed for each of the twenty combinations of aerobic MCRT and anoxic/aerobic volume ratio described for the spreadsheet analysis. In each case, the influent flow rate used in BioWin was the capacity determined in the spreadsheet model. Key results are shown in Figures 11-6 through 11-8, which are discussed below.

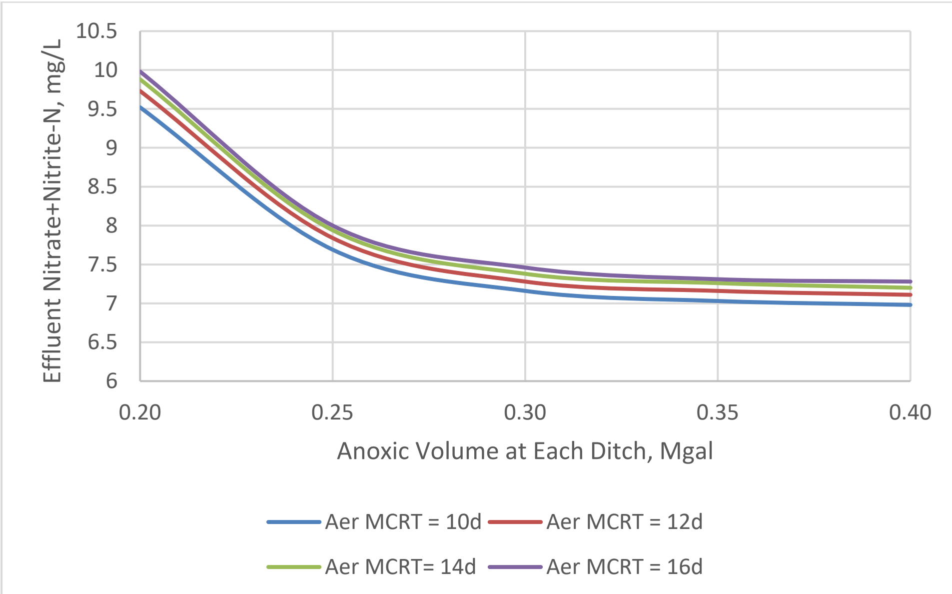


Figure 11-6 Effluent Nitrate+Nitrite-N Determined from BioWin Simulations

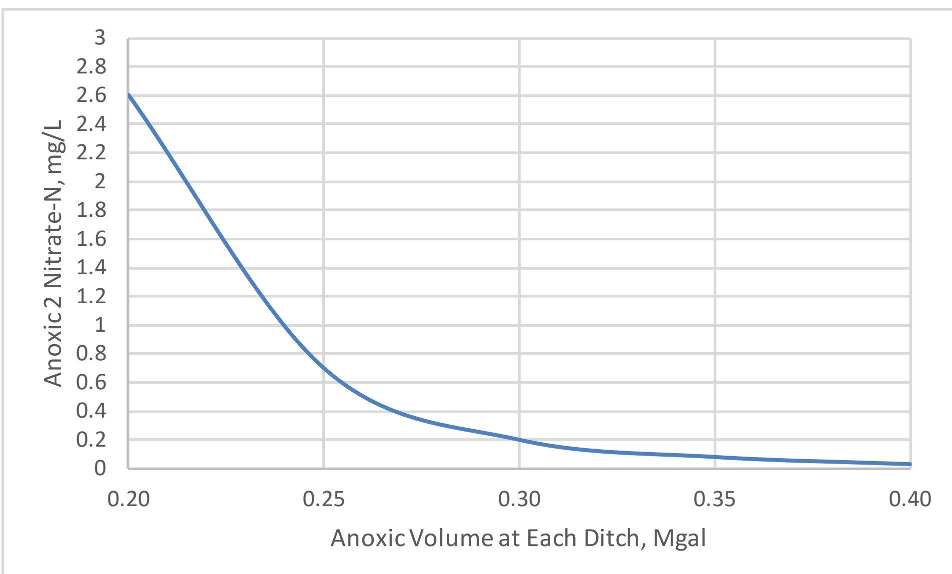


Figure 11-7 Anoxic2 Nitrate-N Determined from BioWin Simulations

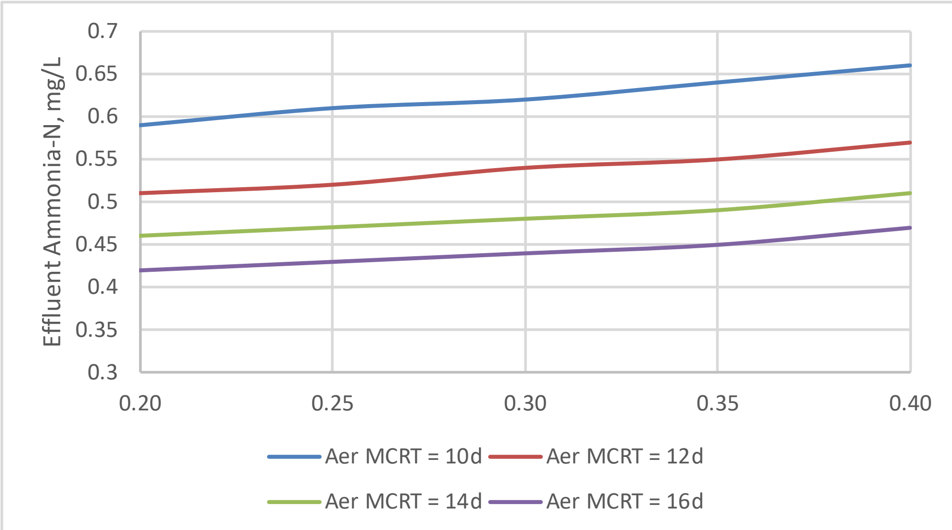


Figure 11-8 Effluent Ammonia-N Determined from BioWin Simulations

Appropriate sizing of the anoxic basin is indicated when essentially all of the nitrate-nitrogen returned to the anoxic basin is removed in the anoxic basin and the effluent nitrate+nitrite-N concentration remains within the design objective. In this case the design objective was to meet an effluent nitrate+nitrite-N concentration below 8.0 mg/L, providing a 2 mg/L safety buffer below the permit limit of 10 mg/L. As shown in Figure 11-6, this limit was satisfied for all anoxic volumes above 0.25 Mgal at each ditch, although the results for 0.25 Mgal are marginal and not recommended. The aerobic MCRT has only a minor impact on the denitrification performance. Essentially complete nitrate removal (<0.2 mg/L) in the anoxic zones was indicated for anoxic volumes over 0.30 Mgal per ditch (Figure 11-7). Higher nitrate concentrations in the second anoxic zone (Anoxic 2) are indicative of inadequate anoxic volume and/or inadequate readily biodegradable COD.

Although an anoxic volume of only 0.3 Mgal at each ditch would be expected to perform adequately, an anoxic volume of 0.35 Mgal would provide additional resiliency against potential adverse conditions, which could include a reduction in the influent readily biodegradable COD below the value assumed for this analysis (i.e., F_{bs} < 0.17). Another potential adverse outcome could occur if a value of the SND switching function constant lower than the current BioWin default used in this analysis was found to more accurately represent the performance of the Discovery Bay oxidation ditches after improvements. The value of the switching function constant is sensitive to the degree of mixing and to the extent to which oxygen delivery is distributed over the entire ditch volume rather than be localized at two rotor locations. Therefore, increased mixing and less DO variations in the ditch could occur with supplemental aeration equipment (discussed later in this section) and could lead to a lower switching function value after improvement than before. While it is believed that the current value of the switching function should be appropriate even after improvements, it is nice to have an additional safety buffer. For this reason, an anoxic volume of 0.35 Mgal at each ditch is suggested.

In the previous Master Plan Amendment 2, an anoxic volume of 0.4 Mgal at each ditch was suggested. This higher volume is believed to be mostly the result of the lower switching function constant value used at that time (0.05 mg/L, which was the BioWin default value at that time).

As shown in Figure 11-8, the effluent ammonia-N concentration is mostly a function of the aerobic MCRT, with some variation due to anoxic volume (higher ammonia concentrations with higher anoxic volumes). To provide a safety buffer below the permit limit of 0.7 mg/L, a target value of 0.5 mg/L is suggested. This would require an aerobic MCRT of at least 14 days.

11.7.2 Dynamic BioWin Simulations to Confirm Performance

Based on the steady state simulations discussed above, the recommended anoxic volume at each ditch is 0.35 Mgal and the tentatively recommended aerobic MCRT is 14 days. The spreadsheet model indicates a Plant 2 capacity of 1.45 Mgal/d average annual flow for these conditions.

To estimate the impact of diurnal flow and load variations, a hypothetical influent flow pattern was used in five-day dynamic BioWin simulations. The influent flow was assumed to be 50%, 100%, 150%, and 100% of the average annual flow (1.45 Mgal/d), respectively, in successive 6 hour blocks of time during each day. Influent concentrations for all parameters were held constant at the “worst-case” values previously indicated (i.e., 358 mg/L for BOD and TSS and 75 mg/L for TKN).

Several dynamic runs were completed based on BioWin default kinetics for the ammonia oxidizing bacteria (AOB) to investigate impacts of varying the DO and aerobic MCRT. A subsequent simulation was performed with revised AOB kinetics, which may be more representative of actual conditions in the oxidation ditches. All of the simulations are discussed below.

11.7.2.1 Dynamic BioWin Simulations with Default AOB Kinetics

The resulting variability in effluent ammonia-nitrogen concentrations and the daily average values that would be measured in hypothetical Plant 2 effluent flow-proportional composite samples are shown in Figure 11-9. A similar graph showing effluent nitrate- and nitrite-nitrogen results is presented in Figure 11-10.

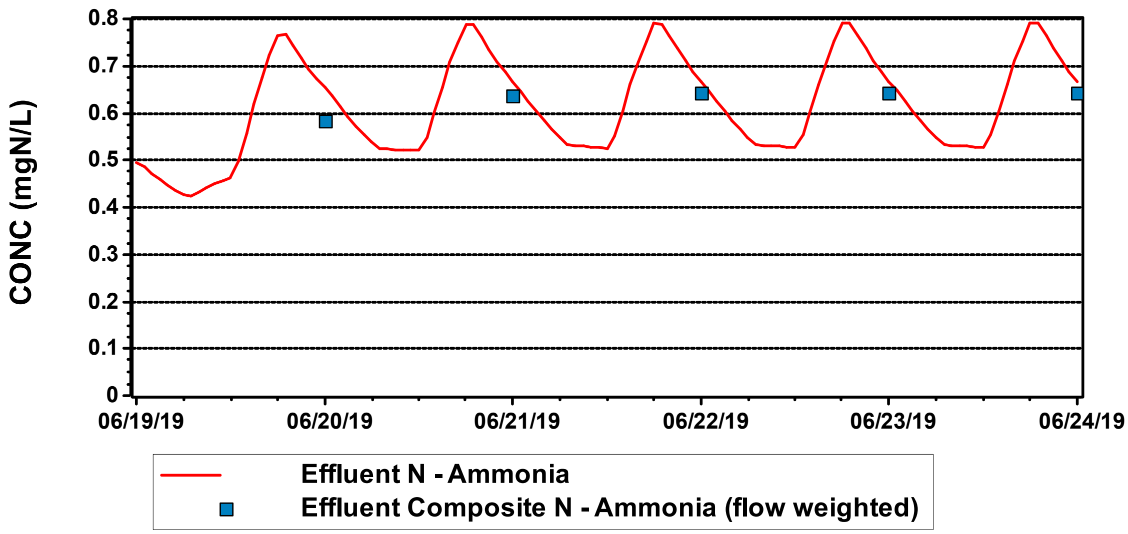


Figure 11-9 Effluent Ammonia-N Determined from Dynamic BioWin Simulation (1.45 Mgal/d, Aerobic MCRT = 14d, DO at Rotor = 2.5 mg/L)

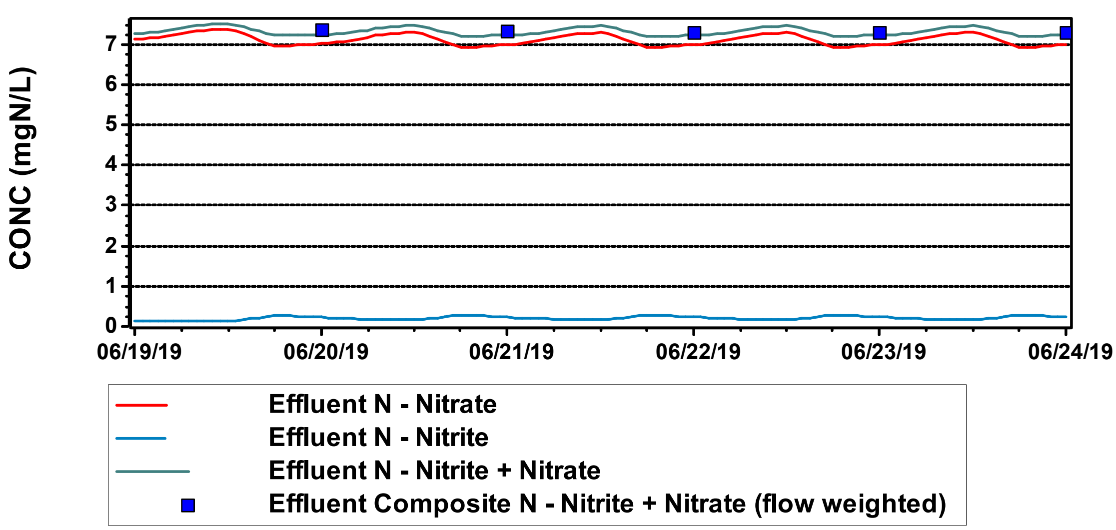


Figure 11-10 Effluent Nitrate and Nitrite Determined from Dynamic BioWin Simulation (1.45 Mgal/d, Aerobic MCRT = 14d, DO at Rotor = 2.5 mg/L)

As shown in Figure 11-9, the assumed diurnal flow and load variation resulted in significant diurnal variations in the effluent ammonia concentration and resulted in 24-hour flow weighted composite effluent ammonia-N concentrations near 0.64 mg/L, which is below the permit limit of 0.7 mg/L, but uncomfortably close. It is noted that these results are based on assumed diurnal flow and load variations and results could vary somewhat with an actual flow and load diurnal pattern for Discovery Bay. This topic should be investigated in detail during final design. From Figure 11-10, it is apparent that the effluent nitrate+nitrite-N was fairly stable and always below the target value of 8 mg/L.

To help lower the effluent ammonia concentration, the oxidation ditch dissolved oxygen concentration could be increased, but this would require more aeration capacity and would result in higher energy consumption than operation at lower dissolved oxygen. The results of a dynamic BioWin simulation with the dissolved oxygen concentration increased from 2.5 to 3.0 mg/L at the rotors are shown in Figures 11-11 and 11-12. As indicated in Figure 11-11, the effluent ammonia-N daily composite concentration was lowered to about 0.60 mg/L. Nitrate and nitrite performance remained very good (Figure 11-12).

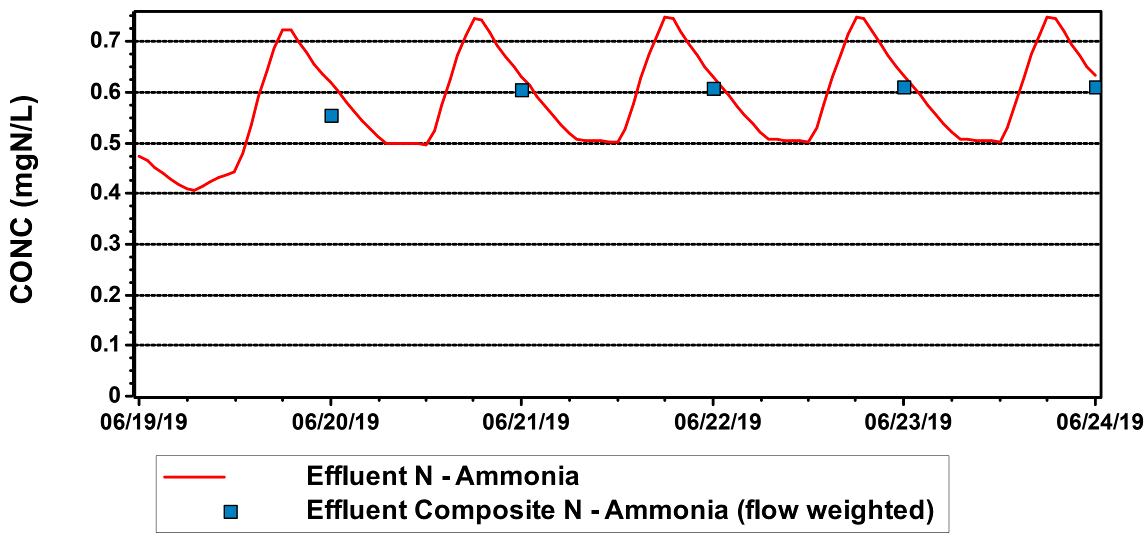


Figure 11-11 Effluent Ammonia-N Determined from Dynamic BioWin Simulation (1.45 Mgal/d, Aerobic MCRT = 14d, DO at Rotor = 3.0 mg/L

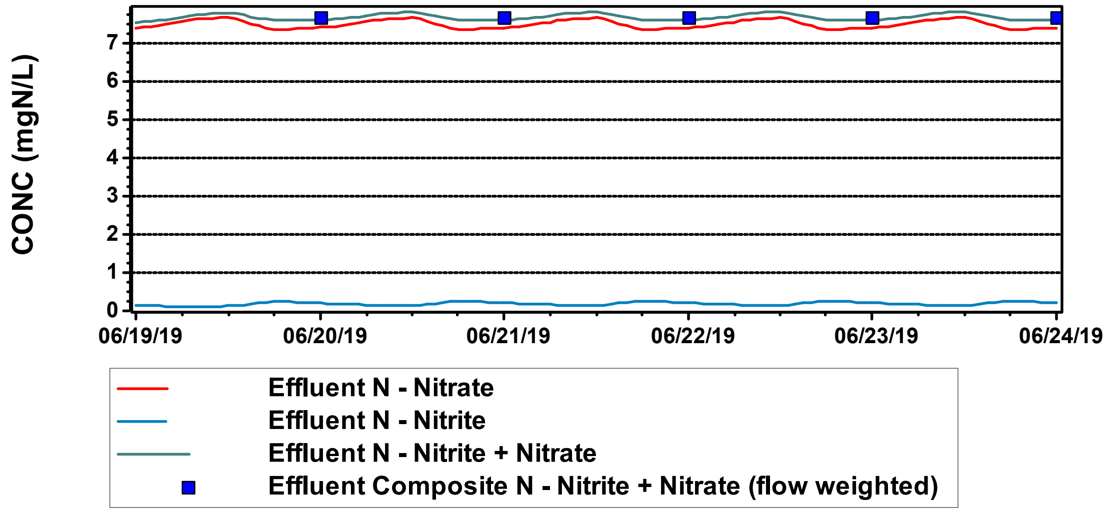


Figure 11-12 Effluent Nitrate and Nitrite Determined from Dynamic BioWin Simulation (1.45 Mgal/d, Aerobic MCRT = 14d, DO at Rotor = 3.0 mg/L)

To help further lower the effluent ammonia concentration, the aerobic MCRT was increased to 16 days and the influent flow was decreased to the corresponding capacity of 1.36 Mgal/d in another dynamic BioWin simulation. The dissolved oxygen concentration at the rotors was kept at the higher value of 3.0 mg/L. As shown in Figure 11-13, the effluent ammonia-N composite concentration was lowered to about 0.56 mg/L, while the nitrate+nitrite remained at desired levels (Figure 11-14).

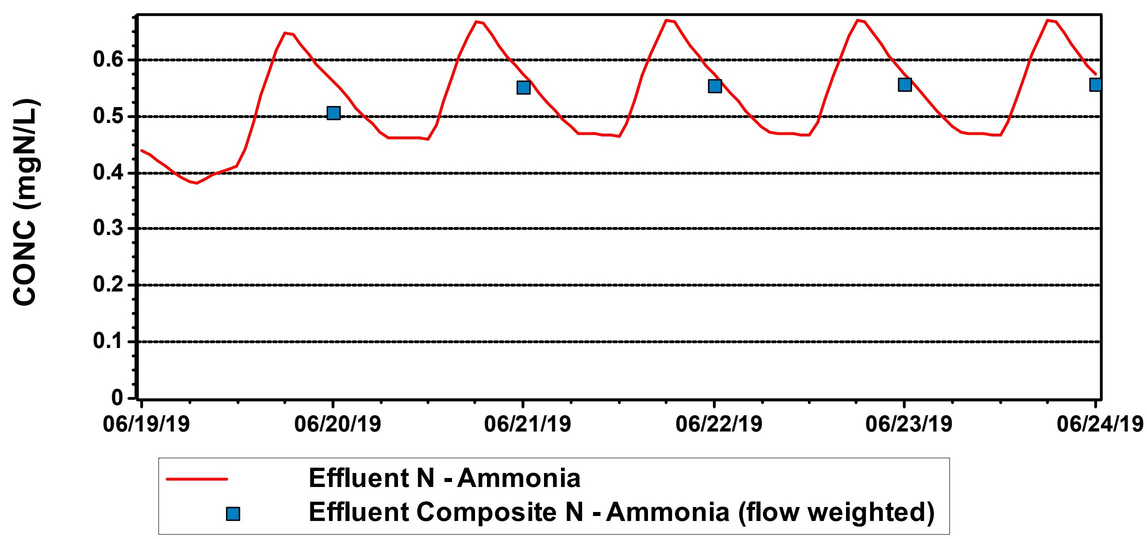


Figure 11-13 Effluent Ammonia-N Determined from Dynamic BioWin Simulation (1.36 Mgal/d, Aerobic MCRT = 16d, DO at Rotor = 3.0 mg/L)

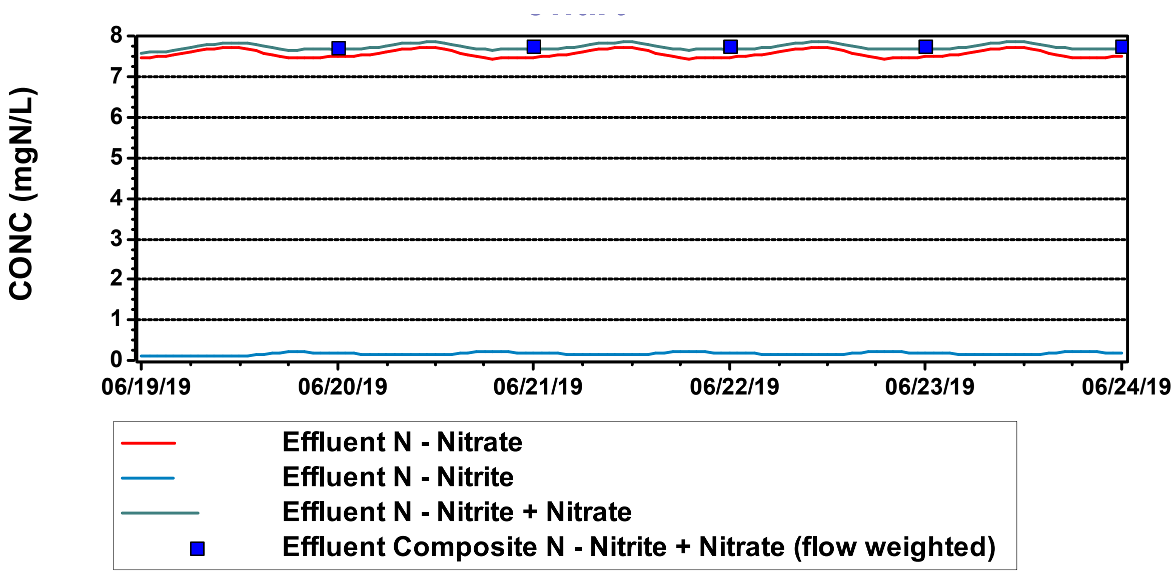


Figure 11-14 Effluent Nitrate and Nitrite Determined from Dynamic BioWin Simulation (1.36 Mgal/d, Aerobic MCRT = 16d, DO at Rotor = 3.0 mg/L)

11.7.2.2 Dynamic BioWin Simulations with Revised AOB Kinetics

Throughout the oxidation ditch, ammonia-n concentrations always will be very low and near the effluent concentration (typically below 0.7 mg/L). With these low concentrations, it is likely that AOBs that can scavenge ammonia at very low concentrations will be selected and acclimated. These type of bacteria are referred to as “K-strategists” because the ammonia-n concentration at which their growth rate is reduced to 50 percent of maximum (this is the ammonia half saturation constant Kn) is much lower than for AOBs that proliferate when ammonia concentrations are much higher (these are called μ-strategists [or r-strategists], where μ is the specific growth rate). For example, Kn values for K-strategists could be around 0.3 mg/L versus the 0.7 mg/L BioWin default for AOB. However, the maximum specific growth rate ($\mu_{max,20}$) for K-strategists are also believed to be lower than the BioWin default (perhaps 0.7 g/g-d versus 0.9 g/g-d), which partially offsets the decrease in Kn with regard to ammonia removal. The exact values for Kn and $\mu_{max,20}$ that will be applicable to the oxidation ditches in Discovery Bay is not well established in scientific literature, although it is generally recognized that values lower than BioWin defaults are appropriate. This topic was discussed with Dr. Christopher Bye, Senior Process Engineer and Director of Software Development at Envirosim, the developer of BioWin and with Dr. Imre Takacs, CEO of Dynamita and developer of the SUMO simulation software, which is similar to BioWin. Borth Drs. Bye and Takacs agree that it is entirely reasonable to use a lower Kn value for oxidation ditches and other nearly complete-mix reactors where the ammonia concentration is always and everywhere very low.

Based on the above, the dynamic BioWin simulation based on an average Plant 2 flow of 1.45 Mgal/d, an aerobic MCRT of 14 days, and a dissolved oxygen concentration of 2.5 mg/L at the rotors was repeated with a Kn value of 0.3 mg/L and a $\mu_{max,20}$ value of 0.7 g/g-d. The ammonia-n and nitrate+nitrite-n results are shown in Figures 11-15 and 11-16, respectively. As shown in Figure 11-15, the composite ammonia-n concentration was reduced to about 0.52 mg/L, compared to 0.64 mg/L when default AOB kinetics were used (Figure 11-9). The effluent nitrate+nitrite-n concentrations were not impacted by the change in AOB kinetics and remained under good control (Figure 11-16).

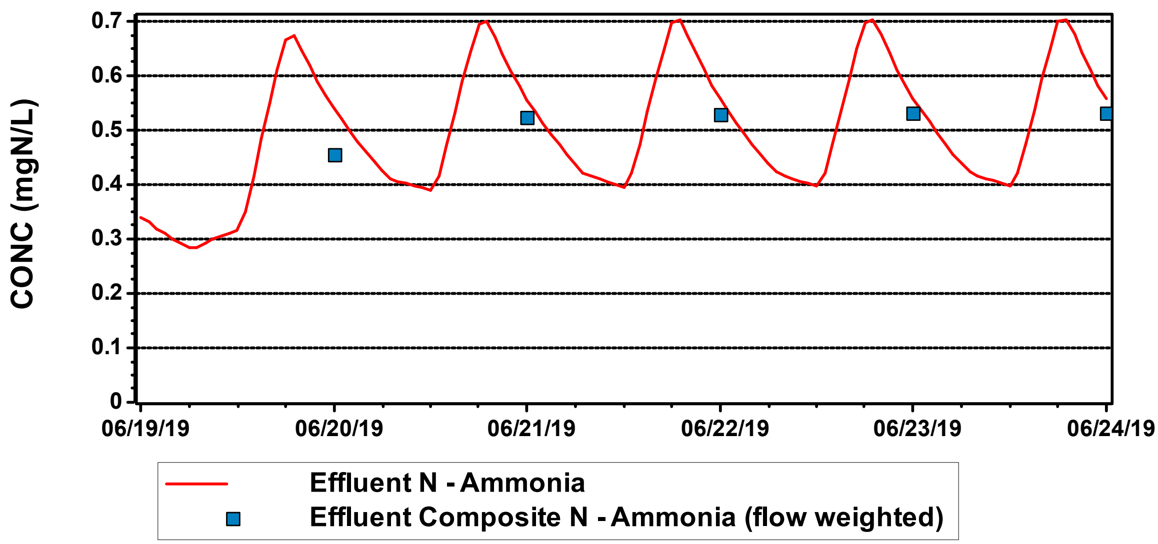


Figure 11-15 Effluent Ammonia-N Determined from Dynamic BioWin Simulation with Revised AOB Kinetics (1.45 Mgal/d, Aerobic MCRT = 14d, DO at Rotor = 2.5 mg/L)

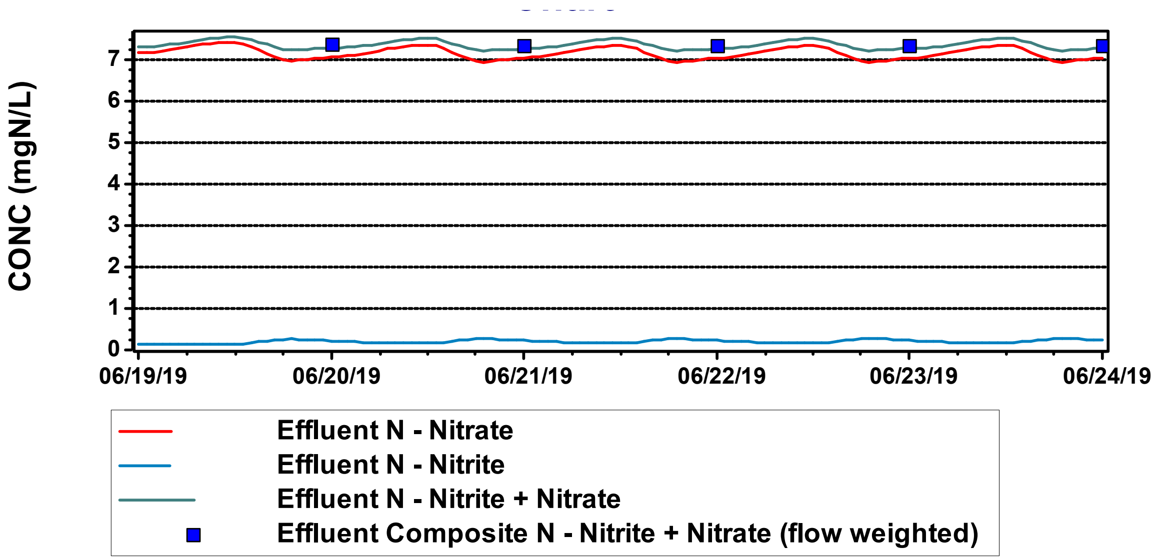


Figure 11-16 Effluent Nitrate and Nitrite Determined from Dynamic BioWin Simulation with Revised AOB Kinetics (1.45 Mgal/d, Aerobic MCRT = 14d, DO at Rotor = 2.5 mg/L

11.7.2.3 Conclusions from Dynamic Simulations

As discussed in the foregoing subsections, Plant 2 would be expected to easily meet effluent nitrate+nitrite-n requirements and just meet effluent ammonia-n requirements when operated at a capacity of 1.45 Mgal/d, an anoxic volume of 0.35 Mgal at each ditch, aerobic MCRT of 14 days, and a dissolved oxygen concentration at the rotors of 2.5 mg/L, when using BioWin default AOB kinetics. The effluent ammonia-n can be lowered by operating at a higher aerobic MCRT (for example, 16 days, which would lower Plant 2 capacity to 1.36 Mgal/d) and/or a higher dissolved oxygen concentration at the rotors (for example, 3.0 mg/L, which would require additional aeration capacity and would result in higher power costs compared to 2.5 mg/L). However, it is unlikely that it would be necessary to increase the aerobic MCRT or the dissolved oxygen concentration to attain ammonia-n concentrations safely below permit requirements, based on revised kinetics for K-strategist AOBs.

The recommended approach is to base the Master Plan on a Plant 2 capacity of 1.45 Mgal/d AAF (and corresponding capacity for Plant 1), with an anoxic volume of 0.35 Mgal at each ditch, an aerobic MCRT of 14 days, and dissolved oxygen concentrations of 2.5 mg/L at the rotors (2.0 mg/L average within the entire oxidation ditch volume). This determination should be confirmed during preliminary and detailed design when the plant influent characteristics database is updated based on revised influent sampling and after additional monitoring is completed to confirm the actual diurnal load pattern and fraction of readily biodegradable COD (F_{bs}). In the worst-case scenario, if a lower capacity is then established for Plant 2 (this is considered unlikely), more use of Plant 1 might be appropriate under critical worst-case operating conditions (peak month load combined with design peak hour flow, temperature of 13°C, and SVI of 175 mL/g).

11.8 PLANT 1 AND PLANT 2 CAPACITY ASSESSMENTS UNDER VARIOUS SCENARIOS

Capacity assessments for Plant 1 and Plant 2, each with an anoxic volume of 0.35 Mgal/d at each oxidation ditch, were completed using the spreadsheet capacity model for various scenarios. Two main flow and load conditions were evaluated: 1) cold temperatures with peak flows and loads, and 2) warm temperatures with average flows and peak loads. The cold temperatures with peak flows and loads scenarios correspond to the to the critical design conditions investigated previously and are based on a temperature of 13°C and an aerobic MCRT of 14 days. The warm temperatures with average flows scenarios are intended to represent conditions in the spring, summer, and fall months when oxidation ditches or clarifiers might be taken out of service for maintenance or repair. For these warm conditions, a temperature of 18°C was presumed (most representative of early spring and late fall) and the aerobic MCRT was set to 10 days. The highest diurnal influent peak flow associated with warm conditions and average flows was set at 1.7 times the average annual flow (compared to 3.0 used for the critical peak month). Results of the capacity analyses are shown in Table11-4.

Based on the results shown in Table 11-4, and as discussed previously, Plant 2 alone has a capacity of 1.45 Mgal/d annual average flow (AAF) under critical cold temperature design conditions and is not theoretically able to handle the full future design flow of 1.63 Mgal/d AAF. However, this is based on a combination of worst-case conditions for wastewater flows and loads, sludge settleability, and temperature. In actual practice, Plant 2 alone may be adequate to handle the entire future design flow for most of the year and perhaps throughout the year when conditions are more favorable than those assumed for this analysis.

Under the worst-case conditions discussed above, the capacity of Plant 1 with anoxic basin improvements is estimated to be 0.79 Mgal/d AAF. Therefore, the combined capacity of Plants 1 and 2 (2.24 Mgal/d AAF) would far exceed the future design flow (1.63 Mgal/d AAF).

In warm weather conditions, Plant 2 has a capacity of 1.86 Mgal/d AAF with one clarifier out of service and 1.37 Mgal/d with one oxidation ditch out of service. Therefore, at the future design flow of 1.63 Mgal/d, Plant 2 alone would be adequate with a clarifier out of service, but not with an oxidation ditch out of service.

The statements above are based on basin volumes and do not consider aeration capacity, which is discussed below.

Table 11-4 Secondary Treatment System Capacity Assessment Results

Scenario	Description	Mixed Liquor Temp, °C	Aerobic MCRT, days	Total MCRT, days	AAF(a) Capac., Mgal/d	Max Month MLSS, mg/L	Max Month WAS, lb/d
1	Plant 2, Cold, Peak Flows and Loads, All Units in Service	13	14	18.9	1.45	3,606	4,297
2	Plant 2, Warm, Average Flows, Peak Loads, All Units in Service	18	10	13.5	2.17	3,983	6,645
3	Plant 2, Warm, Average Flows, Peak Loads, One Clarifier Out of Service	18	10	13.5	1.86	3,417	5,700
4	Plant 2, Warm, Average Flows, Peak Loads, One Oxidation Ditch Out of Service	18	10	13.5	1.37	5,028	4,194
5	Plant 1, Cold, Peak Flows and Loads, All Units in Service	13	14	18.9	0.79	3,903	2,236
6	Plant 1, Warm, Average Flows, Peak Loads, All Units in Service	18	10	13.5	1.17	4,310	3,595
7	Plant 1, Warm, Average Flows, Peak Loads, One Clarifier Out of Service	18	10	13.5	0.90	3,307	2,759

(a) AAF = Average Annual Flow

(b) SOR = Standard Oxygen Requirement

11.9 EVALUATION OF AERATION CAPACITY AND SUPPLEMENTAL AERATION

The same spreadsheet model described previously in this section and used to generate Table 11-4 was used to determine standard oxygen requirements (SORs) for the ditches in Plant 1 and Plant 2 under various critical operating conditions with and without units out of service. The results are shown in Table 11-5. In all cases, peak month and peak hour loads were presumed.

The SORs shown in Table 11-5 can be compared to the estimated existing rotor capacities, which were developed in Section 11.1. As indicated in that section, the total standard oxygen delivery capacity per ditch with all four rotors running is estimated to be 7,260 lb/d, while the worst-case scenario with one rotor out of service (an inside rotor) results in a reliable oxygen delivery capacity of 5,110 lb/d.

For scenarios in which rotor capacity may be deficient (discussed below), one possible option for increasing capacity is to use portable floating rotors equivalent in capacity to the existing fixed rotors (30 hp). One such portable rotor is already existing at the plant, and all three ditches have been provided with features needed to allow use of the portable rotor. However, operation of the existing portable rotor has been problematic because it has blunt-end pontoons that tend to be pushed downward at the front end due to the oncoming water velocity in the ditch. It may be possible to get revised pontoons with pointed ends, such as used in pontoon boats, to overcome this problem, but this concept has not been proven. Additionally, it is currently unknown how hydrodynamic conditions in the ditches are impacted by a portable rotor and how the capacities of all rotors (fixed and portable) would be impacted by those conditions. For this analysis, it is assumed that pontoon-mounted portable rotors can be modified for successful operation in the ditches and that a 30 hp portable rotor would have a capacity of about 1,800 lb/d (about half-way between existing inside and outside rotors when all are running). Of course, these assumptions would have to be verified by appropriate investigations before a final decision could be made to rely on this solution. Alternative supplemental aeration systems should be investigated also as discussed later in this section.

As indicated in Table 11-5, If all three oxidation ditches were in service and the flow split was 35% to Plant 1 and 65% to Plant 2, the SORs in the oxidation ditches would be slightly higher in the summer (the first row in Table 11-5) than in the critical winter design condition (the second row in Table 11-5). In the summer, the required SOR in the Plant 1 ditch would be 6,135 lb/d, while the required SOR in each of the Plant 2 Ditches would be 5,734 lb/d. These are less than the total rotor capacity of 7,260 lb/d per ditch, indicating that no additional rotor capacity is needed if all existing units are in service. However, the required capacities are greater than the existing reliable rotor capacity of 5,110 lb/d per ditch. One portable rotor (perhaps the existing unit modified, or equivalent) could be used in any of the three ditches to mitigate the loss of a fixed rotor.

As indicated in the Table 11-5 (last two rows), the worst-case design condition occurs with peak summer temperatures and with Plant 1 out of service or one of the oxidation ditches in Plant 2 out of service. In this case, the SOR in each of the two ditches remaining in service would be 8,574 lb/d. This SOR could occur in any combination of two ditches, therefore all ditches (both plants) must be provided with a reliable rotor capacity of 8,574 lb/d. If all four existing fixed rotors were in service in a given ditch, the rotor capacity deficit would be 8,574 – 7,260 = 1,314 lb/d, which could be met with one portable rotor. However, if a rotor should fail, an additional portable rotor would be required in the ditch in question. Therefore, all three ditches would have to be capable of accommodating two portable rotors. Since two portable rotors would be required in one ditch (the one with a failed rotor) and one portable rotor would be required in the second of the two ditches in service under this scenario, a total of three portable rotors must be available and able to be relocated from one ditch to another. If fixed (non-portable) supplemental aeration systems were to be implemented, however, the equivalent of two portable rotors would have to be installed in all three ditches (total of six portable rotor equivalents). These conditions define the requirements for supplemental aeration.

Table 11-5 Oxidation Ditch Standard Oxygen Requirements Under Various Scenarios

Units Out of Service	Temp, °C	Aerobic MCRT, days	Total MCRT, days	Total Flow, Mgal/d	% Flow to Plant 1	% Flow to Plant 2	Plant 1 SOR (a), lb/d	Plant 2 SOR (a), lb/d	Plant 2 SOR (a) per Ditch, lb/d
None	13	14	18.9	1.63	35	65	6,135	11,468	5,734
None	25	10	13.5	1.63	35	65	5,996	11,152	5,576
Plant 1	13	14	18.9	1.45	0	100	0	15,696	7,848
Plant 1	25	10	13.5	1.63	0	100	0	17,148	8,574
Plant 2 Ditch	25	10	13.5	1.63	50 (b)	50 (b)	8,574	8,574	8,574

(a) Peak hour standard oxygen requirement (SOR) based on a dissolved oxygen concentration of 2.5 mg/L at the rotors, 2.0 mg/L average in ditch.

(b) Although Plant 2 with one ditch and three clarifiers in service would theoretically have more capacity than Plant 1 with one ditch and two clarifiers, a 50/50 flow split is selected to limit the oxygen requirement at Plant 2 to the value indicated in order to minimize standby aeration requirements in the oxidation ditch at Plant 2.

11.10 EVALUATION OF IN-GROUND CONCRETE BASINS VERSUS ABOVE-GRADE STEEL TANKS FOR ANOXIC VOLUME

Based on the analysis presented above, the recommended improvements include the construction of a 350,000-gallon anoxic basin ahead of each oxidation ditch, or the equivalent. Two alternatives are considered in this section: 1) in-ground concrete anoxic basins at each oxidation ditch, and 2) above-grade steel tanks at or near the oxidation ditches. Each of these alternatives is discussed below.

11.10.1 In-Ground Concrete Anoxic Basins

This alternative was recommended in the previous Master Plan Amendment 2 completed in 2015. At that time, the anoxic volume was to be 400,000 gallons (subdivided into two compartments) at each oxidation ditch. To fit within available site space, suggested inside dimensions for each of the 200,000-gallon compartments were approximately 41 feet square and 16 feet deep (liquid depth), subject to adjustment in detail design. With the reduction in anoxic volume to 350,000 gallons at each ditch (two 175,000-gallon compartments), the basin depth can be reduced from 16 feet to 14 feet, while maintaining the same footprint. However, compared to the previous estimated structural configuration, it is now recognized that a thicker slab will likely be required to resist groundwater buoyant forces. This results in increased concrete requirements, even though the basin depth is reduced. The final structural configuration is subject to verification in detail design. The proposed locations for the anoxic basins are shown in Figures 11-17 and 11-18, presented later in this document.

The desired internal mixed liquor recirculation (IMLR) flow from each oxidation ditch to its adjacent anoxic basin is 500% of the influent flow to that ditch. It is desirable to design the Plant 2 anoxic facilities to allow for the flexibility to treat the entire future design flow with Plant 1 out of service. In that case, the design average day maximum monthly flow to Plant 2 would be 1.96 Mgal/d, or 0.98 Mgal/d to each ditch. The corresponding diurnal peak flow is estimated at 1.5 x 0.98 Mgal/d = 1.47 Mgal/d, indicating a design IMLR flow rate of 7.35 Mgal/d at each ditch (500% of the influent flow). Two IMLR pumps, each with a capacity of 3.7 Mgal/d are suggested. It is considered adequate to have a spare pump stored on-site for reliability, rather than have three installed pumps per ditch. Each IMLR pump would be connected through a separate 16-inch pipeline with a magnetic flow meter. The IMLR pumps would be variable speed and controlled to obtain the desired ratio of flow to the plant influent flow. The return flow from each anoxic basin to the corresponding oxidation ditch would be accomplished with a new 36-inch pipeline to replace the existing 24-inch ditch influent pipeline.

For maximum operational flexibility and to have identical components, the improvements at Oxidation Ditch 1 in Plant 1 would be essentially the same as those at Oxidation Ditches 2 and 3 in Plant 2, except that the anoxic basins would be located to the side of the oxidation ditch (see Figure 11-18), instead of at the end, resulting in additional piping lengths.

A cost estimate for the proposed improvements is shown in Table 11-6.

Table 11-6 Cost Estimate for Concrete Anoxic Basins and Related Facilities

	Cost, $ (a)			
Item	Ditch 1 Anoxic	Ditch 2 Anoxic	Ditch 3 Anoxic	Total
Dewatering	165,000	165,000	165,000	495,000
Shoring	0	243,000	121,500	364,500
Excavation and Backfill	189,000	115,500	152,250	456,750
Concrete Structure and Guardrails	689,880	689,880	689,880	2,069,640
Pumps and Mixers	110,000	110,000	110,000	330,000
Piping and Appurtenances	251,800	120,600	120,600	493,000
Sitework	60,000	60,000	60,000	180,000
Electrical and Instrumentation	280,000	280,000	280,000	840,000
Subtotal 1	1,745,680	1,783,980	1,699,230	5,228,890
Subtotal 1, Rounded	1,746,000	1,784,000	1,699,000	5,229,000
Contingencies @ 20%	349,000	357,000	340,000	1,046,000
Subtotal 2	2,095,000	2,141,000	2,039,000	6,275,000
Engineering, Admin, and Environmental @ 25%	524,000	535,000	510,000	1,569,000
Total	2,619,000	2,676,000	2,549,000	7,844,000

(a) Mid 2019 cost level, ENR 20-Cities CCI = 11,300.

11.10.2 Steel Tank Anoxic Basins

Under this alternative, the anoxic volume per ditch and the IMLR flow per ditch would be the same as the concrete basin alternative. However, circular steel tanks above grade would be used instead of in-ground concrete basins. Additionally, for Plant 2, a single set of anoxic tanks would be used in conjunction with Oxidation Ditches 1 and 2. Therefore, for Plant 1, there would be two 175,000-gallon steel tanks, whereas for Plant 2, there would be two 350,000-gallon steel tanks. The tanks at each plant normally would be operated in series; however, piping would be provided to allow either one of the two tanks to be taken out of service while the other tank remains in service.

For this study, it is assumed that the water level in each tank would be 12 ft above grade. Although other configurations are possible, it is desirable to keep the water surface elevation somewhat low to minimize pumping requirements.

Currently, the influent and return activated sludge flows from the headworks into the oxidation ditches at each plant by gravity. Since it would be necessary to re-route these flows into the elevated tanks, a new pump station is required at each plant. Furthermore, since the IMLR flow from each ditch must also be pumped to the anoxic tanks, it would be cost-effective to combine the IMLR flow with the influent and RAS flow for combined pumping, avoiding separate IMLR flow pump stations at each ditch. The IMLR flow into the pump station from each oxidation ditch would be controlled by a motorized gate in the pump station. Providing flexibility for Plant 2 to take the entire influent flow (Plant 1 out of service), the required capacity of the pump station at Plant 2 would be as follows:

Peak Hour Influent Flow	4.89 Mgal/d
Maximum RAS Flow	3.0 Mgal/d
Maximum IMLR Flow	14.7 Mgal/d (from two ditches)
Total Pumped Flow	22.59 Mgal/d

The pump station at Plant 1 would have approximately half the capacity of that at Plant 2.

At each plant, a 24-inch influent pipe would be extended from the existing headworks to the new pump station. IMLR feed piping from each oxidation ditch to the pump station and IMLR return piping from the anoxic tanks back to the oxidation ditches would be 24 inches in diameter. A splitter box would be required at Plant 2 to split the return flows to Oxidation Ditches 2 and 3.

A cost estimate for the steel tank alternative is shown in Table 11-7. By comparing Tables 11-6 and 11-7, it is seen that the capital cost of the steel tank alternative is much higher than that for the concrete basin alternative. Additionally, the steel tank alternative would have higher power costs due to pumping into the anoxic basins. Therefore, the steel tank alternative is rejected.

Table 11-7 Cost Estimate for Above-Grade Steel Tank Anoxic Basins and Related Facilities

Item	Cost, $ (a)		
	Plant 1	Plant 2	Total
Combined Pump Station (b)	1,400,000	2,200,000	3,600,000
Anoxic Tanks with Mixers (b)	800,000	1,250,000	2,050,000
Site Piping	485,000	1,010,000	1,495,000
Mixed Liquor Splitter Box	0	120,000	120,000
Sitework	50,000	100,000	150,000
Subtotal 1	2,735,000	4,680,000	7,415,000
Contingencies @ 20%	547,000	936,000	1,483,000
Subtotal 2	3,282,000	5,616,000	8,898,000
Engineering, Admin, Environmental @ 25%	821,000	1,404,000	2,225,000
Total	4,103,000	7,020,000	11,123,000

(a) Mid-2019 cost level, ENR 20-Cities CCI = 11,300.
(b) Electrical and instrumentation included.

11.11 RECOMMENDED IMPROVEMENTS

Based on the evaluations presented in this section, the tentatively recommended secondary treatment improvements (to be verified during preliminary design) include the following:

- 350,000-gallon concrete anoxic basin with two compartments and mixers at each oxidation ditch.
- Two 3.7 Mgal/d submersible IMLR pumps in each oxidation ditch.
- Magnetic flow meter for each IMLR pump discharge in a concrete vault.
- If portable rotors are confirmed to be the best solution for supplemental aeration, provide three portable rotors (possibly including the existing unit modified and total capacity to be confirmed) to be located in any combination of two oxidation ditches (two rotors in one ditch and one in the other) and modify all ditches to include features (including electrical supply) needed to accommodate two portable rotors operating at the same time. Alternatively, provide other supplemental aeration systems (fixed or portable) that will meet the requirements discussed in this section, as modified by future investigations (see below).

Proposed layouts for the anoxic basins at Plant 1 and Plant 2 are shown in Figures 11-17 and 11-18.

The total capital cost for the anoxic basins and associated improvements is estimated to be approximately $7.8 million (from Table 11-8). At this time, an allowance of $0.8 million is suggested for supplemental aeration systems in the oxidation ditches, resulting in a total estimated capital cost of approximately $8.6 million.

While the improvements described above and the associated costs are believed to be reasonably accurate and are appropriate in the context of a Master Plan document, the following additional investigations must be completed to confirm recommended improvements prior to or during preliminary design:

1. As soon as possible, make improvements to the influent sampling systems and methods to assure representative results and accumulate a reliable database to be evaluated for design (this topic is discussed in more detail in Section 5).
2. After the new sampling system is implemented, complete special monitoring effort to determine diurnal load pattern and fraction of readily biodegradable COD.
3. Based on updated monitoring data and diurnal load pattern, confirm process design calculations for nitrification and denitrification performance and for aeration requirements.
4. Conduct investigations to confirm the oxygen delivery capacities of the existing brush rotors under various combinations of inside and outside rotors running.

5. After the capacities of the existing brush rotors are confirmed, investigate alternatives for providing any additional supplemental oxygen as may be required, noting that supplemental oxygen supply methods may impact the performance of the existing brush rotors. Alternative supplemental oxygen supply methods could include modified portable brush rotors, aeration diffusers (with blowers), jet aeration, and others.

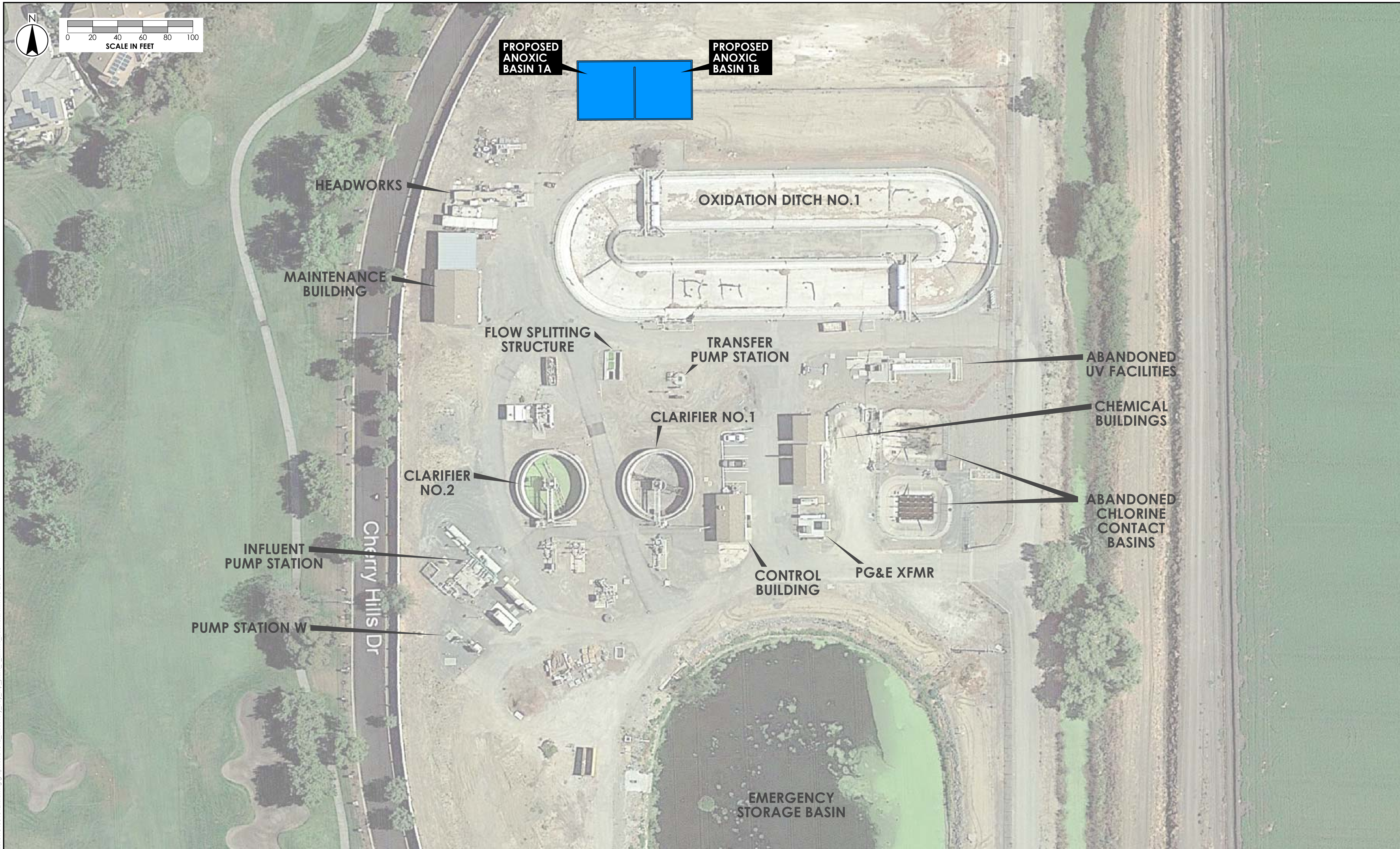


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Figure 11-17
Plant 1 Proposed Anoxic Basins

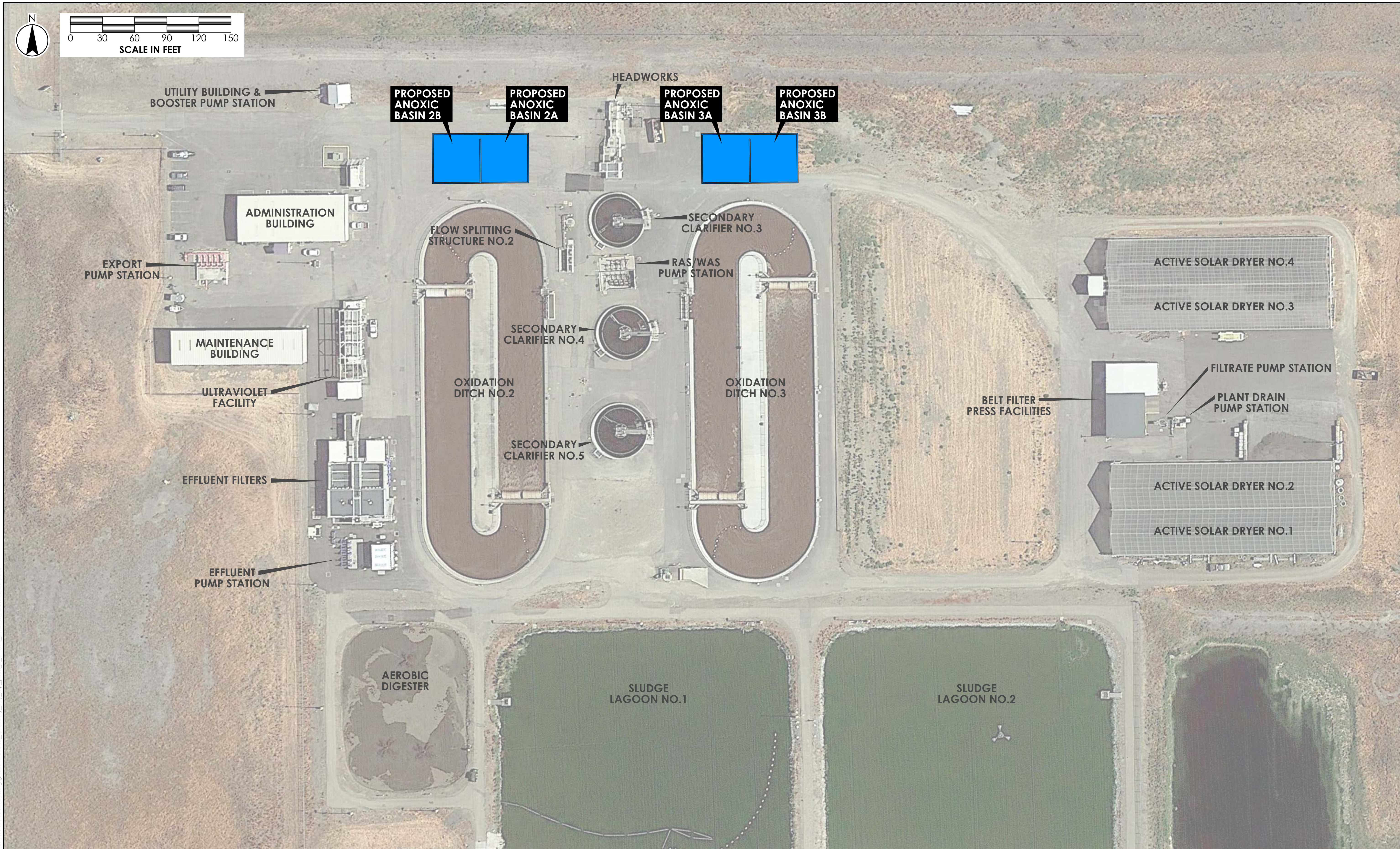

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Figure 11-18
Plant 2 Proposed Anoxic Basins

12.0 SECONDARY EFFLUENT LIFT STATION

The influent wastewater flow is split to Plants 1 and 2 at the Influent Pump Station and secondary treatment is provided separately by the two plants. The secondary effluent flows from the two plants are then re-combined in the sump of the Secondary Effluent Lift Station, which is located on the Plant 2 site. The Secondary Effluent Lift Station is used to pump the secondary effluent to the downstream filters, Parshall flume, and UV disinfection system. If desired, a portion or all the effluent flow of the Secondary Effluent Lift Station can be routed (temporarily) to the sludge lagoons. This feature is currently being used to trim flows in excess of 4.0 Mgal/d to the sludge lagoons as needed to avoid exceeding the current UV disinfection system capacity.

The Secondary Effluent Lift Station consists of a rectangular concrete sump that is mostly below grade, three large (12-inch discharge, 15 horsepower) and two small (8-inch discharge, 5 horsepower) vertical turbine pumps and ancillary facilities. As developed in Section 7, the reliable capacity of the pump station with two large and two small pumps running is approximately 5.6 Mgal/d, which exceeds the future design requirement of 5.13 Mgal/d (4.89 Mgal/d plus 5% recycle allowance).

No improvements to the Secondary Effluent Lift Station are needed.

13.0 TERTIARY FILTRATION

This section includes background information on the existing tertiary filters as well as consideration of possible flow equalization and dissolved air floatation facilities ahead of the filters.

13.1 BACKGROUND

In the previous Master Plan, dated February 2013, various alternatives for tertiary filters were investigated with and without flow equalization. The recommended project was to proceed with flow equalization and continuous backwash upflow sand filters. As discussed in Master Plan Amendment 2 Update, dated September 2015, continuous backwash upflow sand filters with methanol addition for denitrification were pilot tested at the plant, but found to be not cost-effective compared to anoxic basins for denitrification. Therefore, the District constructed the filters without added features for denitrification.

During detail design for the filters, the District opted to not build a dedicated flow equalization basin ahead of the filters. Instead, equalization would be accomplished by diverting excess peak flows into the sludge lagoons for later return and processing. The filters were designed with a reliable equalized peak flow capacity of 4.74 Mgal/d. At that time, the future (buildout) plant influent peak day and peak hour flows were projected to be 4.84 and 7.26 Mgal/d, respectively.

After the filters were put into service, filter backwash flows had not been optimized and excess backwash volumes were wasted to the sludge lagoons. This required a return flow from the sludge lagoons to the secondary treatment system. This operation with large return flows from the sludge lagoons was found to be unacceptable because the algae contained in the sludge lagoon return flow could not be adequately removed by the secondary treatment system and the filters, leading to poor quality filtered effluent. To mitigate this issue, filter backwash flows were routed directly to the secondary treatment system without going through the sludge lagoons and the filter backwash protocol was optimized to greatly reduce the volume of backwash water. The filters have been operating successfully in this manner for several years now.

Because of the problems created with large return flows from the sludge lagoons and the concern that problematic return flows could occur under buildout conditions, dedicated flow equalization ahead of the filters and dissolved air floatation treatment of lagoon return flows are considered below.

13.2 UPDATED CONSIDERATION OF FLOW EQUALIZATION

Based on recent flow reductions and the analysis presented in Section 5 of this document, the projected future plant influent peak day and peak hour flows are now 3.42 and 4.89 Mgal/d, respectively. Assuming a 5% recycle flow allowance, the future peak hour flow from the secondary treatment system is estimated to be 5.13 Mgal/d. Although it is reasonable to expect that the filters could pass this flow (loading rate with 6 filter cells in service would be 3.96 gpm/ft^2, which is acceptable), flows in excess of 4.2 Mgal/d are diverted to the sludge lagoons based on limitations of the downstream UV disinfection system and/or

Export Pump Station. Considering that the 4.2 Mgal/d limitation is relatively close to the peak hour secondary effluent flow (including recycles), it is expected that any such diversions that would occur in the future buildout condition would be extremely rare and short-lived. Therefore, resultant return flows from the lagoons, if any, would be relatively insignificant. As such, it is judged that new dedicated flow equalization facilities ahead of the filters would not be necessary or cost-effective. This judgement is a direct result of the recent reductions flow and the fact that the future design influent peak day flow has been reduced from 4.84 Mgal/d to 3.42 Mgal/d and the future design influent peak hour flow has been reduced from 7.26 to 4.89 Mgal/d for this Master Plan versus the previous Master Plan.

13.3 CONSIDERATION OF DISSOLVED AIR FLOATATION FOR SLUDGE LAGOON RETURN FLOWS

Dissolved air floatation facilities (DAF) can be used to remove algae from any return flows from the sludge lagoons. Therefore, use of the sludge lagoons for flow equalization and DAF treatment of return flows would be an alternative to dedicated flow equalization ahead of the filters. Additionally, DAF treatment could be used for any sludge lagoon return flows due to factors other than flow equalization.

As mentioned above, any return flows from the sludge lagoons caused by flow equalization ahead of the filters are now expected to be insignificant. Furthermore, filter backwash water will not be routed to the sludge lagoons. These facts substantially eliminate the need to consider DAF treatment.

Ongoing inflows to the sludge lagoons include decant flows from the aerobic digester, drainage flows (including filtrate and belt wash water) from the sludge dewatering belt presses, and rainfall on the lagoons. The total return flow from the sludge lagoons to the Decant Pump Station and subsequently to the secondary treatment system includes the net of the inflows offset by sludge dredging and evaporation from the lagoons.

Based on solids balance calculations, the future design annual average total return from the sludge lagoons is estimated to be about 0.08 Mgal/d, not including the impacts of any sludge dredging from the lagoons.

To estimate the possible impact of lagoon dredging on return flows, it is recognized that the belt press filtrate return flow to the lagoons is offset by the flow of sludge dredged from the lagoon. However, belt press wash water resulting from dewatering sludge from the lagoons would be a net flow to the lagoons. For example, if the equivalent of one belt press was used to dewater sludge from the lagoons and was operated 24 hours per week (typical current belt press run time), the wash water flow generated would be 70 gpm over 24 hours per week, which is an average of about 14,000 gpd (0.014 Mgal/d).

In the buildout condition, it is believed that sludge dredging from the lagoons will generally not be needed because pre-existing sludge will have been completely removed and no new sludge is planned to be added to the lagoons. Therefore, the 0.08 Mgal/d estimated return flow mentioned above is a reasonable estimate of the total buildout average return flow from the lagoons. This is approximately 5% of the average annual influent flow of 1.63 Mgal/d. It is believed that this level of return flow from the lagoons can be adequately handled while producing an acceptable filtered effluent without the need for DAF treatment of lagoon return flows. This statement is based on existing successful operations with even more return flow from the lagoons (as a percentage of influent flow) since lagoon dredging is currently being practiced.

13.4 SUMMARY

The existing filters are adequate for the buildout condition. Dedicated flow equalization ahead of the filters is not needed and DAF treatment of sludge lagoon return flows is not needed.

14.0 UV DISINFECTION

Ultraviolet (UV) disinfection is currently employed at the Discovery Bay Wastewater Treatment Plant (WWTP) to meet total coliform effluent limits and UV dose requirements specified in the WWTP's National Pollution Discharge Elimination System (NPDES) permit for discharge into Old River. Currently, the WWTP disinfection system is comprised of two channels, each equipped with Trojan Technologies Inc. (Trojan) UV3000Plus™ systems, as further described in Section 14.1.1. The UV disinfection system was designed by Trojan such that each channel is able to provide a dose of 100 mJ/cm^2, at a design flow of 4.8 million gallons per day (MGD) and a UV transmittance (UVT) value of 65%.

In October 2017, Moreland Consulting LLC (Moreland) conducted a spot-check bioassay and reported lower measured UV doses than those predicted by the validation model, in particular at higher flows (i.e., approximately 4.2 MGD). However, Moreland also reported considerable turbulence occurring in the channel at high flows. Moreover, the report does not clearly specify how the proper injection and mixing of microbial surrogates and SuperHume™ was ensured during the bioassays, and there is no discussion of hydraulic residence times between injection and sampling. A few additional issues that warrant further discussion with regards to the values reported by Moreland are discussed in this section.

In order to further assess some of the issues raised by Moreland during the 2017 report, evaluate issues identified by Stantec upon reviewing Moreland's report, and address some capacity questions raised by the Town, Stantec proposes the following work to be carried at the WWTP:

- Verification of hydraulic capacity of the UV channels,
- Confirmation of proper flow split between the two channels,
- Verification of appropriate mixing within each channel, along its width, length, and depth, and
- Confirmation of UV dose delivery at specific flows and UVT values.

This section provides a brief overview of the proposed UV disinfection performance verification approach and methodology.

14.1 BACKGROUND

At the WWTP, the combined secondary effluent from Plant 1 and Plant 2 is either pumped to the effluent filtration system or is diverted to the sludge lagoons. The two UV disinfection channels (Trojan UV3000Plus™ equipment) are located in Plant 2 downstream of the tertiary filters. This background section provides:

- An overview of the existing system,
- UV disinfection implementation history,
- UV disinfection system design criteria based on regulatory requirements specified in the WWTP's NPDES permit,
- A summary of relevant information included in the UV Disinfection Guidelines for Drinking Water and Water Reuse published by the National Water Research Institute (NWRI) in collaboration with the American Water Works Association Research Foundation (AWWARF), August 2012, (hereafter referred to as the 2012 NWRI Guidelines), and
- A summary of the 2017 Moreland report and the issues identified by Stantec.

14.1.1 UV Disinfection System Overview

Both UV disinfection channels contain four banks with 64 lamps each (i.e., 8 modules/bank and 8 lamps/module) for a total of 256 lamps per channel. The Trojan UV3000Plus™ system uses low-pressure high-output (LPHO), amalgam UV lamps. The system is programmed to continuously deliver a dose of 100 mJ/cm^2 to achieve the required total coliform limitation requirements and 5-log poliovirus inactivation. The system control center controls the number of online banks and the UV lamps ballast power level (between 60 and 100 percent). The total number of operating hours is recorded for each lamp. The system includes a fully automatic physical/chemical cleaning system.

Using the reduction equivalent dose (RED) prediction equation provided in Trojan's 2012 Addendum to their May 2007 UV3000Plus™ Validation Report (hereafter referred to as the 2012 Addendum), one channel with four banks online is predicted to disinfect up to a flow capacity of 4.85 MGD, at a UVT of 65%, which corresponds to the system design criteria. This is sufficient to treat the current peak hourly flow of 3.96 MGD and can "nearly" handle the future anticipated peak hourly flow of 4.89 MGD (discussed in more detail in Section 5 of this report). Performance, at this higher flow must be verified on-site. At the design UVT of 65%, the current design meets the redundancy level outlined in the 2012 NWRI Guidelines, which proposes that the WWTP either has a complete standby UV reactor train or a standby bank is available in each train (i.e., channel). The hydraulic capacity of the UV channels must also be verified to determine the maximum flow that will result in acceptable water levels in the UV channels.

As described in Section 14.1.5, from October 2016 to September 2019, a considerable percentage of UVT values were lower than the design value of 65%. This is important as UVT greatly impacts dose delivery. For example, for each channel, at UVT values of 55% and 60%, a dose delivery of 100 mJ/cm^2, can be achieved for flows up to 2.3 MGD and 3.4 MGD, respectively, instead of the 4.85 MGD.

14.1.2 UV Disinfection Implementation History

Plant 2, where the UV disinfection treatment unit is currently located, was constructed in the years 2000 through 2002. Both channels have undergone upgrades since being installed. In 2010, Channel 1 was upgraded to the Trojan UV3000Plus™ equipment from the previous Bailey/ Fisher and Porter UV system. In 2017, the Trojan UV3000™ system installed in Channel 2 was replaced with the Trojan UV3000Plus™ system. The current UV disinfection system drawings are shown in Figures 14.1 and 14.2.

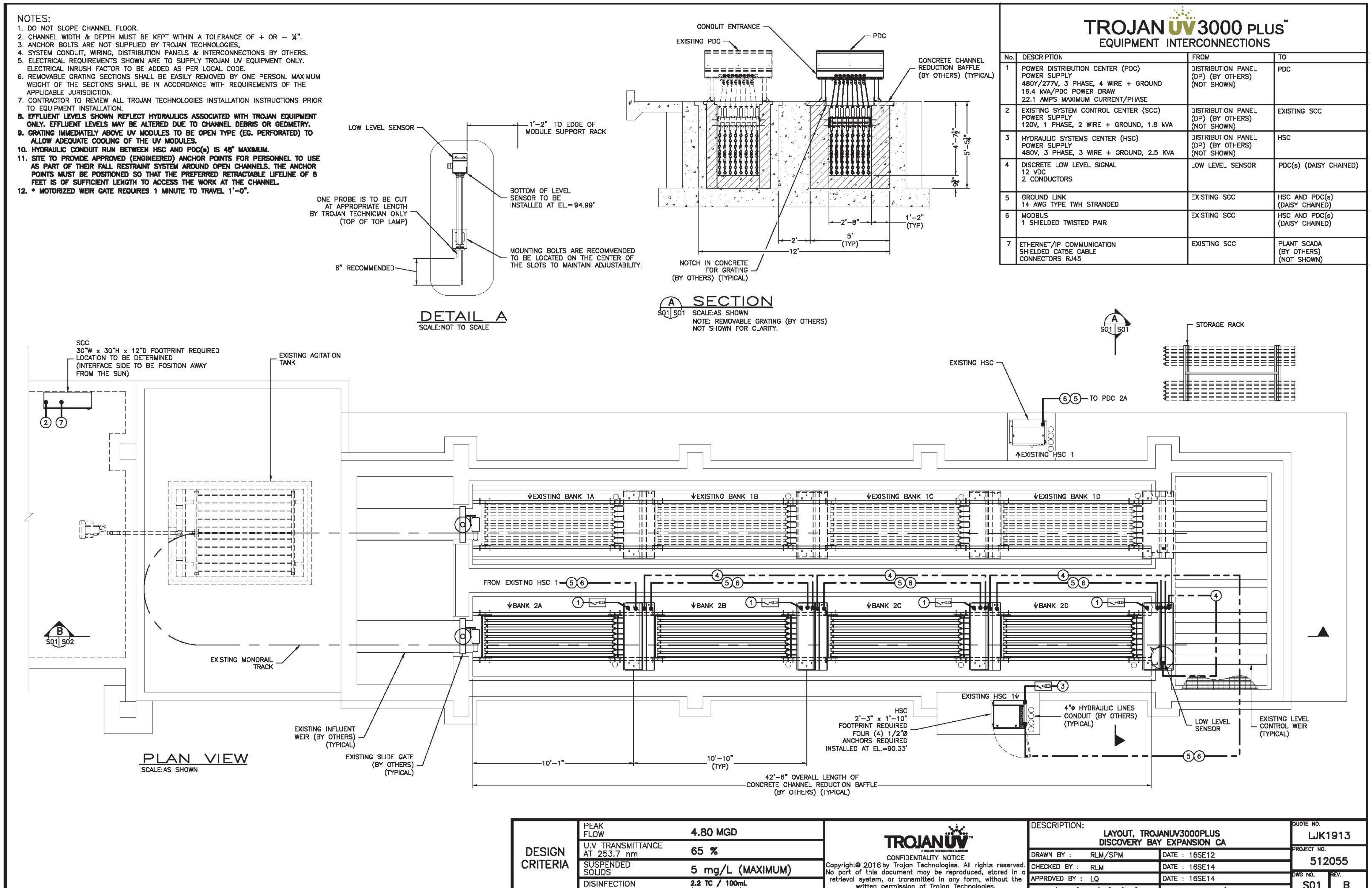


Figure 14-1 Plan view of the UV system at the Discovery Bay WWTP

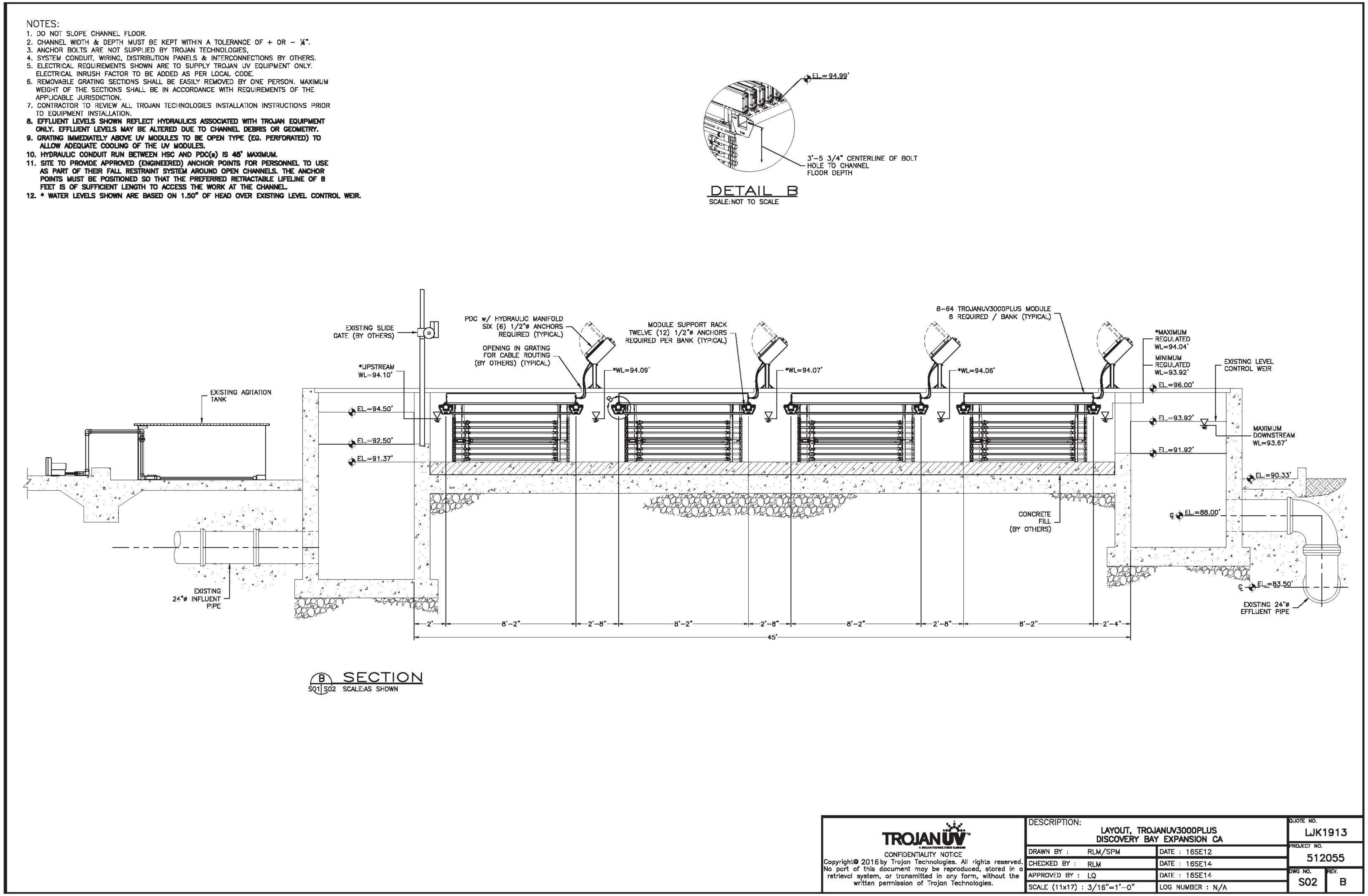


Figure 14-2 Profile view of the UV system at the Discovery Bay WWTP

14.1.3 Regulatory Requirements

The 2012 NWRI Guidelines apply to disinfected tertiary recycled water, as defined in California's Water Recycling Criteria, Title 22, Division 4, Chapter 3, of the California Code of Regulations (Title 22). The WWTP's current NPDES permit from 2014 and the future NPDES permit expected to take effect in December 2019 contain requirements that, for the most part, align with the 2012 NWRI Guidelines and Title 22 requirements, as described in this section.

<u>Total Coliforms</u>

- Interim total coliform effluent limitation (effective immediately through December 30, 2022):
  - Maximum 7-day median of 23 MPN per 100 mL
  - Cannot exceed 240 MPN/100 mL more than once in any 30-day period
- Final total coliform effluent limitation (effective December 31, 2022):
  - Maximum 7-day median of 2.2 MPN/100 mL
  - Cannot exceed 23 MPN/100 mL more than once in any 30-day period
  - Cannot exceed of 240 MPN/100 mL at any time

<u>UV Dose</u>

The minimum hourly average UV dose shall be:

- Interim UV Dose (effective immediately through December 30, 2022): 80 mJ/cm^2
- Final UV Dose (effective December 31, 2022): 100 mJ/cm^2

<u>Turbidity</u>

To ensure that filtration is performing adequately, and not negatively impacting UV disinfection, the filtered effluent turbidity shall not exceed:

- 2 NTU as daily average
- 5 NTU more than 5% of the time within a 24-hour period
- 10 NTU at any time

<u>UVT</u>

The minimum hourly average UVT value (at 254 nm), measured at UVS-001 and UVS-002, shall not fall below 55%.

14.1.4 Third-Party Spot-Check Performance Verification

Trojan performed an offsite validation of their equipment at the Whittier Narrows Water Reclamation Plant in Los Angeles, California county. The Trojan UV3000Plus™ equipment was validated per the 2003 NWRI Guidelines and the equations provided in the original validation report were updated per the 2012 NWRI Guidelines. The 2012 NWRI Guidelines specifies that a full-scale spot-check commissioning test be conducted to verify that the actual operation matches the intended design. In October 2017, Moreland conducted a spot-check bioassay at the Discovery Bay WWTP. A summary of critical results is presented in this section. The full report is available in Appendix A, for reference.

The Moreland report concluded that the UV system at the Discovery Bay WWTP did not meet the 2012 NWRI Guidelines performance requirements. The 2012 NWRI Guidelines require that at least eight tests be conducted on site, and at least seven of these eight tests must perform equally or better than predicted using performance equations developed during validation. A total of nine tests were performed (eight with UV light irradiation and one control without UV light irradiation) on bank B and/or C in Channel 2.

Table 14.1 is a reproduction of the data reported by Moreland. The values show that only four tests performed equally or better than predicted by the validation equation, and that in general, the UV system overperformed at lower flows, and underperformed at higher flows. Based on this data, scaling factors of 0.75 and 0.98 were used to downrate the UV disinfection system, for flow rates above and below 2.6 MGD, respectively. Based on these scaling factors, the report recommends a setpoint UV dose of 102 mJ/cm^2 when the flow is less than or equal to 2.6 MGD and a setpoint UV dose of 133 mJ/cm^2 when the flow is greater than 2.6 MGD, to ensure 5-log inactivation of poliovirus.

Table 14-1 Results of the Spot-check Bioassay Test (adapted from tables in the 2017 Moreland Report)

Test Run	Bank	UVT (%)	Ballast Power Level (%)	Flow		UV Dose (mJ/cm^2)		Scaling Factor[1] (SF)
				MGD	gpm/lamp/bank	Measured	Predicted	
1	C	66.35	100	4.232	45.92	24.43	32.64	0.75
2	B	66.20	100	4.198	45.55	20.77	32.60	0.64
3	C	66.35	76	2.593	28.14	41.47	34.99	**1.19**
4	B	66.30	76	2.601	28.22	34.13	34.81	0.98
5	BC	54.60	100	4.221	45.80	32.41	31.75	**1.02**
6	B	55.80	100	2.650	28.75	25.67	25.75	0.997
7 (control)		55.50	0	2.615	28.38	Control		
8	B	56.75	68	1.007	10.93	52.49	38.49	**1.36**
9	C	56.95	68	1.025	11.12	49.34	38.43	**1.28**

[1] Bold values indicate a scaling factor greater than 1

14.1.4.1 Observations and Comments

Following review of the 2017 Moreland report, Stantec identified a number of items that required further attention.

1. Incorrect end of lamp life (EOLL) factor and RED prediction equation

The Trojan UV3000Plus™ was validated per the 2003 NWRI Guidelines and the equations provided in the original validation report were updated per the 2012 NWRI Guidelines. The reduction equivalent dose (RED) is a function of several factors as shown below. The Trojan 2012 Addendum reports the updated RED prediction equation, including the factor values, to be used to calculate the predicted UV dose based on the specified factors.

$$RED = f(EOLL, FF, CR, Q, UVT, P, Banks)$$

Where, RED = Reduction equivalent dose (mJ/cm^2)

EOLL = end of lamp life (lamp aging factor)

FF = fouling factor

CR = confidence ratio

Q = flow rate (gpm/lamp)

UVT = UV transmittance at 254 nm (%)

P = ballast power setting as a percentage of maximum setting (%)

Banks = number of banks online

Trojan received conditional acceptance from the Department of Drinking Water (formerly the California Department of Public Health) for the use of an EOLL of 0.98. Trojan used an EOLL value of 0.98 in their validation testing. However, the 2017 spot-check bioassay test reported using an EOLL of 0.91. Moreover, it could not be confirmed whether the correct RED prediction equation from the Trojan 2012 Addendum was used to predict the REDs during the 2017 spot-check bioassay. Inputting an EOLL of 0.91 (with everything else the same) into the 2012 Addendum RED prediction equation yields different results than those reported in the 2017 Moreland report.

The predicted REDs calculated using the 2012 Addendum RED prediction equation (with an EOLL of 0.98, a fouling factor of 0.95, and the reported confidence ratio) are shown in Table 14.2 along with the resulting updated scaling factors. If an EOLL of 0.98, a fouling factor of 0.95, and the reported confidence ratio are used in Trojan's RED prediction equation with the test conditions reported in the 2017 Moreland report, six of the eight tests pass with a scaling factor greater than 1.

Table 14-2 Proposed Revised RED Values Using Trojan's Validation Equation

UVT (%)	Number of Banks	Ballast Power Level (%)	Flow (gpm/lamp/bank)	Measured UV Dose (mJ/cm^2)	Moreland Report Predicted RED (mJ/cm^2)	Moreland Report SF[1]	Updated RED[2] (mJ/cm^2)	Updated SF[1] [2]
66.35	1	100	45.92	24.43	32.64	0.75	30.39	0.80
66.20	1	100	45.55	20.77	32.6	0.64	30.35	0.68
66.35	1	76	28.14	41.47	34.99	**1.19**	32.58	**1.27**
66.30	1	76	28.22	34.13	34.81	0.98	32.41	**1.05**
54.60	2	100	45.80	32.41	31.75	**1.02**	29.56	**1.10**
55.80	1	100	28.75	25.67	25.75	0.997	23.98	**1.07**
56.75	1	68	10.93	52.49	38.49	**1.36**	35.83	**1.46**
56.95	1	68	11.12	49.34	38.43	**1.28**	35.77	**1.38**

[1] Bold values indicate a scaling factor greater than 1

[2] Updated values calculated using the RED prediction equation from Trojan's 2012 Addendum with an EOLL factor of 0.98, a fouling factor of 0.95, and the reported confidence ratio

2. Missed sample hold time

Appendix C of the 2017 Moreland report contains the raw data for the spot-check bioassay test. The date sampled was October 3, 2017. The date received and the analysis start date were October 5, 2017. The 2012 NWRI Guidelines states, “Samples shall be chilled immediately to 4°C and delivered to the laboratory and analyzed within 24 hours. Samples shall not be held for longer than 24 hours before analysis” (page 54). The minimum sample hold time of 24 hours was exceeded.

Stantec discussed this issue with GAP, and confirmed that this is not a concern, since treated wastewater is typically stable.

3. Collimated beam apparatus dose response curve sampling

The 2012 NWRI Guidelines states, “A series of sub-samples (five minimum) shall be exposed for a range of times calculated to achieve a range of UV doses from 20 to 150 mJ/cm^2, with a minimum interval of 25 mJ/cm^2. The exposed sample shall be plated in triplicate at dilutions appropriate to give 20 to 200 plaque forming unit per plate (pfu/plate)” (page 50). The collimated beam analysis dose response curve for the 2017 spot-check bioassay test reported four UV doses with an interval of 20 mJ/cm^2 (20, 40, 60 and 80 mJ/cm^2). Additionally, the raw data included in Appendix C does not show that the samples were plated in triplicate.

Stantec does not believe this issue is of concern. The UV doses used to develop the dose-response curve bracket the doses tested during the spot-check bioassay. Additionally, the standard collimated beam equation from the 2012 NWRI Guidelines (the same equation used in the Trojan validation) was used to convert the measured log inactivation values to UV doses.

14.1.5 Historical UVT Data

UVT data was provided from October 2016 to September 2019. The average daily UVT and minimum daily UVT over time is shown in Figure 14.3. Figures 14.4 and 14.5 show the UVT percentiles for average daily UVT and minimum daily UVT, respectively. For any selected UVT value, the figures show the percentage of the recorded measurements the UVT values were equal to or lower than that selected value. For example, for Figure 14.4, approximately 48% of the reported average daily UVT measurements were equal to or lower than 64%.

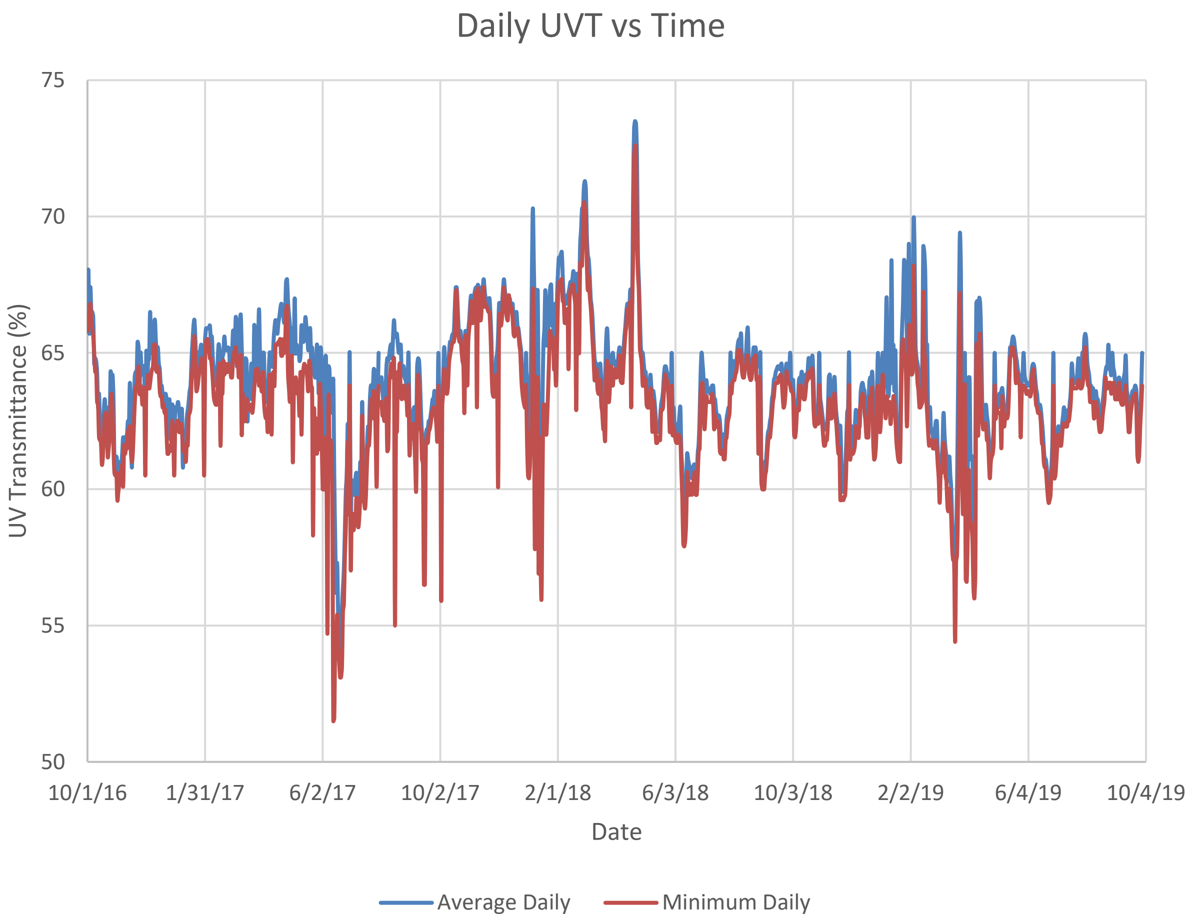


Figure 14-3 Average and minimum daily UVT from October 2016 to September 2019

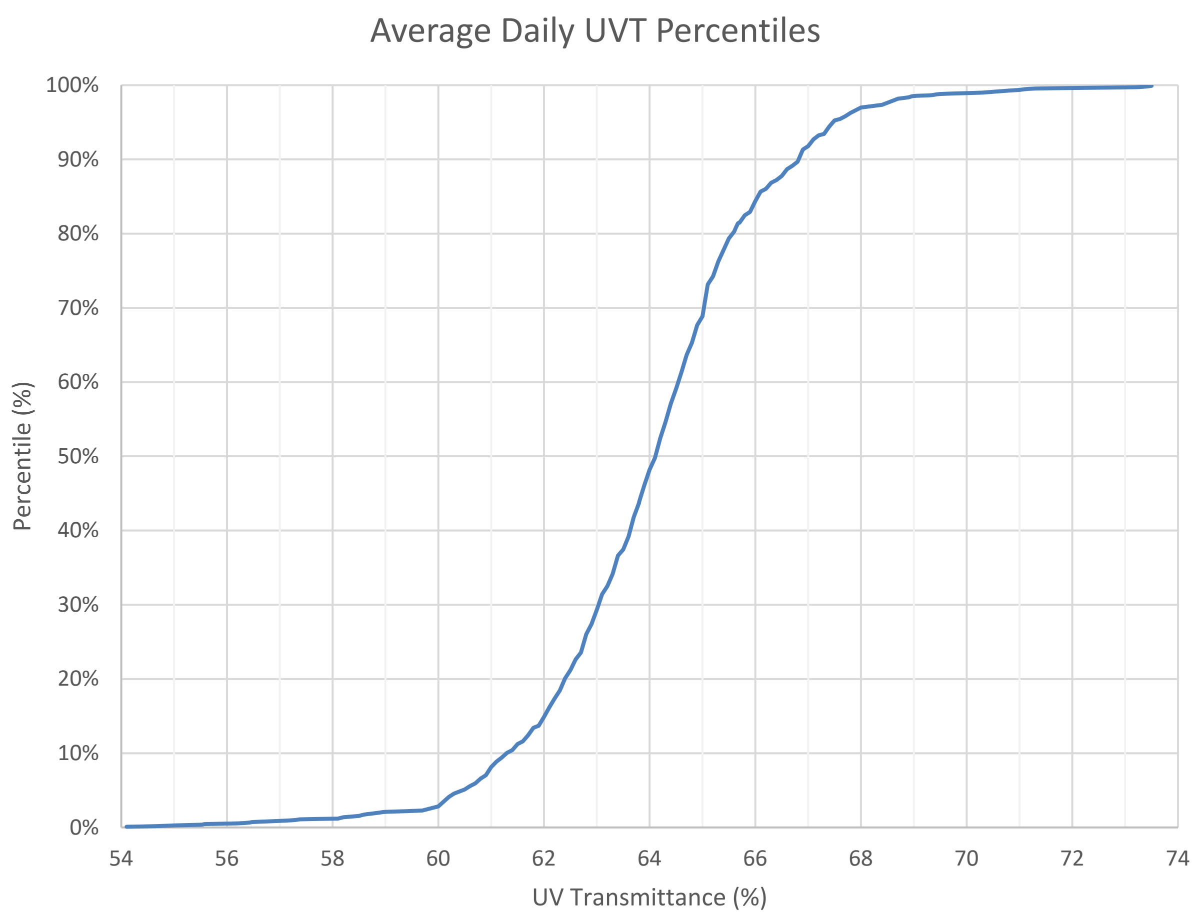


Figure 14-4 Average daily UVT cumulative percentile plot. Includes data from October 2016 to September 2019

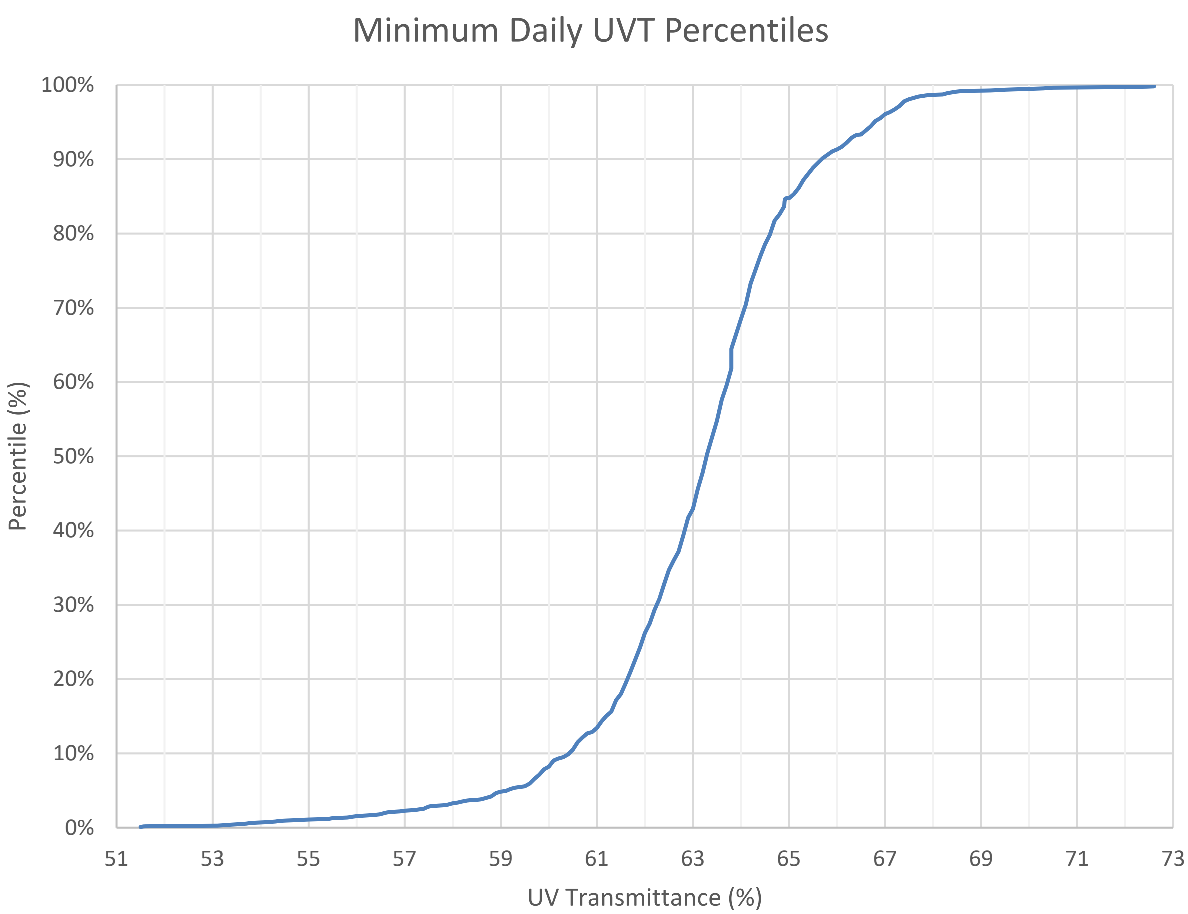


Figure 14-5 Minimum daily UVT cumulative percentile plot. Includes data from October 2016 to September 2019

14.1.5.1 Impact of UVT on UV Disinfection Performance

UVT greatly affects the UV dose delivered. As mentioned previously, one channel was designed to deliver a UV dose of 100 mJ/cm^2 for a design flow of 4.8 MGD at a UVT of 65%. However, as shown in the section above, a considerable percentage of the UVT values measured onsite is below 65% (84.7% of the minimum daily UVT values were lower than 65%). At lower UVTs, the disinfection capacity of the UV channels is reduced. Figures 14.6 and 14.7 show the relationship between required UVT and flow rate to achieve a UV dose of 100 mJ/cm^2 as calculated based on the Trojan RED prediction validation equation (assuming a ballast power level of 100%). Figure 14.6 shows the predicted REDs with all four banks in one channel online (to satisfy the redundancy option of one complete standby channel suggested by the 2012 NWRI Guidelines). Figure 14.7 shows the REDs with both channels online and three banks online per channel (to satisfy the redundancy option of one standby bank per channel suggested by the 2012 NWRI Guidelines).

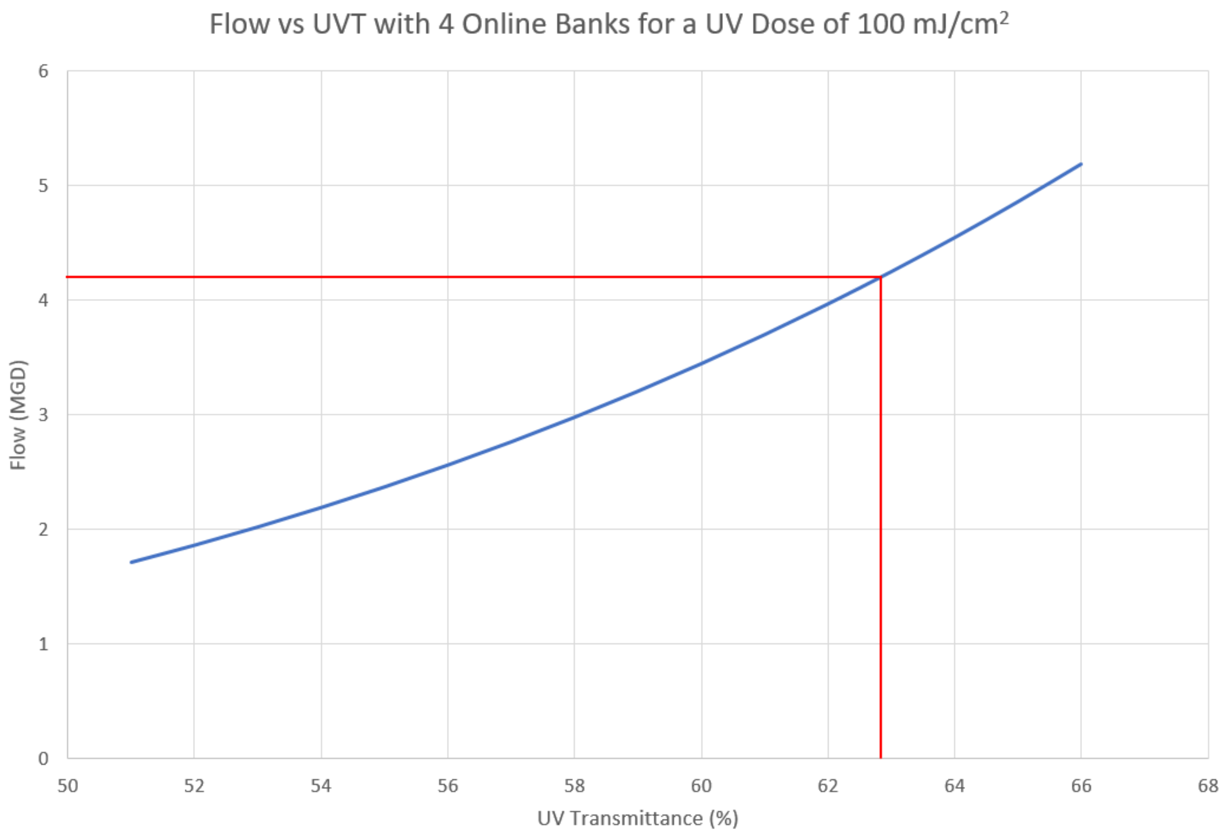


Figure 14-6 Relationship between UVT and flow for a RED of 100 mJ/cm2 with one channel (four banks) online. The red lines show a flow of 4.2 MGD and the corresponding UVT required to achieve a RED of 100 mJ/cm2

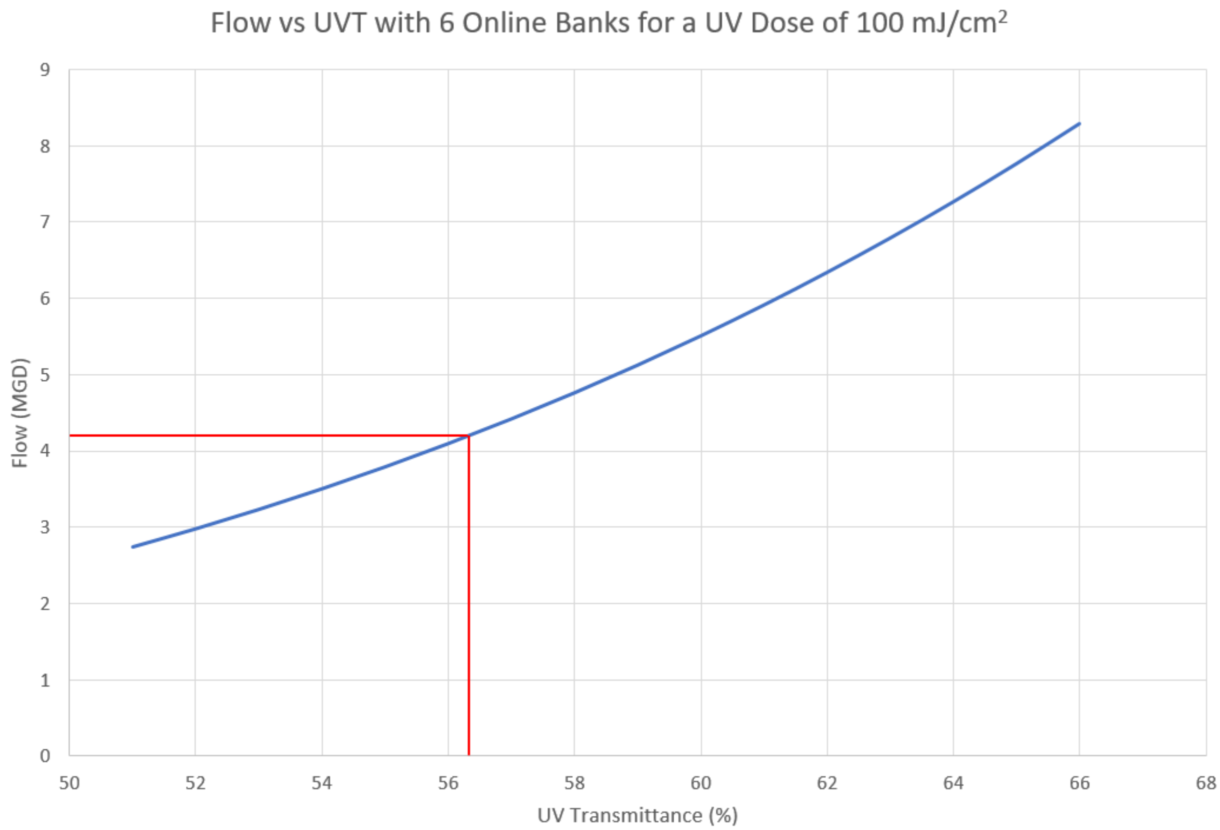


Figure 14-7 Relationship between UVT and flow for a RED of 100 mJ/cm2 with two channels online and three banks online per channel. The red lines show a flow of 4.2 MGD and the corresponding UVT required to achieve a RED of 100 mJ/cm2

Both Figures 14.6 and Figure 14.7 show the flow that can be disinfected four or six banks online, under the assumed conditions, as filtered water UVT varies. The hydraulic capacity of each UV channel must be verified. Although, there would currently be a 4.2 MGD limitation based on the effluent pump capacity downstream of the UV channel. Assuming hydraulics do not limit performance, at a flow of 4.2 MGD, one channel could deliver the required UV dose for any UVT values above 62.9%. Therefore, two channels would be required to deliver the required dose, below these UVT values. The data reported from September 2016 to October 2019, show that 58.3% of the minimum daily UVT measurements and 72.6% of the average daily UVT measurements were above 62.9%. Six banks online (three per channel) can deliver the required UV dose for any UVT above 56.4% at a flow of 4.2 MGD. From September 2016 to October 2019, 98.3% of the minimum daily UVT measurements reported and 99.4% of the average daily UVT measurements reported were above 56.4%. Therefore, for most of the UVT data from the last three years, the required UV dose can be delivered for flows up to 4.2 MGD, using the two existing channels. Stantec understands that the existing channels are set up to run one channel at a time. System controls would need to be modified to allow for dual channel operation.

As shown in Figures 14.4 - 14.7, for many of the UVT conditions currently measured on site, it is necessary to use two channels to achieve the required UV dose of 100 mJ/cm^2. Assuming the UV channels are operating as designed and the channels have proper hydraulics, the UV channel will deliver the required UV dose for all flow and UVT condition relationships below the blue lines in Figures 14.6 and 14.7 for four or six banks online, respectively. If the flow and UVT condition relationship is above the blue line in Figure 14.6, one channel is not sufficient to deliver the required dose. If the flow and UVT condition relationship is above the blue line in Figure 14.7, flow must be diverted to the sludge lagoons upstream of the tertiary filters.

For most normal flows and UVT conditions, one channel with four banks online is sufficient to deliver the required UV dose. Two channels will likely be needed during periods of wet weather flows and for periods of low UVT. For conditions where the 100 mJ/cm^2 is not met with six banks online, flow must be diverted to the sludge lagoons upstream of the tertiary filters.

14.2 PROPOSED UV DISINFECTION SYSTEM PERFORMANCE ASSESSMENT

After reviewing the 2017 Moreland report, it is proposed to evaluate the performance of the UV system at the Discovery Bay WWTP, once the issues outlined in this report are resolved (e.g., proper hydraulics through the channel and appropriate mixing). An overview of the proposed performance evaluation approach is summarized below. The main components of the system performance assessment include a hydraulic evaluation and retesting the delivered UV dose for different flows and UVTs.

14.2.1 Hydraulics Evaluation

Hydraulics can greatly affect the performance of a UV system. Stantec recommends that a hydraulic evaluation be completed prior to the system performance assessment. The hydraulic capacity of the channels must be verified to confirm whether it can treat flows up to 4.2 MGD. Additionally, it must be determined if any preferential paths exist, if there is appropriate mixing, and if there is proper flow splitting between the two channels. Potential hydraulic improvements may be recommended based on the results of the hydraulic evaluation.

14.2.1.1 Velocity Profiles

A proposed outcome of the Moreland report was to downrate the UV disinfection system 25%, or by a factor of 0.75, for flows above 2.6 MGD. This factor was derived through a comparison between predicted and measured UV dose delivered values. In practice, this would mean that the Discovery Bay systems would have to deliver 133 mJ/cm^2 instead of 100 mJ/cm^2 to meet the disinfection requirements outlined in Section 14.1.3.

However, the report highlights concerns associated with turbulence at high flows, conditions under which lower UV doses were measured than those predicted. The impact of this observed turbulence and dose delivery was not evaluated.

In addition, the report does not address the following issues:

- Microbial surrogate and UV absorber mixing – the information provided did not detail how the microbial surrogate (i.e., MS-2 bacteriophage) and the UV absorber (i.e., SuperHume^{TM}) were mixed, or how much time was allowed between injecting the microbial surrogate and UV absorber and collecting samples. A total of five hydraulic residence times (HRTs) between injection and collection is the typical accepted standard to allow for conditions to stabilize prior to collecting samples, which is also stated in the 2012 NWRI Guidelines. One HRT (normally in minutes) is defined as the volume of the channel divided by the flow rate. Simplistically, this is the amount of time that one needs to allow for the volume in the channel to be replaced. Therefore, a safety factor of five is normally allowed to be conservative. Lower HRTs can be justified based on site-specific data.
- Flow splitting between channels – due to inherent limited scope of the Moreland report, no information was provided about flow splitting between channels. Stantec believes this is important, as it is the desire of the Town to use the channels in a duty/standby mode, and thus proper flow splitting must be verified.

Stantec recommends verifying hydraulic profiles, as well as flow splitting at different flows. The proposed approach would rely on monitoring velocities along the channel across its width, length, and depth. Figure 14.8 shows an example of a cross-section gridline, which can be used for velocity point measurements. A comparison of point velocities and area average values would then be compared to provide information on preferential paths, if any. Figure 14.9 shows an example of possible cross-section locations.

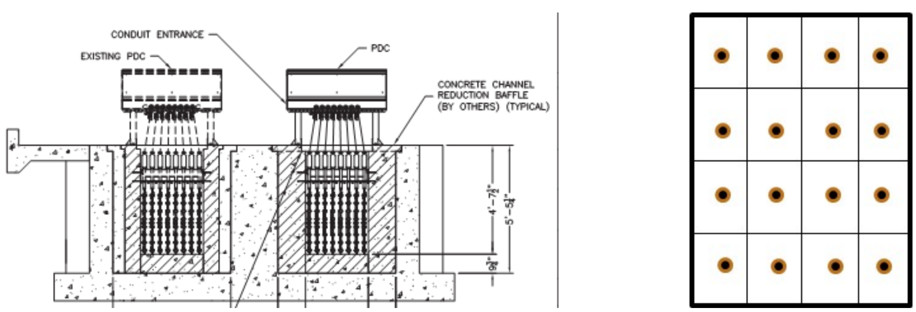


Figure 14-8 Example of a Channel Cross-Section Velocity Profile Gridline

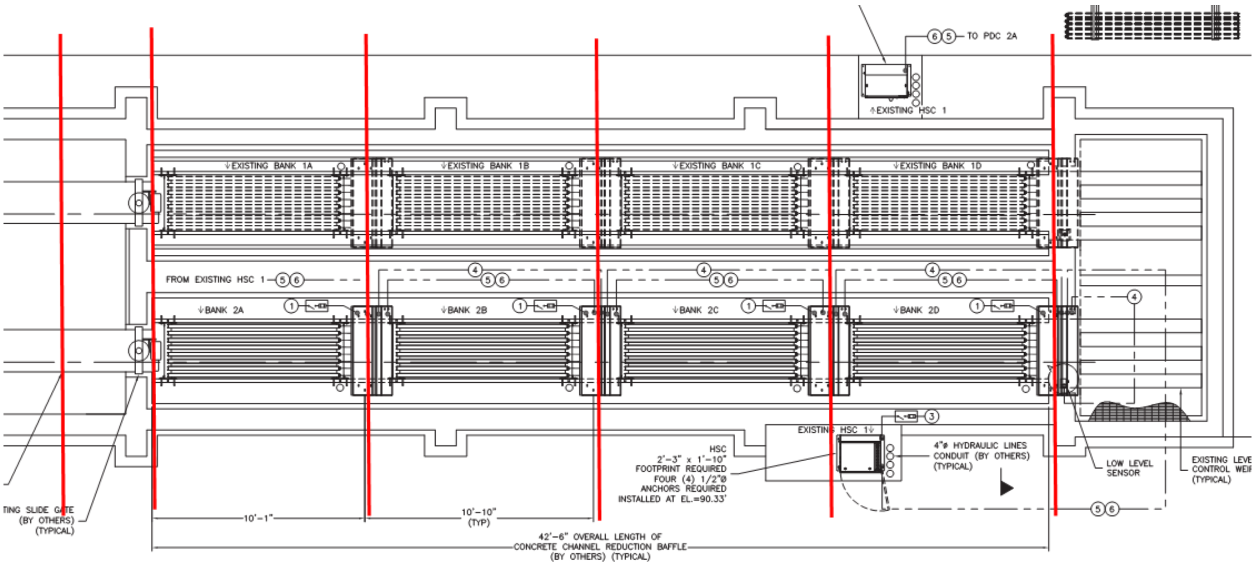


Figure 14-9 Potential Cross-Section Locations

Once the data is collected, it would be evaluated for point and average velocities. Cross-section and longitudinal profiles would be developed for each channel. Any issues associated with poor mixing and/or preferential paths (among other potential issues) would be addressed prior to any bioassay tests being conducted.

Stantec proposes to evaluate velocity profiles with one and two channels online. This would allow verification of hydraulics through each channel and how flow partitions between channels occur.

14.2.1.2 Headloss Across the Channel

Stantec proposes to monitor headloss across the channel by measuring water levels before and after each bank. This would be done during the hydraulic evaluation and bioassay testing. Stantec recommends this be done for different flows to determine the hydraulic capacity of the UV channels based on the distance from the water surface to the lamps.

14.2.2 UV Disinfection Performance (Bioassay Testing)

The intent of the field performance evaluation is to confirm that the system is performing as expected, based on validation testing performed by Trojan. In addition, Stantec recommends verification of system performance under current and future operating conditions, namely different flow rates and UVT values. As summarized in Section 14.1.3 the UV disinfection system must be able to deliver a minimum hourly average UV dose of 80 mJ/cm^2 until December 30, 2022 and a dose of 100 mJ/cm^2, effective December 30, 2022.

On-site testing performed by Moreland (refer to Section 14.1.4) suggests that the system might be underperforming at higher flow rates (i.e. 4.2 MGD). The Moreland report also highlights some potential issues associated with hydraulic turbulence in the channel. Stantec proposes verifying these results and also testing conditions that are relevant to future operation conditions, namely a peak flow of 4.2 MGD (based on the current effluent pump capacity), and UVT values currently measured on site. Although the design UVT for the system is 65%, the majority of the data measured (as per Section 14.1.5) is lower than this value. From September 2016-October 2019, the average daily UVT was measured as low as 54.1% and the minimum daily UVT was measured as low as 51.5%. Table 14.3 summarizes the proposed test matrix. The different tests include UVT values ranging between 55% and 65%, and flows ranging between 1.0 and 4.2 MGD. Stantec also proposes to test both channels, rather than just one, if the hydraulic evaluation points towards difference in flow splitting profiles.

The test matrix assumes that flow and UVT values can be varied and measured within the desired range. The feasibility of increasing flows up to 4.2 MGD will require field verification testing. As indicated in this section, the UV disinfection system was sized to deliver 100 mJ/cm^2 at a design flow of 4.8 MGD and 65% UVT. For bioassay testing, Stantec proposes to use SuperHume™ as a UV absorber, and MS-2 bacteriophage as the microbial surrogate.

Table 14-3 Proposed Performance Evaluation Test Matrix

Test No	No. Channels	No. Banks	UVT[1] (%)	BPL[2] (%)	Flow[1] (MGD)	Pred. RED (mJ/cm^2)
1	1	2	55	100	4.2	30.5
2	1	1	55	100	2.5	27.1
3	1	1	55	100	1	52.8
4	1	1	60	100	4.2	21.1
5	1	1	60	100	2.5	27.1
6	1	1	60	70	1	46.0
7[3]	1	1	65	100	4.2	28.3
8[3]	1	1	65	100	4.2	28.3
9	1	1	65	70	1	61.9
10	0	0	55	0	1	0

[1] UVTs and flows to be tested may change depending on the results of the hydraulic evaluation
[2] BPL – Ballast Power Level
[3] Different bank location to be used during testing

14.2.2.1 Parameters to be monitored

To assess the accuracy of the test results, several water quality and system parameters are recommended to be monitored throughout the system performance assessment. The water quality parameters include water temperature, turbidity, UV absorbance/transmittance, and free chlorine residual. The system parameters to be monitored include channel water levels, UV intensity sensors (duty and reference), power input to the lamps, and electrical supply voltage.

14.2.2.2 Hydraulic Residence Time Mixing Requirements

The number of HRTs required between injection and sampling is recommended to be tested on site. It is generally accepted to use five HRTs as the default waiting period between injection and sample collection. However, Stantec suggests confirming the adequacy of this value, and whether a shorter time is warranted. As an example, Trojan used four HRTs when validating this system.

It is proposed to use SuperHume™ addition to verify proper mixing conditions. The amount of SuperHume™ added would be determined the day of testing, based on treated effluent UVT. Proper mixing would be determined through measurement of UVT before and after addition of a UVT absorber. Sample collection locations would be identified on site during testing preparation.

14.3 SUMMARY AND RECOMMENDED NEXT STEPS

The Discovery Bay WWTP contains two UV channels containing Trojan UV3000Plus™ equipment that was designed to each deliver a 100 mJ/cm^2 UV dose at a flow of 4.8 MGD and a UVT of 65%.

The system was validated by Trojan and a spot-check bioassay was performed in 2017 by Moreland. The 2017 Moreland report concluded that four of eight tests performed equally or better than predicted. However, when Stantec performed the calculations using the Trojan 2012 Addendum RED prediction equation and Trojan provided factor values, six of eight tests performed equally or better than predicted.

As summarized in Section 14.1.5 and 14.1.5.1, a considerable percentage of UVT values measured from September 2016 to October 2019 were lower than the assumed design UVT of 65%. As additional UV disinfection capacity is required when UVT drops, there are a number of conditions under which two channels must operate to deliver the required dose. For most normal flows and UVT conditions, one channel with four banks online is sufficient to deliver the required UV dose. Two channels will likely be needed during periods of wet weather flows and for periods of low UVT. The existing system controls will have to be modified to allow for dual channel operation. For conditions where the 100 mJ/cm^2 is not met with six banks online, flow must be diverted to the sludge lagoons upstream of the tertiary filters. Currently, the effluent pump has a capacity of 4.2 MGD. Based on the Trojan RED prediction equation, at a flow of 4.2 MGD, one channel with four banks online could deliver the required UV dose for UVTs above 62.9% and two channels with three banks online per channel could deliver the required UV dose for UVTs above 56.4%.

To verify the UV system's performance, Stantec recommends evaluating the following:

1. The hydraulic capacity of the channels,
2. Velocity profiles, including proper flow splitting between the two channels, and
3. The delivered UV dose at different flows and at different UVTs.

The total cost for the UV system testing and possible improvements is estimated to be $200,000. This includes $110,000 for hydraulic and system performance testing and an allowance of $90,000 for hydraulic system improvements and control system modifications The additional investigations described above must be completed to confirm recommended improvements.

15.0 EFFLUENT PUMP STATION, PIPELINE, AND OUTFALL DIFFUSER

As developed in Section 7, the existing Export Pump Station, together with the export pipeline and the outfall diffuser (in its original design condition), has a reliable capacity of about 4.2 Mgal/d and does not require expansion. However, the existing outfall diffuser has been compromised, resulting in decreased capacity for the combined export facilities. Therefore, the outfall diffuser must be restored as discussed in the remainder of this section.

15.1 OUTFALL DIFFUSER BACKGROUND, CONDITION ASSESSMENT, AND UPGRADE/REPAIR OPTIONS

On June 8, 2019 WorleyParsons Group Inc. (WP) submitted a report on the condition of the existing outfall diffuser. The following is a summary of their assessment and options for upgrading and repair of the diffuser.

15.1.1 Background Information

The project sanitary outfall is in eastern Contra Costa County, California about 60 miles from San Francisco, in a section of the Old River flanked by earthen levees. The site is located adjacent to the west levee (left riverbank) and south of the Contra Costa Water District (CCWD) Los Vaqueros Pump Station. Based on the Kleinfelder Inc. geotechnical report (2004), the Old River at the site location has the following tidal water level fluctuations and information:

- 100-year Flood Elevation – 7.5 feet (ft.)
- Mean High Water Elevation – 2.4 ft.
- Mean Higher High-Water Elevation – 3.5 ft.
- Mean Lower Low Water Elevation – -0.05 ft.
- Extreme Low Water Elevation – -2.0 ft.
- Flow velocity – 3 to 4 ft./s

The outfall diffuser consists of the following:

- Total outfall length 228.5 ft. (actual pipe length from the levee connection point);
- HDPE Pipe Diffuser length 123 ft. including concentric reducer length;
- Outfall diameter 18 inches (in.), 10 in., and 6 in.;

▪ Number of diffuser ports	36;
▪ Port spacing	average of 3 ft. between ports; and
▪ Port diameter	2 in. Series 35 Longneck Tideflex Valve

15.1.2 Condition Assessment

On May 15, 2013, Bishop Diving & Salvage (BDS) completed an underwater visual inspection of the outfall and observed that 2 out of the 36 diffuser ports were missing and no flow (except for one port) was observed in the 6-inch pipeline segment. On December 2, 2017 a second inspection by BDS showed similar outcomes with 2 out the 36 ports missing and no flow observed in the 6-inch pipeline segment. Also, some of the Tideflex valves appeared to have cracks and may not be sealing properly. On December 7, 2017 a CCTV camera inspection of the outfall, completed by Subtronic Corporation, discovered a blockage in the 10-inch pipeline segment and was not able to proceed further into the pipe. It is assumed that beyond this point the pipeline is either fully or partially obstructed with sediment and organics resulting in reduced flow capacity.

15.1.3 Outfall Upgrade/Repair Options

Four upgrade/repair options were proposed by WP. These options include removal/replacement or abandoning/replacement of the existing HDPE sections of the diffuser. These options also include using the existing diffuser concept of 36 ports or using a new design of 3-5 ports. WP noted that all options would include some level of disturbance of the site during the implementation of the repairs/upgrades and may trigger a permit review by the Regional Water Quality Control Board (RWQCB) and several other regulatory agencies.

The following is a brief summary description on the options presented in the WP report:

Option1: Remove the existing HDPE diffuser (123') and replace with a new, similar, 36-port HDPE diffuser. The new diffuser would be placed in placed in an excavated trench approximately 2.5 feet below the existing riverbed. This option is basically a maintenance project; the Regional Water Board permit should not need updating. Work in the river would require environmental permits. These permits may be complicated if State/Federal agencies continue to believe endangered species may be impacted by construction. This issue is common to all options.

Option 2: Remove the existing HDPE diffuser (123') and replace with an 18" (no reductions) diffuser with only 3 to 5 discharge ports in an excavated trench. This option would require a new dispersion model with field verification. With 3 to 5 ports rather than 36, it is expected that the acute and chronic mixing zones would need to be longer than 5 feet to achieve the same 13.2:1 and 23:1 dilution credits, respectively.

Option3: Abandon the existing diffuser in place (and remove the existing ports and valves) and replace it with a new diffuser similar to Option 2, except that the diffuser would be installed flush to the river bed rather than buried a few feet in a trench below the river bed (per Options 1 and 2).. Option 3 requires a new dispersion model with field verification like Option 2.

Option 4: Remove the existing 10" and 6" diffuser segments (about 47' long) and replace them with 18" HDPE with the ports sized and spaced per the original design and Option 1. This is like Option 1, but with reduced disturbance of the river because only a portion (47') of the diffuser (123") would be replaced.

Common Upgrades to All Options

WP recommends installation of an articulated concrete block matt (ACBM) over the "header" to "prevent scour in the region of the diffuser" for all options. To prevent damage WP also recommends providing a metal cage over the diffusers to prevent damage from boating activities (e.g. vessel anchors). Environmental analysis is required to address the impact of the ACBM and metal cage.

All options may include a flush system for periodic cleaning of the diffuser with either a return line for disposal of flushed material onshore or with direct discharge into the river (no return line). The flush system would involve the installation of ball valves equipped with pneumatic actuators at each of the diffuser ports, and an airline to activate them. Also, the system would include a downstream discharge ball valve (6 inches) equipped with a pneumatic actuator (with separate airline for activation) and an alternative 6-inch return line for discharge onshore. A portable air compressor would be connected to the air manifold (installed on shore) to supply air and activate the various valves for periodic maintenance cleaning. Actuators for the valves would be specified as normally open (NO) for the diffuser port valves and normally closed (NC) for the flush valve in absence of pressurized air.

Cost Estimate

The following magnitudes and relative differences between the estimated costs for each option were presented in the WP Report.

Option	Construction Cost ($)	Engineering & Inspection Cost ($)	Capital Cost ($)
1	298,500	164,000	463,000
2	296,900	164,000	461,000
3	334,000	164,000	498,000
4	183,000	164,000	347,000

The estimate of engineering and inspection cost is the same ($164,000) regardless of option.

15.2 CONCLUSIONS AND RECOMMENDATIONS

Option 4 has the apparent lowest cost at $347,000, not including environmental analysis, permitting, and administration. With these items included, a total budget allowance of $500,000 is recommended. Replacing just the 10” and 6” segments (47’) would result in less river disturbance and appears to provide a larger zone of passage for aquatic life around the excavation disturbance area during construction. It is recommended that the Town get an opinion from a qualified CEQA/NEPA consultant with extensive experience in Delta waterways as to the relative environmental complexity and risk associated with repairing the diffuser.

16.0 EMERGENCY STORAGE RETURN PUMPING

The Plant 1 site includes an earthen emergency storage basin with a volume of approximate 5 million gallons. During an emergency when Plant 1 and/or Plant 2 may not be able to handle the entire influent flow, a portion or all the influent flow can be diverted to the emergency storage basin for temporary holding until such time as the stored volume can be treated. At the present time, however, the only way to return stored wastewater is to use portable pumping equipment.

As part of the Influent Pump Station and Pump Station W Improvements project designed in 2012, a 12-inch drainpipe from the emergency storage basin to Pump Station W was designed but then eliminated from the final construction project to save money. This pipeline is still a desirable feature and should be added when adequate budget is available. The estimated construction cost for this pipeline is $50,000. With contingencies, engineering, and administration, the total budgetary cost is $75,000.

17.0 EFFLUENT DISPOSAL ALTERNATIVES

The purpose of this section is to evaluate the possibility of disposal or reuse of the District's wastewater effluent on land based on the assumption that treatment requirements and resultant costs for treatment plant improvements may be less onerous than they are for continued discharge to Old River.

17.1 OVERVIEW OF EFFLUENT DISPOSAL OPTIONS

The Town of Discovery Bay Community Service District (TDBCSD) currently discharges its effluent almost entirely to Old River, a tidal tributary of the San Joaquin River. A minor amount of effluent is reused within the treatment plant. The current NPDES permit Order No. R5-2014-0073-01 sets average monthly effluent concentrations of Nitrate+Nitrite at 10 mg/L as N and Ammonia at 0.7 mg/L as N, which are to take effect on December 31, 2023. Until December 31, 2023, interim maximum daily effluent concentrations for Nitrate+Nitrite and Ammonia are 31 mg/L as N and 8.4 mg/L as N, respectively. An updated permit is currently being reviewed and is expected to be adopted in December 2019. The anticipated average monthly and average weekly limits for Nitrate+Nitrite are 10 and 17 mg/L as N, respectively, and the anticipated average monthly and weekly limits for ammonia are 0.7 and 1.4 mg/L as N, respectively. These limits are expected to take effect on December 31, 2023. Until then, interim maximum daily effluent limits for Nitrate+Nitrite and Ammonia are expected to be 39 mg/L and 8.4 mg/L, respectively.

The TDBCSD WWTP currently produces effluent with Nitrate+Nitrite concentrations of about 30 mg/L or less. At future design flow and loading conditions and assuming no improvements to the secondary treatment process, it is expected that Nitrate+Nitrite concentrations in the final effluent would be in the range of 30 to 40 mg/L as N, while Ammonia would likely be below the future permit limit of 0.7 mg/L (monthly average). As discussed in Section 11, significant improvements to the secondary treatment system are needed to meet the future surface water discharge requirements for Nitrate+Nitrite and Ammonia. Unfortunately, the improvements and operations needed to remove Nitrate+Nitrite will actually make it more difficult to meet the Ammonia limit of 0.7 mg/L (but still possible with careful design and operation).

If 100 percent of the wastewater effluent were to be reclaimed for crop or landscape irrigation (no discharge to Old River - this would require winter storage of effluent), it is possible that the need to remove Nitrate+Nitrite and Ammonia could be eliminated, thereby saving approximately $8 million in costs for secondary process improvements. However, the required effluent storage reservoir(s) would have to be sealed to prevent percolation and the reuse operation would have to be controlled so that nitrogen is applied at agronomic rates to avoid nitrate pollution of groundwater.

Separately from this Master Plan, the District completed an independent investigation of storage and 100% effluent reuse for crop irrigation. That analysis showed the total cost for effluent storage and irrigation to be approximately $17 million. Since this is much higher than the cost of secondary process improvements for river discharge, the alternative was eliminated from further consideration.

Another potential alternative for keeping the wastewater effluent out of Old River and possibly obtaining less stringent discharge requirements is to dispose of the effluent by percolation into groundwater. This alternative is considered in the following subsection.

17.2 EFFLUENT DISPOSAL VIA PERCOLATION

Effluent disposal via percolation is evaluated below based on anticipated discharge requirements and based on suitability of soil for percolation. Additionally, issues associated with the Delta Protection Act must be considered when evaluating potential disposal sites. Specifically, locating percolation ponds within the Sacramento San Joaquin Delta Primary Zone may be prohibited or may result in more stringent requirements for effluent discharge. For that reason, the future percolation ponds should not be located within the Delta Primary Zone. A map of the Sacramento San Joaquin Delta showing the boundaries of the Primary and Secondary Zones is provided on Figure 17-1 (courtesy of Water Education Foundation).

17.2.1 Effluent Discharge Requirements for Percolation Disposal

Typically, a permit for effluent discharge via percolation would include effluent discharge requirements and groundwater monitoring requirements to ensure that no groundwater degradation is caused by the percolation ponds. Based on experience with similar facilities, the TDBCSD WWTP effluent would likely have to meet requirements for Title 22 disinfected secondary -23 recycled water. The effluent monitoring requirements may include BOD, TSS, and conductivity. Some additional constituents may also be included, which may be specific to Discovery Bay WWTP. In addition to effluent requirements, the permit would likely include groundwater requirements. Typically, permits for percolation discharge include a groundwater Nitrate limit of 10 mg/L and a Total Coliform limit not to exceed 2.2 MPN/100 mL. It should be noted, however, these requirements are approximate and the actual requirements for TDBCSD may be different. For example, proximity of surface waters may require percolated effluent standards to match those of effluent discharged into the surface water.

Based on the foregoing and considering that some level of denitrification may be achieved in the percolation ponds (if properly operated), it is reasonable to assume that effluent requirements for Ammonia, Nitrate and Nitrite would be less stringent than those for surface discharge, but more stringent than for effluent discharge to irrigation fields (where plants would uptake excess nitrogen from the effluent). Therefore, some improvements to the secondary treatment process would likely be required. Currently, the specific permit requirements for percolation disposal and the nature and cost of the secondary process improvements needed to comply with those requirements are unknown.

17.2.2 Soil Suitability for Percolation

The potential viability of effluent disposal by percolation depends on whether adequate lands with suitable soil characteristics can be identified within reasonable proximity to the wastewater treatment plant. For this analysis, all non-urban and flat land within 5-miles of the treatment facility was considered. The evaluation included soil review with respect to permeability, depth to groundwater, depth to an impermeable layer, and surface slope. Soil data was obtained from the USDA web soil survey database which can be found at: https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx.

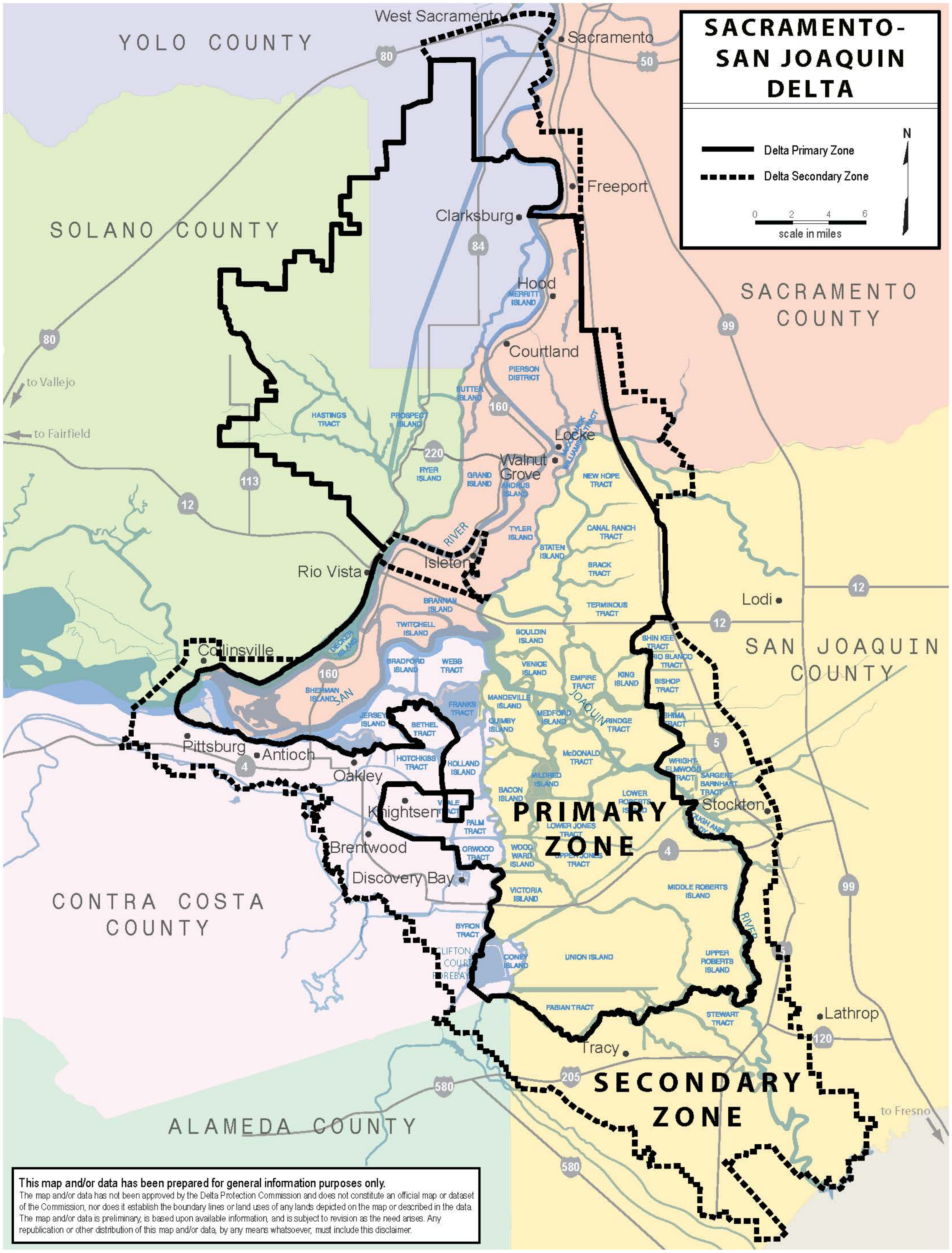


Figure 17-1 Sacramento San Joaquin Delta Zones

Area suitability for effluent discharge using percolation ponds was assessed based on the following criteria:

- Soil Permeability: Ideally, the minimum soil permeability should be 0.2 inch/hr or higher. Lower soil permeability can also be evaluated; however, percolation ponds constructed on low permeability soils are usually not cost effective, especially in areas where property values are high.
- Depth to Groundwater: A minimum of 3 ft of unsaturated soil should exist between the bottom of the percolation basins and groundwater to maintain some natural attenuation of pollutants and to maintain reasonable percolation rates.
- Depth to an Impermeable Layer: There must be a sufficient depth of permeable soils below the percolation basins to allow the effluent to flow horizontally away from the basins without surfacing. The minimum required depth to an impermeable layer depends on the horizontal permeability of the soil above that layer.
- Surface Slope: Ideally, surface slopes should be in the range of 0 to 2% to allow cost-effective construction of percolation basins. Steeper slopes can be considered but are usually not cost-effective.

Considering the criteria described above, areas evaluated were grouped into three categories as shown on Figure 17-2 and described below:

1. Areas potentially suitable for percolation ponds (shown in green) include areas with soils that meet all the criteria below:
    a. Permeability – high (>0.2 inch/hr)
    b. Depth to groundwater – minimum of 60 inches
    c. Depth to an impermeable layer – minimum of 80 inches
    d. Surface slope – 2% or less
2. Areas likely unsuitable for percolation ponds (shown in yellow) include areas that cannot be classified as potentially suitable and that have soils that meet all the criteria below:
    a. Permeability – permeability from low to high (0.06 to 0.2 inch/hr) or high (>0.2 inch/hr)
    b. Depth to groundwater – minimum 36 inches
    c. Depth to an impermeable layer – minimum 36 inches
    d. Surface slope – less than 2%
3. Areas unsuitable for percolation ponds (shown in red) include areas with soils that meet one or more of the following criteria:
    a. Permeability – less than 0.06 inch/hr
    b. Depth to groundwater – less than 36 inches
    c. Depth to an impermeable layer – less than 36 inches
    d. Surface slope – higher than 2%

As shown on Figure 17-2, most of the area within a 5-mile radius of the wastewater treatment plant is either unsuitable (red) or likely unsuitable (yellow). Areas closest to the plant are unsuitable. Areas east from the treatment plant are within the Sacramento San Joaquin Delta Primary Zone and are considered unsuitable for percolation disposal, regardless of soil conditions.

Areas considered to be potentially suitable for effluent disposal via percolation ponds (green) are located west from the Town of Discovery Bay, with some areas in immediate vicinity of the City of Brentwood residential areas. Considering distance from the treatment facility and proximity to residential areas, the area with the apparent highest probability of success is approximately 4 miles west of Plant 2.

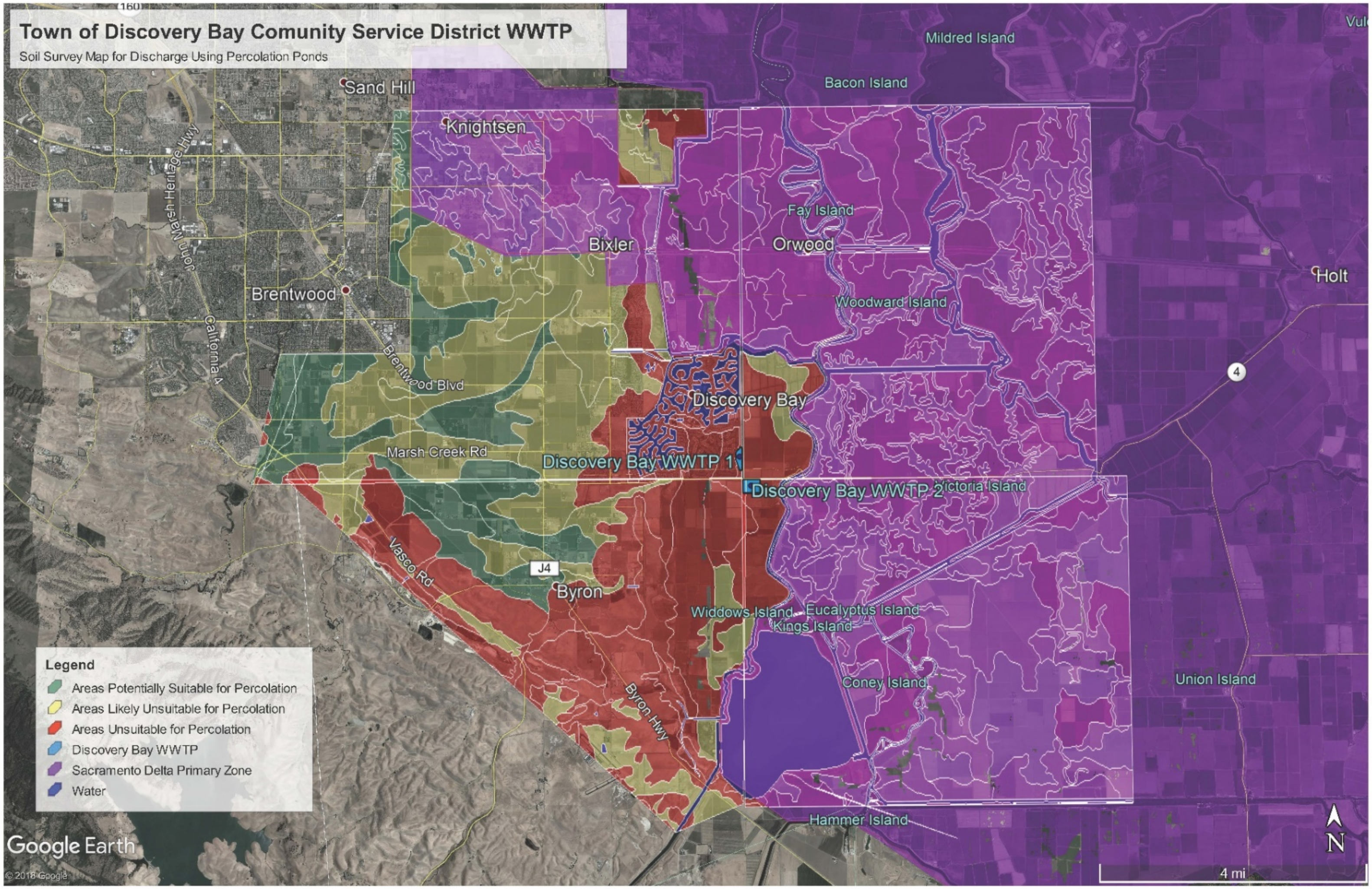


Figure 17-2 Soil Survey Map

17.2.3 Conceptual Cost Estimate for Percolation Disposal

An area considered to have a relatively high potential for percolation disposal is identified in Figure 17-3 and is the basis for development of a conceptual cost estimate. The soil at the site consists of Brentwood clay loam, which has limiting permeabilities ranging from 0.2 inch/hour to 0.57 inch/hour. The depth to groundwater and the depth to an impermeable layer are both more than 80 inches. The area is flat with slopes between 0 and 2 percent. Based on preliminary analysis, the minimum active area required to percolate 1.63 Mgal/day is approximately 36 acres. The area was calculated assuming a pond infiltration rate of 1.7 inch/day, which is approximately 1/3 of the minimum soil permeability indicated in the soils survey (0.2 in/hr = 4.8 in/d). The 1/3 factor is considered a minimum design safety allowance. Based on hydrogeologic studies that would be required before actual design, an even lower rate could be required to allow for horizontal movement of groundwater away from the percolation basins. For comparison, infiltration rates observed for City of Brentwood percolation ponds are between 1.1 to 2.2 inch/day.

The minimum active pond area indicated above is based on water surface area and does not include surrounding berms and buffer areas. The total required area should be divided into multiple percolation ponds (at least three) for flexibility of operation and maintenance, including yearly removal from service for resting, drying, and tilling. The minimum active area requirement would have to be met with the largest pond out of service.

In addition to the percolation ponds, the District would have to construct a 4.2 mi long effluent pipeline and new effluent pump station. Based on a preliminary analysis, the total capital cost to transition to effluent disposal via percolation would be approximately $14.3 million, as shown in Table 17-1.

Table 17-1 Conceptual Cost Estimate for Percolation Disposal

Item	Cost, $ (a)
Effluent Pump Station Improvements	*$500,000*
Effluent Pipeline, 4.2 miles @ $160/lf	*$3,520,000*
Mass Grading for Percolation Ponds	*$2,000,000*
Rip Rap Side Slopes @ $50/syd	*$278,000*
AB Perimeter Road, 20 ft wide @ $70/cyd	*$260,000*
New Property Acquisition, 60 ac @ $140k/ac	*$840,000*
Miscellaneous Pipelines and Structures at Perc. Ponds	*$800,000*
Subtotal	***$8,198,000***
Gen. Cond., Overhead, Profit @ 20%	1,640,000
Contingency @ 25%	2,050,000
Total Construction Cost	***$11,888,000***
Permitting, Engineering, Construction Management and Admin	$2,378,000
Total Capital Cost	**$14,266,000**

(a) Cost estimate is based on 20-Cities ENR of 11,500.

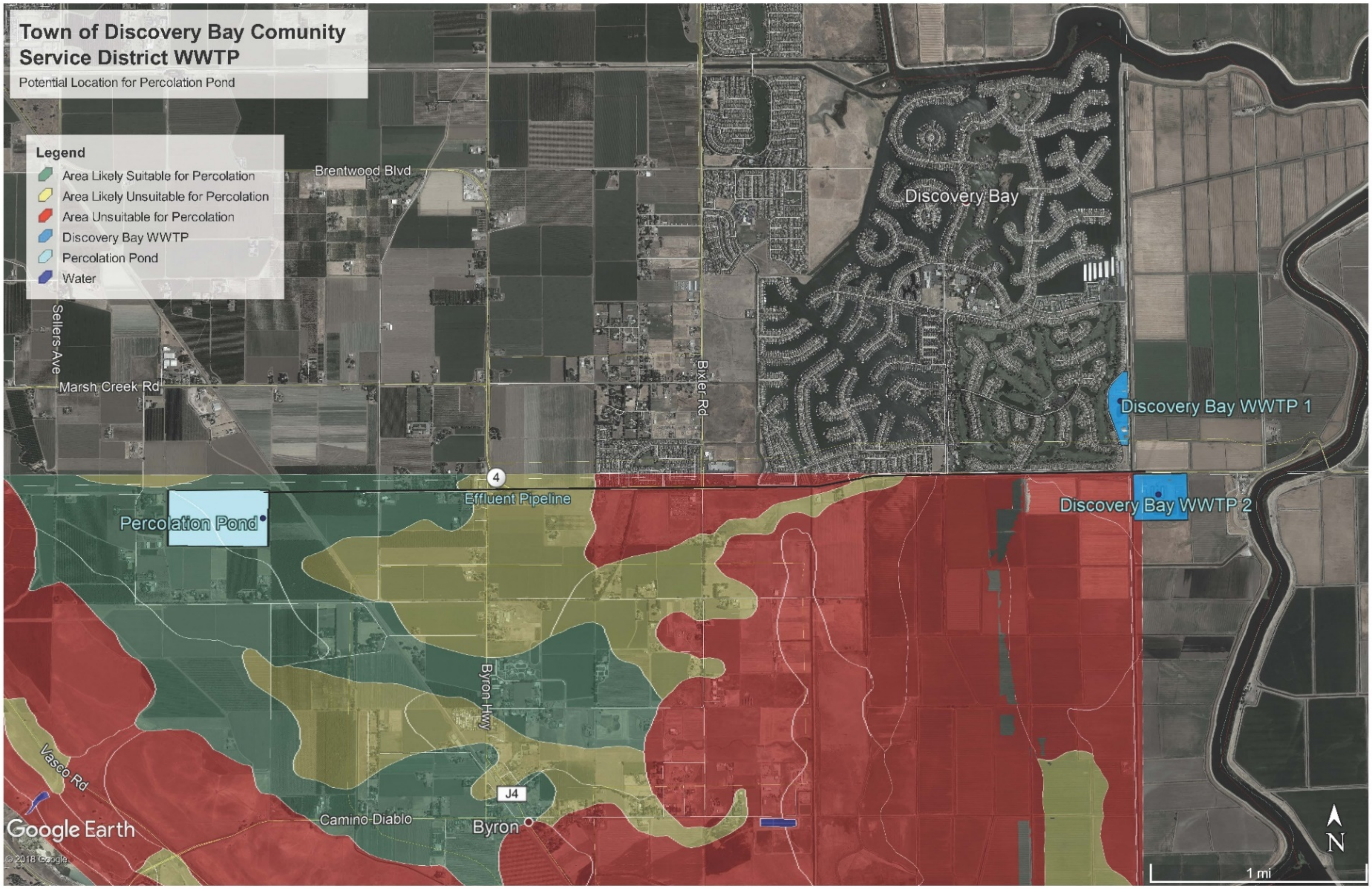


Figure 17-3 Possible Location for Percolation Ponds

17.2.4 EBMUD Site Evaluation

In addition to the evaluation presented above, two parcels owned by East Bay Municipal Utility District (EBMUD) were evaluated for the potential to be used for effluent percolation ponds. The parcels are located approximately 3.5 mi North-West from Plant 2 as shown on Figure 17-4. Most of the soil at the two parcels is classified as Capay clay and is likely unsuitable for percolation ponds. Depth to groundwater is 3 to 6 feet, soil permeability ranges from moderately low to moderately high at 0.06 to 0.2 in/hr, and depth to an impermeable layer is more than 80 inches. The remainder of the area is unsuitable for percolation disposal due to very low percolation rates. In addition to poor suitability, the two EBMUD parcels are located within the Sacramento San Joaquin Delta Primary Zone, which is not recommended for percolation disposal as previously mentioned.

For the reasons identified above, the two EBMUD parcels are not recommended for further investigation.

17.2.5 Conclusion

Based on the apparent high cost of implementing percolation disposal, this alternative does not appear to be feasible, even if no secondary treatment improvements would be needed. Furthermore, it is likely that substantial secondary treatment improvements would be required, but perhaps not as much as for river discharge (about $8 million).

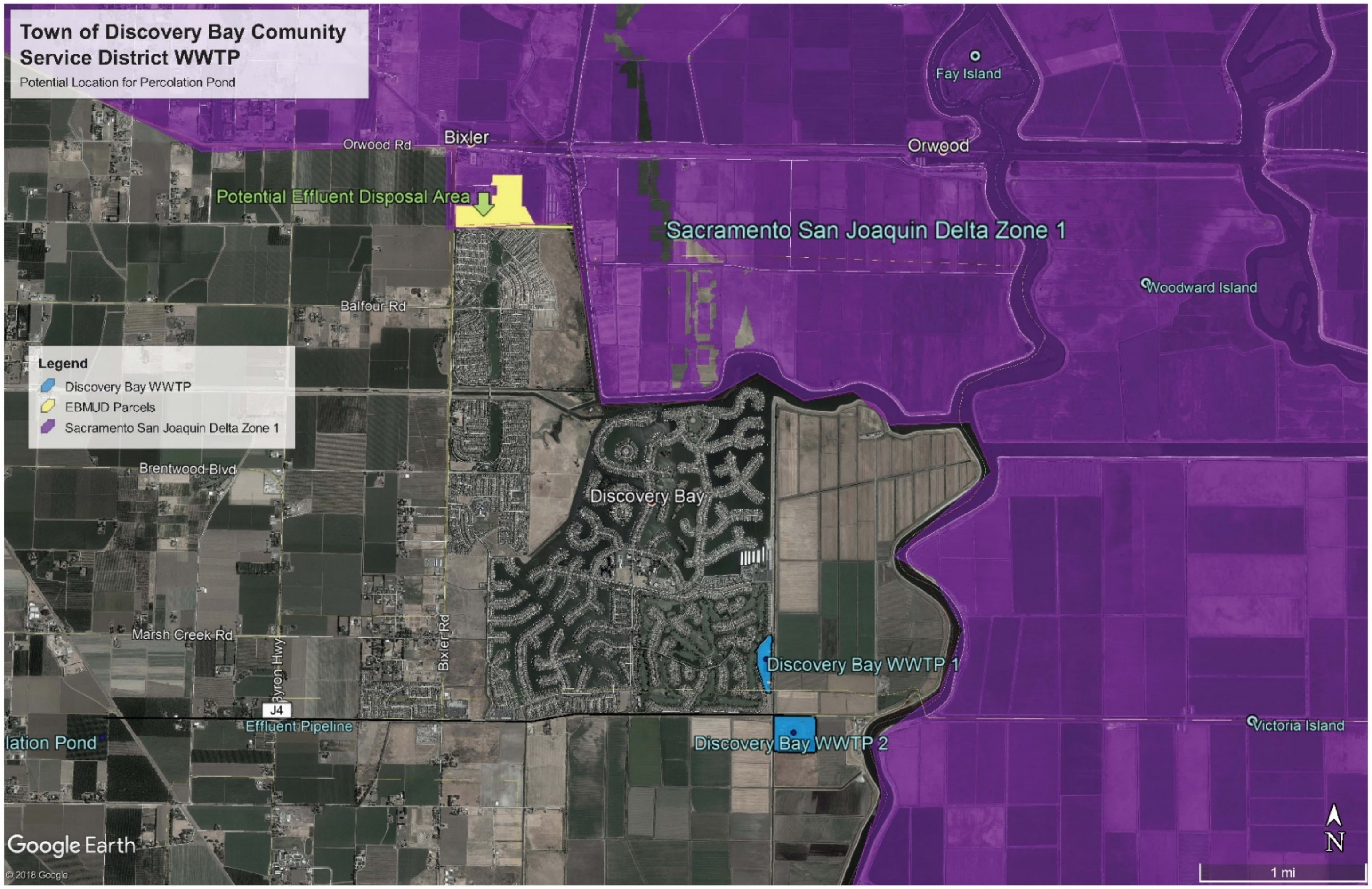


Figure 17-4 East Bay MUD Parcels – Potential Effluent Disposal Sites

18.0 SOLIDS HANDLING

All of the solids handling facilities for both Plant No. 1 and No. 2 are located at Plant No. 2. In this section, the existing facilities are described, capacities are evaluated, and recommended improvements are discussed. Additionally, biosolids disposal alternatives are evaluated.

18.1 DESCRIPTION OF EXISTING FACILITIES AND OPERATIONS

The solids handling facilities include waste activated sludge (WAS) pumping systems at each plant, a small aerobic digester (0.69 million gallons), two sludge lagoons (5.75 million gallons each), three belt presses, and four active solar sludge dryers. Solids from the secondary process at each plant are pumped as WAS to Plant No. 2 for processing. Normally, the WAS is pumped into the aerobic digester to get some volatile solids reduction and to allow some thickening (by decanting) and then is pumped to the belt presses where it is dewatered and then loaded into the active solar dryers with a self-unloading truck. The active solar dryers dry the sludge to 75% to 80% solids to reduce volume and kill pathogens. The sludge is then stockpiled on-site and then, once per year, hauled to a landfill for disposal.

Until several years ago, the existing sludge lagoons were used to store solids prior to dewatering and further handling. Due to inadequate capacity of belt presses and solar dryers at the time, a large volume of sludge was accumulated in the lagoons. Pursuant to the previous Master Plan, additional belt presses and solar dryers were added. Currently, under normal operating conditions, no new solids are being added to the lagoons; instead, stored solids are gradually being dredged out of the lagoons and combined with WAS in the aerobic digester for subsequent dewatering, drying, and export from the plant site. The capacities of the existing belt presses and solar dryers to handle the future design sludge production during each month of the year are evaluated later in this section.

The sludge lagoons, in addition to being capable of storing solids prior to dewatering if desired, are also used for other purposes as described elsewhere in this Master Plan. For example, secondary effluent flows in excess of 4.0 Mgal/d are diverted to the lagoons for temporary storage as a means of limiting the flow to the downstream filters and UV disinfection system. Additionally, poor quality effluent can be temporarily diverted to the lagoons to avoid discharge.

According to the District Engineer, the final sludge product out of the active solar dryers does not quite meet Class A Exceptional Quality limits under EPA 503 regulations. However, Class A is easily attained after stockpiling the dried solids on site for at least 30 days after removal from the active solar dryers. Historically, this allowed the District to apply the dried sludge on agricultural property immediately south of Plant No. 2. However, this method of disposal was discontinued because no crop was being grown to take up the nitrogen in the sludge and the Regional Board was concerned with nitrate pollution of groundwater and required the installation of a monitoring well. Rather than implementing a farming operation to take up nitrogen and constructing a monitoring well, the District switched to landfill disposal. Other disposal alternatives are investigated later in this section because landfill disposal of sludge is being phased out by the State of California.

The existing aerobic digester is not large enough for meeting EPA 503 Class B Criteria for pathogen reduction, but this is not of concern, since Class A sludge is produced after solar drying. The main functions of the aerobic digester are to provide some volatile solids reduction and to provide a homogenous feed source for the belt presses. The volatile solids reduction reduces sludge export quantities and helps prevent odors in the solar dryers. There is a decant system in place in the aerobic digester that allows some thickening of the sludge prior to being sent to dewatering. Sludge in the digester is approximately 1% solids prior to dewatering. There is also an overflow from the aerobic digester to the sludge lagoons. The aerobic digester is aerated and mixed with four 25 horsepower aerators.

The dewatering system consists of three 1.5 meter mono-belt belt presses and ancillary facilities. Dewatered sludge cake is normally 12% to 16% solids and is transferred by auger directly into a self-unloading truck. The maximum capacity of each of the existing dewatering presses is 100 gpm or 900 dry lbs per hour, whichever is most limiting. Based on the normal 1% solids of the aerobic digester feed source, the throughput of each press is limited to 100 gpm, which results in a solids loading rate of approximately 500 dry lbs per hour.

The active solar dryers consist of four chambers, each 40 feet wide by 204 feet long. Each dryer holds about 190 wet tons of sludge at the beginning of each drying cycle. Sludge is loaded into the dryers with the self-unloading truck. A mechanical mole turns the sludge inside the dryers while the sludge is drying. The drying time (after the chamber is fully loaded) is cyclical with the seasons, ranging from about 2 weeks in the hottest part of the summer to 6 or 8 weeks in the coldest part of the winter.

The District has a floating dredge that can be moved to either of the two sludge lagoons and is used to pump sludge to the aerobic digester. However, the dredge is old and obsolete and should be replaced as repair parts are not available.

18.2 CAPACITY EVALUATION FOR SOLIDS HANDLING FACILITIES

The capacities of the various portions of the solids handling system vary throughout the year. Cold temperatures in the winter months result in higher sludge yields from the secondary treatment system and, therefore, higher loadings to aerobic digester. This impact is compounded by slowed aerobic digestion, leading to higher solids loading to the belt presses and active solar dryers. As mentioned previously, the required drying time in the active solar dryers is much higher in the winter than in the summer.

To assess solids loadings and solids handling capacity throughout the year, solids balance calculations for the entire wastewater treatment plant were developed on a month-by-month basis, assuming average influent loadings at all times. Temperatures in the secondary treatment system and in the aerobic digester and drying times for the active solar dryers were assumed to vary with monthly average ambient air temperatures as shown in Table 18-1. The total mean cell residence time (MCRT) in the secondary treatment system was assumed to be 19 days and the sludge yield variation with temperature at that MCRT was assumed to be consistent with the sludge yield curves presented in Figure 14.20-b of the Water Environment Federation Manual of Practice No. 8 (MOP8), Fifth Edition. Volatile solids reduction in the aerobic digester for waste activated sludge was assumed to vary with the temperature multiplied by the sludge age in accordance with Figure 6-42 of the EPA Process Design Manual for Sludge Treatment and Disposal (September 1979). However, any solids dredged from the lagoons and added to the aerobic digester were assumed to be fully digested, passing through the aerobic digester without reduction. The average mixed liquor suspended solids concentration in the aerobic digester was assumed to be 10,500 mg/L in accordance with typical operations in the range from 10,000 to 11,000 mg/L.

Table 18-1 Assumed Conditions for Monthly Solids Balances

Month	Avg. Ambient Air Temperature, °F	Secondary Process Temperature, °C	Aerobic Digester Temperature, °C	Drying Time in Solar Dryers (a), days
Jan	49	13.4	10.5	47.7
Feb	53	15.2	12.8	42.5
Mar	56	16.5	14.4	38.6
Apr	61	18.8	17.2	32.1
May	66	21.0	20.0	25.7
Jun	73	24.1	23.9	16.6
Jul	75	25.0	25.0	14.0
Aug	75	25.0	25.0	14.0
Sep	73	24.1	23.9	16.6
Oct	65	20.5	19.4	27.0
Nov	56	16.5	14.4	38.6
Dec	48	13.0	10.0	49.0

(a) Drying time after chamber is fully loaded.

Two sets of solids balance calculations were completed for the future buildout condition (1.63 Mgal/d annual average flow). In the first set of calculations, it was assumed that there were no solids sent to or returned from the sludge lagoons. In the second set of calculations, it was assumed that solids would be dredged from the sludge lagoons and added to the aerobic digester to the maximum extent possible, as limited by the capacities of the belt filter presses and active solar dryers.

For both sets of solids balance calculations, total capacity utilizations for the existing belt presses and for the existing active solar dryers were determined on a month-by-month basis. For the belt filter presses, the total capacity was determined as the capacity of three belt presses operating at 100 gpm for 35 hours per week (total capacity = 630,000 gallons per week). Therefore, for example, if the flow to the belt filter presses averaged 200,000 gallons per week in a given month, then for that month the total belt press capacity utilization expressed as a fraction would be 200,000/630,000 = 0.32 (32%). For the active solar dryers, the ratio of the total cycle time (time to load the dryer plus drying time) divided by the time to load the dryer indicates how many dryers are theoretically required. For example, if it would take 2 weeks to load a dryer and 4 weeks to dry the solids, the total cycle time would be 6 weeks and the theoretical number of dryers required would be 6/2 = 3.0. Since there are four existing dryers, the capacity utilization for the dryers would be 3/4 = 0.75 (75%).

The results of the solids balances with no solids to or from the sludge lagoons are shown in Figure 18-1. As shown in the figure, total belt press utilization ranges from a low of 0.24 in the summer to a high of 0.32 in the winter. If the utilization is calculated as a fraction of the reliable capacity of two belt presses, the range would be 0.36 to 0.48. The latter values are relevant based on the actual practice of using Belt Press 1 only as a backup unit. From these results, it is clear that three belt presses (2 duty + 1 standby) are more than adequate for the future buildout condition when no solids are returned from the sludge lagoons, which should be the normal condition, as it is assumed that the existing solids in the sludge lagoons will be fully removed prior to buildout.

As shown in Figure 18-1, solar dryer utilization (based on four dryers) ranges from a low of 0.35 in the summer to a high of 0.71 in the winter. If the utilization is calculated as a fraction of the reliable capacity of three solar dryers, the range would be 0.47 to 0.95. From these results, it is clear that three belt presses are adequate for the future buildout condition when no solids are returned from the sludge lagoons.

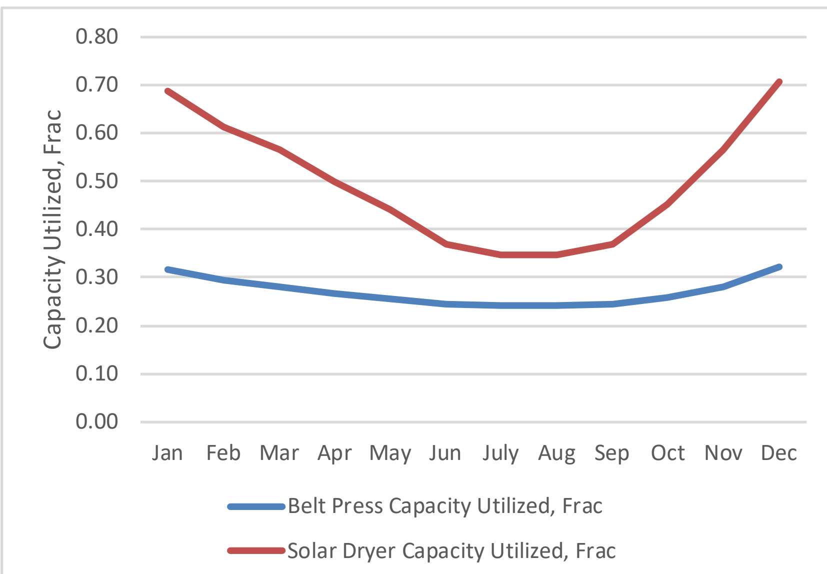


Figure 18-1 Belt Press and Solar Dryer Capacity Utilization for Future Design Condition with No Solids to or from Sludge Lagoons

The results of the solids balances with maximum possible dredging of the sludge lagoons are shown in Figure 18-2. As shown in the figure, the amount of solids that could be removed from the sludge lagoons is limited by the belt presses in the summer and by the active solar dryers in the winter (this is indicated when the units in question are at 100% capacity). The maximum amount of solids that could be removed from the sludge lagoons ranges from about 1300 lb/d in the winter to 5400 lb/d in the summer, giving an annual total removal of about 1.4 million pounds. Assuming the sludge blanket in the lagoon to be at a solids content of 4%, this implies a sludge blanket of about 7 feet deep could be removed in one year. At current plant flows and loads, the ability to remove solids from the lagoon is even greater. Therefore, the existing belt presses and active solar dryers have tremendous capacity to remove solids from the sludge lagoons in addition to keeping up with ongoing digested WAS production.

The evaluation of maximum possible solids removal from the sludge lagoons presented above is based on extremely aggressive operations of the belt presses and active solar dryers, without consideration of the residence time in the aerobic digester. With the solids loadings discussed above, the residence time in the aerobic digester would range from about 8 days in the summer to 14 days in the winter, resulting in about 200 degree-C-days in the summer and 140 degree-C-days in the winter. This would typically not be considered adequate to avoid odors in the active solar dryers if all of the solids were from freshly digested waste activated sludge (400 degree-C-days would be desirable). However, with a high fraction of the solids coming from the lagoons, the odor potential may be mitigated. The maximum amount of solids that could be removed from the lagoons while at the same time handling ongoing flows of digested waste activated sludge would have to be confirmed by actual experience. Nevertheless, it is clear that there would be a very substantial capacity for removing solids from the lagoons.

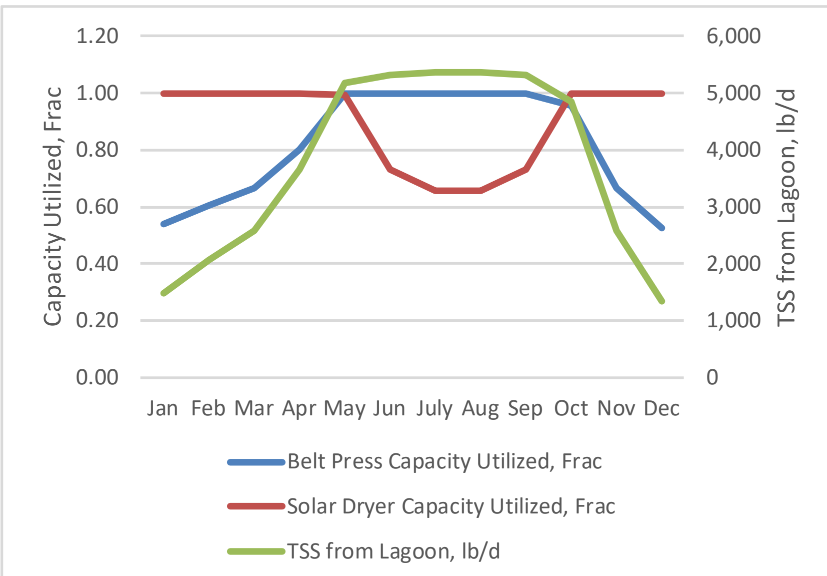


Figure 18-2 Belt Press and Solar Dryer Capacity Utilization for Future Design Condition with Maximum Allowable Solids Removals from the Sludge Lagoons

Based on the foregoing evaluations, no additional belt press or solar dryer units are needed through buildout.

18.3 Belt Press No. 1 Replacement

As developed above, no additional belt presses are required to meet future design conditions. Even though Belt Press 1 is old and needs frequent repairs when used, it is a backup unit that is seldom used. Furthermore, except when removing solids from the sludge lagoons (which should not be necessary in the future), one duty and one standby belt press would be adequate under buildout conditions. Therefore, it does not make sense to replace Belt Press 1. However, to maintain maximum flexibility, it does make sense to continue making repairs to this unit when needed, provided those repairs are at reasonable costs that are far less than the cost of replacing the unit.

18.4 Fate of Sludge Lagoons

As developed above, it is possible that the existing sludge lagoons could be emptied of existing solids in the near-term future by dredging and routing the solids through the existing aerobic digester, belt presses, and active solar dryers. Furthermore, the solids handling system has adequate capacity to process all anticipated future sludge flows without routing any new solids to the sludge lagoons. Nevertheless, these lagoons can continue to be used beneficially and should not be removed from service. Existing and future possible uses of the sludge lagoons include the following:

- Emergency storage of solids in the event of a failure or other removal from service of key solids handling facilities (aerobic digester, belt presses, or active solar dryers).
- Peak flow trimming storage for secondary effluent to limit the flow to the filters and UV disinfection systems.
- Temporary storage of subpar effluent to avoid discharge violations.

18.5 Biosolids Disposal/Reuse

As mentioned above, disposal of wastewater sludge on landfills will be phased out. Senate Bill (SB) 1383 sets the goal to reduce disposal of organics (including wastewater sludge) on landfills. CalRecycle is currently drafting the regulations to support the bill and it is expected that required reductions in organic loads will begin in January 2022 and that 75% reduction will be required by January 2025. Therefore, it is likely that between 2022 and 2025 most of the wastewater treatment plants that dispose their solids at landfills will have to abandon this disposal practice and find a different disposal method.

To replace the current practice of landfill disposal, the following alternative disposal/reuse alternatives are evaluated in this section:

- Land Application of Biosolids on District-Owned Lands
- Contract Hauling and Reuse of Biosolids by Synagro
- Hauling of Biosolids to Lystek for Handling and Reuse

In addition to the above, initial consideration was given to hauling the biosolids to the East Bay Municipal Utilities District (EBMUD) for handling with other solids processed by EBMUD. However, after discussion with EBMUD, it was clear that there was no possibility of a cost-effective operation that would be mutually beneficial.

A possible option not mentioned above is for the District to contract with local farmers for biosolids application on the farmers' properties. Evaluation of this alternative would require contacts with farmers to identify willing participants and to negotiate terms for land application. Such evaluations are beyond the scope of this Master Plan.

New biosolids reuse opportunities are likely to be developed as wastewater agencies discontinue landfill disposal in the next few years in response to SB1383. Therefore, the District should continue to review available options in future years.

18.5.1 Land Application of Biosolids on District-Owned Lands

As mentioned previously, the District has historically land-applied biosolids on land south of Plant 2, but discontinued the practice when State regulators required planting and harvesting of crops to take up the nitrogen in the biosolids and also required monitoring wells to assure that groundwater was not being adversely impacted. Compliance with State requirements to allow re-instatement of biosolids reuse on the land in question is evaluated below.

Based on a nitrogen mass balance, the capacity of the approximate 25-acre property for biosolids disposal is approximately 200 dry tons per year, which is about 57% of the total biosolids expected to be produced at design flow and loading conditions (349 dry tons per year). This estimate is based on planting alfalfa in fall and spring and harvesting 6 to 7 times per year. The estimate also accounts for application of plant effluent for irrigation during dry months of the year, which would contribute approximately 15% to the crop nitrogen uptake. To allow land application of 100% of the biosolids produced under future design conditions on District owned lands, the District would have to purchase additional property for this use – at least 25 acres should be targeted, including buffer areas.

The following steps are required to re-instate biosolids application on the existing property south of Plant 2:

- Due to high background soil nitrogen concentrations, start planting the property a minimum of two years before biosolids are going to be applied. This would enable crop uptake of nitrogen that is already available in the soil before additional biosolids application. It would also provide the opportunity to fine-tune crop management practices in advance of biosolids applications.
- Initiate discussions with the Regional Board regarding biosolids land application and obtain or update any required permits. The existing NPDES permit does not include disposal of biosolids via land application and the District will have to either amend the existing permit to include this or obtain a general permit (General Waste Discharge Requirements for the Discharge of Biosolids to Land for Use as a Soil Amendment in Agricultural, Silvicultural, Horticultural, and Land Reclamation Activities, (General Order)).
- Construct any facilities needed for system operation and monitoring which may include:
    - Groundwater monitoring wells
    - Irrigation system including pipelines, valves, and sprinklers
    - Containment berms and tailwater collection system.

- Hire additional staff and procure needed farm equipment to manage the agricultural operations or find a local farmer that can perform this job on a contract basis. Finding a local farmer has proven difficult in the past due to the relatively small property size. One option may be to contract with local sheep owners to periodically bring their floc for grazing.

If the District acquires additional land for application of biosolids, the actions indicated above would have to be expanded to cover the additional property.

18.5.2 Contract Hauling and Reuse of Biosolids by Synagro

Synagro is national waste recycling company that provides a wide array of solids handling services, which may include sludge collection, transport, treatment, and disposal. For Discovery Bay,, the most appropriate service by Synagro would be collection, transport, and agricultural application of Class A biosolids. The solids could be taken periodically, every few weeks, or seasonally, depending on operator preferences. Synagro’s fees are currently in the range of $60 to $70 per wet ton in the dry season and $80 to $90 per wet ton in the wet season. Since the District has the ability to stockpile solids on-site and export only during the dry season, only the lower cost would be applicable.

The District’s current solids handling operations would continue, with the only difference being that the final stockpiled solids would be loaded into Synagro trucks for their subsequent handling, versus loading into other trucks for landfill disposal.

18.5.3 Hauling of Biosolids to Lystek for Handling and Reuse

Lystek Organic Material Recovery Center (OMRC) in Fairfield is a regional recycling facility owned and operated by Lystek under a unique, public-private partnership with the Fairfield Suisun Sanitation District (FSSD). Lystek can receive sludge cake at less than 30% solids for processing through the Lystek Thermal Hydrolysis Process (Lystek THP®), which produces LysteGro®, a US EPA recognized and CDFA licensed, Class A biofertilizer product that is sold into the surrounding market area. In addition to thermal hydrolysis, this facility has recently incorporated a soil blending process that allows them to accept Class A biosolids, blend it with sand and ash to produce a soil amendment which is then sold locally.

Since TDBCSD already operates active solar dryers that have adequate capacity through buildout, transport of dried Class A biosolids to Lystek would be the most cost-effective option. It would not be cost effective to transport wet sludge cake directly off the belt presses to Lystek for thermal hydrolysis processing. The cost for Lystek to handle dried Class A biosolids would be $60 to $70 per ton, not including hauling costs. Since Synagro’s service would include hauling for the same cost, the Lystek alternative is not evaluated further.

18.5.4 Life Cycle Cost Comparison of Biosolids Disposal Options

Based on foregoing discussion, three alternatives were considered for future biosolids reuse:

1. Land application of all biosolids on District-owned land (requires additional land acquisition).
2. Maximize land application of biosolids on existing District property and contract with Synagro (or similar service) for hauling and land application of the remainder.
3. Hauling and land application of all biosolids by Synagro (or similar service).

Estimated life cycle costs for these alternatives are shown in Table 18-2. As indicated in the table, the most cost-effective solution appears to be hauling and land application of biosolids by Synagro (or other service provider to be selected by the District).

18.6 Recommended Improvements

Based on the evaluations presented in this section and based on additional input by the District's Engineer regarding existing damaged conduits in the solar dryers, the only recommended improvements are as follows:

- New sludge dredge for sludge lagoons - $125,000
- Repair damaged solar dryer conduits - $55,000
- Total - $180,000

In advance of upcoming limitations on landfill disposal, the District should solicit bids from Synagro and other similar companies for hauling and disposal/reuse of its biosolids.

Table 18-2 Life Cycle Cost Analysis of Solids Disposal Options

20-Year Life Cycle Cost Ananlysis		Option 1 Land Application of All Biosolids on District Land	Option 2 Maximize District Land Application + Rest to Synagro	Option 3 Hauling and Land Application of all Biosolids by Synagro
Capital Costs (a)				
Construction Costs				
Rough Site Clearing and Grading @ $2,000/acre		$ 100,000	$ 50,000	$ -
Irrigation System for Land Application		$ 1,595,000	$ 835,000	$ -
Monitoring Wells		$ 280,000	$ 140,000	$ -
Tailwater Collection and Pump Station		$ 150,000	$ 150,000	$ -
Agricultural Equipment		$ 250,000	$ 250,000	$ -
Subtotal		$ 2,375,000	$ 1,425,000	$ -
Gen Conditions, Overhead, Profit, Conting. (b)		$ 1,330,000	$ 798,000	$ -
Total Construction Cost		$ 3,705,000	$ 2,223,000	$ -
Land Acquisition (c)		$ 350,000	$ -	$ -
Engineering And Administration		$ 371,000	$ 222,000	$ -
Total Capital Costs		**$ 4,426,000**	**$ 2,445,000**	**$ -**
O&M Costs				
Labor Cost (d)	$/year	$ 93,600	$ 93,600	$ -
Maintenance (e)	$/year	$ 3,000	$ 3,000	$ -
Fuel for On-Site Handling (f)	$/year	$ 4,500	$ 3,000	$ -
Total O&M Costs	**$/year**	**$ 101,100**	**$ 99,600**	**$ -**
Contract Hauling and Land Applic. Cost (g)	**$/year**	**$ -**	**$ 9,481**	**$ 28,000**
Total Annual Costs	**$/year**	**$ 101,000**	**$ 109,000**	**$ 28,000**
TOTAL 20-year Life Cycle Cost (h)		**$5,553,000**	**$3,661,000**	**$312,000**

(a) All costs are based on based on 20-Cities ENR of 11,500.
(b) General Conditions, Overhead, and Profit = 20%. Contingencies = 30%. Compounded Total Allowance = 56%.
(c) Land acquisition cost = $14,000/acre.
(d) Labor cost = $90/hr.
(e) Includes maintenance cost for monitoring wells, irrigation system, and tailwater collection system, as applicable.
(f) Fuel cost = $5/gal.
(g) Hauling and land application by Synagro includes transportation. Calculated for mid point solids production of 300 dry tons/year and disposal cost of $70/wet ton.
(h) 20 years at net discount rate of 3%, Present Worth Factor = 14.8775, adjusted x 0.75 to allow for lower flows and loads in early years.

19.0 SCADA SYSTEM

In this section, the existing SCADA system is described, and improvement recommendations are presented.

19.1 EXISTING FACILITIES

The existing SCADA server configuration consists of a primary server located at Plant 2 and a backup server located at Plant 1, with fiber optics communications between the two. The supervisory software is Ignition® from Inductive Automation. Allen-Bradley PLCs are used throughout both plants for process control. Comcast cable service is used for Internet access.

Remote sites communicate to the Plants over serial radios, ethernet radios, or cellular modems. Table 19-1 provides an overview of the District's remote sites and their communication paths and PLC hardware.

Table 19-1 Remote Site Communications Overview

Remote Site	Communicates To	Communications Hardware	PLC Type
Bixler Lift Station	Plant 1	MDS 9810 Serial Radio	Modicon
West Village Lift Station	Plant 1	MDS 9810 Serial Radio	Modicon
Lakes Lift Station	Plant 1	MDS 9810 Serial Radio	Modicon
Lakeshore Lift Station	Plant 1	MDS 9810 Serial Radio	Modicon
Newport Drive Lift Station	Plant 1	MDS 9810 Serial Radio	Modicon
West Village Lift Station	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "A"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "C"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "D"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "E"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "F"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "G"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "H"	Plant 1	MDS 9810 Serial Radio	Modicon
Lift Station "J"	Plant 2	MDS Orbit Ethernet Radio	AB MicroLogix

Remote Site	Communicates To	Communications Hardware	PLC Type
Lift Station “R”	Plant 2	MDS Orbit Ethernet Radio	AB MicroLogix
Lift Station “S”	Plant 1 or 2	Sierra Wireless Cellular Modem	AB CompactLogix
Willow Lake WTP	Plant 1 or 2	Sierra Wireless Cellular Modem	AB CompactLogix
Newport WTP	Plant 1 or 2	Sierra Wireless Cellular Modem	AB CompactLogix
Well 1	Plant 1 or 2	Sierra Wireless Cellular Modem	AB CompactLogix
Well 2	Plant 1 or 2	Sierra Wireless Cellular Modem	AB CompactLogix
Well 4	Plant 1 or 2	Sierra Wireless Cellular Modem	AB MicroLogix
Well 7	Plant 1 or 2	Sierra Wireless Cellular Modem	AB MicroLogix

19.2 RECOMMENDED IMPROVEMENTS

19.2.1 SCADA Hardware

The SCADA servers were installed in 2015 and the hardware is due for replacement.

19.2.2 Ethernet Radio System

A new radio tower with a master ethernet radio was installed at Plant 2 for the purpose of eventually converting each of the serial radio sites to ethernet radios. Due to system-wide instability discovered during the conversion of the third remote site to the ethernet radio platform, only two (2) remote sites remain on ethernet radio communications – Lift Station “F” and Lift Station “G”. For this reason, cellular modems have been installed on a number of remote sites.

Given the topography and distances involved, the ethernet radio system should work. It is worthwhile to revisit the ethernet radio system to determine the source of the limitation, as it is very likely a hardware or configuration issue that is preventing a system-wide rollout. A complete radio study / evaluation performed by an experienced communications company is recommended. Replacement radios should be installed, configured, and tested in the field for operability. The results of this effort would help plant staff determine if the remaining lift station upgrade efforts will receive ethernet radios or cellular modems. The use of ethernet radios, where applicable based on the radio survey, will provide substantial cost benefit over the life of the system due to cost saving of the required data plan associated with the cellular modems.

19.2.3 Fiber Optics

There are a number of improvements that need to be made to the fiber optics system between the two plants. This includes general organization and labeling of the fiber strands and upgrading and standardizing on connector types. It is recommended to use different connector types for differing media types. Typically, ST type connectors are used for multimode fiber and SC or LC connectors are used for single-mode fiber.

19.2.4 Golf Course Valve Station

It would improve the reliability and longevity of the communications equipment at the Golf Course Valve Station to have air conditioning installed inside of what has become a communications hub for the plants. Another recommendation is to move the equipment that is presently laying loose on shelves to a new dedicated network rack with an Uninterruptable Power Supply (UPS).

19.2.5 Network Switches

Replacing existing network switches at the plants with switches that eliminate PLC multicast traffic would go a long way in reducing network collisions and slowdowns. Visualizing the switch statuses and diagnostics on the SCADA screens would help the Operations and Maintenance groups troubleshoot future communication bottlenecks and issues.

19.2.6 Video Camera Integration

The final recommendation is to add video cameras throughout the plant and bring the video feeds into the SCADA application for remote viewing.

Table 19-2 Cost Estimate for SCADA Improvements

	Cost, $		
Item	**Unit Price**	**Qty**	**Total Price**
New SCADA Server Equipment and Configuration	40,000	1	40,000
System-wide Radio Study **(note 1)**	10,000	1	10,000
Fiber Optics Improvements	10,000	1	10,000
Network Rack and new UPS at Golf Course Valve Station	15,000	1	15,000
Install Air Conditioning at Valve Station	7,000	1	7,000
Replace Network Switches; Configure SCADA Screens	20,000	1	20,000
Video Cameras and Integration into SCADA	4,000	10	40,000
Subtotal			142,000
Contingencies @ 20%			28,000
Total			170,000

Note 1: If the radio study proves that ethernet radios are viable for additional deployments, the estimated cost of replacing the master ethernet radio and antenna at Plant 2 is $5,000. The estimated cost for ethernet radios and antennas at each remote site is $3,000. Having ethernet radios as an option for the upcoming lift station upgrade projects gives plant staff an alternative to cellular modems, which presently carry a monthly data plan cost of $15/month per site.

20.0 REHABILITATION AND IMPROVEMENT OF PLANT 1

The previous Master Plan Amendment 3, dated March 2016, was developed to address the future use of Plant 1. In this section, the analysis and results from that investigation are summarized and an updated evaluation of Plant 1 improvements is presented.

20.1 SUMMARY OF PREVIOUS MASTER PLAN AMENDMENT 3

In the previous Master Plan Amendment 3, three major alternatives for Plant 1 were evaluated as follows:

Alt. 1: Rehabilitate the existing oxidation ditch and clarifiers, including structural repairs and new mechanical equipment, replace existing MCC-C, and correct additional deficiencies.

Alt. 2: Rehabilitate the existing oxidation ditch, including structural repairs and new mechanical equipment, construct two new clarifiers with modern features, replace existing MCC-C, and correct additional deficiencies.

Alt. 3: Replace the existing Plant 1 secondary treatment facilities with new facilities located at Plant 2.

Capital and annual costs for all three alternatives were developed and are summarized in Table 20-1 (a copy of Table A3-4 from Amendment 3, with costs in 2016 dollars). Based on the costs in Table 20-1 and other considerations, Alternative 1 was recommended for implementation.

Table 20-1 Alternative Overall Cost Comparison

	Cost, $1,000's (a)		
	Alt. 1	**Alt. 2**	**Alt. 3**
Item	**Rehab Plant 1**	**Rehab Plant 1 and Replace Clars**	**Replace Plant 1 Facilities at Plant 2**
Capital Cost	3,973	6,989	13,816
Incremental Annual O&M Cost (b)	58	58	0
Present Worth of Annual O&M Cost (c)	863	863	0
Total Present Worth	4,894	7,910	13,816

(a) First quarter 2016 cost level, ENR 20-Cities CCI = 10,200.
(b) Incremental cost above least cost alternative.
(c) 20 years at 3%, Present Worth Factor = 14.8775.

20.1 UPDATED EVALUTION OF PLANT 1 IMPROVEMENTS

Based on the analysis presented in Section 11, Plant 2 alone will likely not be able to handle critical design peak flow and load conditions in cold winter months and with the design SVI of 175 mL/g. Therefore, it likely will be necessary to operate Plant 1 in these conditions. Additionally, Plant 1 will be required to operate when it is necessary to take an oxidation ditch at Plant 2 out of service for major maintenance or repairs. Accordingly, Plant 1 must remain in operable condition, even if it is not actually operated in most years. Therefore, many of the recommendations for rehabilitation of Plant 1 developed in the previous Master Plan Amendment 3 are still appropriate. However, if it is considered that Plant 1 is mostly a backup to Plant 2 and will be operated only infrequently and mostly in future years as flows and loads approach the buildout condition, some of the previously recommended improvements can be considered as non-essential and can be deferred until such time (if ever) as the District determines it would be cost-effective to implement these improvements. Additionally, some of the previously recommended improvements have already been completed.

In Table 20-2, the improvements recommended in the previous Master Plan Amendment 3 are listed together with the previously estimated costs in 2016 dollars. The improvements are then categorized in subsequent columns to indicate whether they have already been completed and, if not, whether they are considered essential or non-essential. For the essential and non-essential future improvements, updated costs in 2019 dollars are indicated. Also shown in Table 20-2 and categorized as essential and non-essential are improvements that were not listed in the previous Master Plan Amendment 3 but have been identified for this Master Plan update by the District Engineer working with the plant operations Project Manager. For the convenience of having all Plant 1 improvements listed in one place, the anoxic basins and related facilities at Plant 1 needed for meeting new permit limits for nitrate+nitrite-nitrogen and developed in detail in Section 11 are included in Table 20-2.

Table 20-2 Plant 1 Improvements

Item	2016 Cost (a), $1000s	Completed?	2019 Cost (b), $1000s Essential	Non-Essential	Comment
Items in Previous Master Plan Amendment 3					
Influent Pump Station Area Misc. Improvements	40	Partly	10	---	Grating not completed.
Influent Pump Station and Pump Sta W Standby Power	200	Yes	---	---	
Oxidation Ditch Structural Rehab and Guardrail Repair	500	No	554	---	
Oxidation Ditch Rotor and Sump Pump Replacement	360	Yes	---	---	
Clarifiers Structural Rehab	50	No	55	---	Injection grouting of cracks
Clarifiers Mechanical Replacement and Upgrade	540	No	598	---	Mechanisms, launder covers, density baffles
MCC-C Replacement	250	No	277	---	
MCC-C Standby Power	150	No	166	---	
Headworks New Odor Control System	80	Yes	---	---	
Headworks Grating, Instrumentation, and Misc.	25	Partly	28	---	Grating not completed. Cost estimate increased.
Clarifier 2 Lift Station Instrumentation and Controls	50	Yes	---	---	
Clarifier 1 and 2 RAS Pumps and Check Valves Replacement	180	No	---	199	
WAS Pumps and Check Valves Replacement	64	No	---	71	
Storm Drainage Improvements	10	No	25	---	Cost estimate increased.
Transfer Station Instrumentation and Controls	50	Partly	25	---	
Demolish Existing Abandoned Facilities	100	No	111	---	
Additional Items					
Extend Pump Sta. F Forcemain to Pump Sta. W Manhole			25	---	
Coat Electrical Cabinets at Influent Pump Sta.			5	---	
Pump Sta. W Isolation Valve			20	---	
Oxidation Ditch Rotor Frame Elect. and Struct. Rehab.			400	---	
Subtotal 1	2,649		2,299	270	
Contingencies @ 20%	530		460	54	
Subtotal 2	3,179		2,759	324	
Engineering, Admin, and Environmental @ 25%	795		690	81	
Total without Anoxic Basins and Related	3,974		3,449	405	
Anoxic Basins and Related (c)	---		2,619	---	
Total with Anoxic Basins and Related	3,974		6,068	405	

(a) First quarter 2016 cost level, ENR 20-Cities CCI = 10,200.
(b) Mid-2019 cost level, ENR 20-Cities CCI = 11,300.
(c) From Section 11, including contingencies, engineering, administration, and environmental.

21.0 MISCELLANEOUS IMPROVEMENTS

Miscellaneous improvements have been identified at both Plant 1 and Plant 2 as described herein.

21.1 STORMWATER COLLECTION BASIN, PLANT 2

Stormwater from the main part of Plant 2 goes into the Decant Pump Station and directly to one of the oxidation ditches. If there were any chemical spills or other problems on the plant site, the biology would be immediately impacted. While it would be nice to have a stormwater basin to capture and hold the contaminated stormwater runoff until it could be safely brought back, it would be extremely cost-prohibitive to try to intercept the existing stormwater pipes that go into the Decant Pump Station, route them to a new pump station and a stormwater basin. Furthermore, the filter backwash line is tied to a stormdrain pipe and would have to be separated and re-routed.

Instead of separating stormwater pipes and a constructing a new stormwater basin, it is recommended to provide the following improvements:

1. The Decant Pump Station currently discharges to either or both of the oxidation ditches. A new discharge pipe will be provided to allow discharge to either of the sludge lagoons.

2. Provide motorized valves on all three Decant Pump Station discharges and control through SCADA for selecting the discharge location. Provide manual valves to select which sludge lagoon gets the discharge.

3. Provide sludge lagoon level instrumentation and signals to SCADA so Operations staff can make sure the sludge lagoons are not getting too full.

The total capital cost of the recommended improvements is estimated to be $84,000.

21.2 DRAIN SYSTEM IMPROVEMENTS

The existing oxidation ditches have drains through the transfer pump stations. The clarifiers cannot be completely drained using RAS pumps, with approximately 5-feet of water remaining at pump shutoff. The current practice of using a portable trash pump to drain the remaining water in the clarifiers will be continued because it is cost-prohibitive to install new drains and pumps below the existing clarifiers. Operations staff will continue to use portable trash pumps for complete drainage of the clarifier lift stations also. Although there are none currently planned, any future clarifiers will have permanent drain features, to allow complete drawdown of the water (without use of trash pumps).

21.3 CLARIFIER LAUNDER COVERS

As described in Chapter 11, flow is introduced into each clarifier through a center feed well and exits the clarifier over v-notch weirs and through an effluent launder around the perimeter of the tank. The effluent launders are currently uncovered, which allows algae to grow on the weirs and inside the concrete launder troughs. Currently, operations staff manually clean the launders at Plant 2 every week (Plant 1 is out of service), using water hoses. While the cleaning operation is underway, all of the secondary effluent is diverted to the sludge lagoons to prevent the algae debris from going to the filters. It is recommended to install launder covers at the clarifiers to mitigate the algae growth, thereby eliminating the need for manual cleaning and diversions to the sludge lagoons. The estimated capital cost to provide covers is $338,000 (this includes all five existing clarifiers at Plant 1 and Plant 2).

21.4 RECLAIMED WATER LINE EXTENSION

To allow extensive reuse of the District's effluent during the dry season, the District could extend an 8-inch reclaimed water pipe to the golf course (discharging into a water hazard for subsequent irrigation use by the golf course). The existing reclaimed water booster pump station (in Plant 2) can be used, with no improvements currently planned. The reclaimed water pipeline would be installed under Hwy 4 (using trenchless technology, such as bore and jack method) and then northward along the Plant 1 access road. Two alternatives for the remainder of the pipeline are:

- Option A: discharge into the golf course's water hazard near Oxidation Ditch 1 (in Plant 1).
- Option B: extend the pipeline north to Marina Road and discharge in the water hazard near the intersection of Marina Road and Channel Drive.

The estimated capital cost for Option A (extending the line to just north of Plant 1) is $1.37 million and is included in the overall summary of costs presented in Section 22. The estimated capital cost for Option B (extending the line to Marina Road) is $1.66 million.

21.5 RECLAIMED WATER FILLING STATION

To maximize reuse of the wastewater effluent for construction, the District could install a reclaimed water filling station. The bulk water filling station would include an electric actuated isolation valve, a flow meter, backflow prevention devices, and a card reader to allow account access to pre-approved users (for billing and tracking). The location would need to allow access to the general public with appropriate fencing and road improvements. The estimated capital cost for the water filling station is $198,000.

22.0 SUMMARY OF RECOMMENDED IMPROVEMENTS

In the previous sections of this report, various portions of the Town of Discovery Bay wastewater facilities are evaluated and specific recommendations for improvements are made. In some cases, further investigations are needed to confirm the improvements and costs. In particular, the secondary process improvements needed to meet the upcoming permit requirements for nitrate+nitrite-nitrogen and ammonia-nitrogen must be verified based on follow-up investigations that are identified in Section 11.

A list of all the recommended improvements developed in this Master Plan is presented in Table 22-1. For each improvement, a reference is given to the Master Plan section where that improvement is discussed in more detail, a budgetary cost is given, and the timing or condition that would trigger the need for the improvement is indicated. Costs are indicated in three columns to distinguish those improvements that are considered to be essential, those that are non-essential (but still recommended when available budgets allow implementation), and those that are unlikely to be required.

Proposed site plans with recommended improvements are presented in Figures 22-1 and 22-2 for Plants 1 and 2, respectively.

TOWN OF DISCOVERY BAY COMMUNITY SERVICES DISTRICT

SUMMARY OF RECOMMENDED IMPROVEMENTS

Table 22-1 Recommended Improvements

Item	Plant	Description	Rept. Sect.	Reason for Improvement	Trigger for Implementation	Possible Timing (a)			Budgetary Cost, $1000s (b)		
						Begin Design	Begin Const.	Begin Operation	Essential	Non-Essential	Unlikely
1	1&2	Anoxic Basins and Related Improvements for Denitrification	11, 20	Compliance with New Discharge Requirements	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	7,844 (c)		
2	1&2	Supplemental Aeration in Oxidation Ditches	11	Existing Rotors Inadequate for Future Max Oxygen Demand	Before Actual Oxygen Demands Exceed Reliable Rotor Capacity	2019	2021	2023	800 (d)		
3	2	UV Disinfection Testing and Improvement	14	Improve Performance	Desired Now	2019	2021	2023	200		
4	NA	Repair Effluent Diffuser in Old River	15	Restore Diffuser Capacity	Desired Now	2019	2021	2023	500		
5	1	Emergency Storage Drain to Pump Sta. W	16	Avoid Inconvenient and Inefficient Use of Temporary Pump System to Drain Emergency Storage Basin	When Possible	2019	2021	2023		75	
6	2	Solids Handling Improvements	18	Replace Dredge, Conduits	When Desired	TBD	TBD	TBD		180	
7	1&2	SCADA Networking Improvements	19	SCADA Performance Problems	Desired Now	2019	2021	2023	170		
8	1	Influent Pump Station Grating	20	Safety Concern	Desired Now	2019	2021	2023	15		
9	1	Oxidation Ditch Structural Rehab and Guardrail Repair	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	831		
10	1	Clarifiers Structural Rehab	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	83		
11	1	Clarifiers Mechanical Replacement and Upgrade	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	897		
12	1	MCC-C Replacement	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	416		
13	1	MCC-C Standby Power	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	249		
14	1	Headworks Grating	20	Safety Concern	Desired Now	2019	2021	2023	42		
15	1	Clarifier 1 and 2 RAS Pumps and Check Valves Replacement	20	Replace Deteriorated Equipment	When Possible	TBD	TBD	TBD		299	
16	1	WAS Pumps and Check Valves Replacement	20	Replace Deteriorated Equipment	When Possible	TBD	TBD	TBD		107	
17	1	Storm Drainage Improvements	20	Prevent Flooding	Desired Now	2019	2021	2023	38		
18	1	Transfer Station Instrumentation and Controls	20	Existing Controls Failed	Desired Now	2019	2021	2023	38		
19	1	Demolish Existing Abandoned Facilities	20	Provide Clean and Safe Site	When Possible	TBD	TBD	TBD	167		
20	1	Extend Pump Sta. F Forcemain to Pump Sta. W Manhole	20	Allow Bypass of Influent Pump	Desired Now	2019	2021	2023	38		
21	1	Coat Electrical Cabinets at Influent Pump Sta.	20	White Paint to Prevent Overheat	Desired Now	2019	2021	2023	8		
22	1	Pump Sta. W Isolation Valve	20	Replace Existing Ruined Valve	Desired Now	2019	2021	2023	30		
23	1	Oxidation Ditch Rotor Frame Elect. and Struct. Rehab.	20	Needed for Plant 1 Reliability	Permit Compliance Deadline of December 31, 2023	2019	2021	2023	600		
24	2	Decant Pump Station Improvements	21	Allow Discharge to Lagoons	Desired Now	2019	2021	2023	84		
25	1&2	Clarifier Launder Covers	21	Eliminate Tedious Maintenance	When Possible	TBD	TBD	TBD	338		
26	2	Extend Reclaimed Water Pipeline to Golf Course	21	Allow Reuse on Golf Course	When Desired	TBD	TBD	TBD		1,370	
27	2	Water Filling Station for Reclaimed Water	21	Allow Easier Construction Reuse	When Desired	TBD	TBD	TBD		198	
28	NA	Collection System Pump Stations	4	Restore Wet Well Integrity	When Possible	TBD	TBD	TBD	180		
29	2	Reverse Osmosis Facilities	21	Reduce Effluent Salinity, Last Resort	If Required by Regulation -- Very Unlikely	TBD	TBD	TBD			20,000
Total by Category, Excluding Effluent Diffuser in Old River (e)									13,068 (e)	2,229	20,000
Total Essential and Non-Essential, Excluding Effluent Diffuser in Old River (e)									15,297		

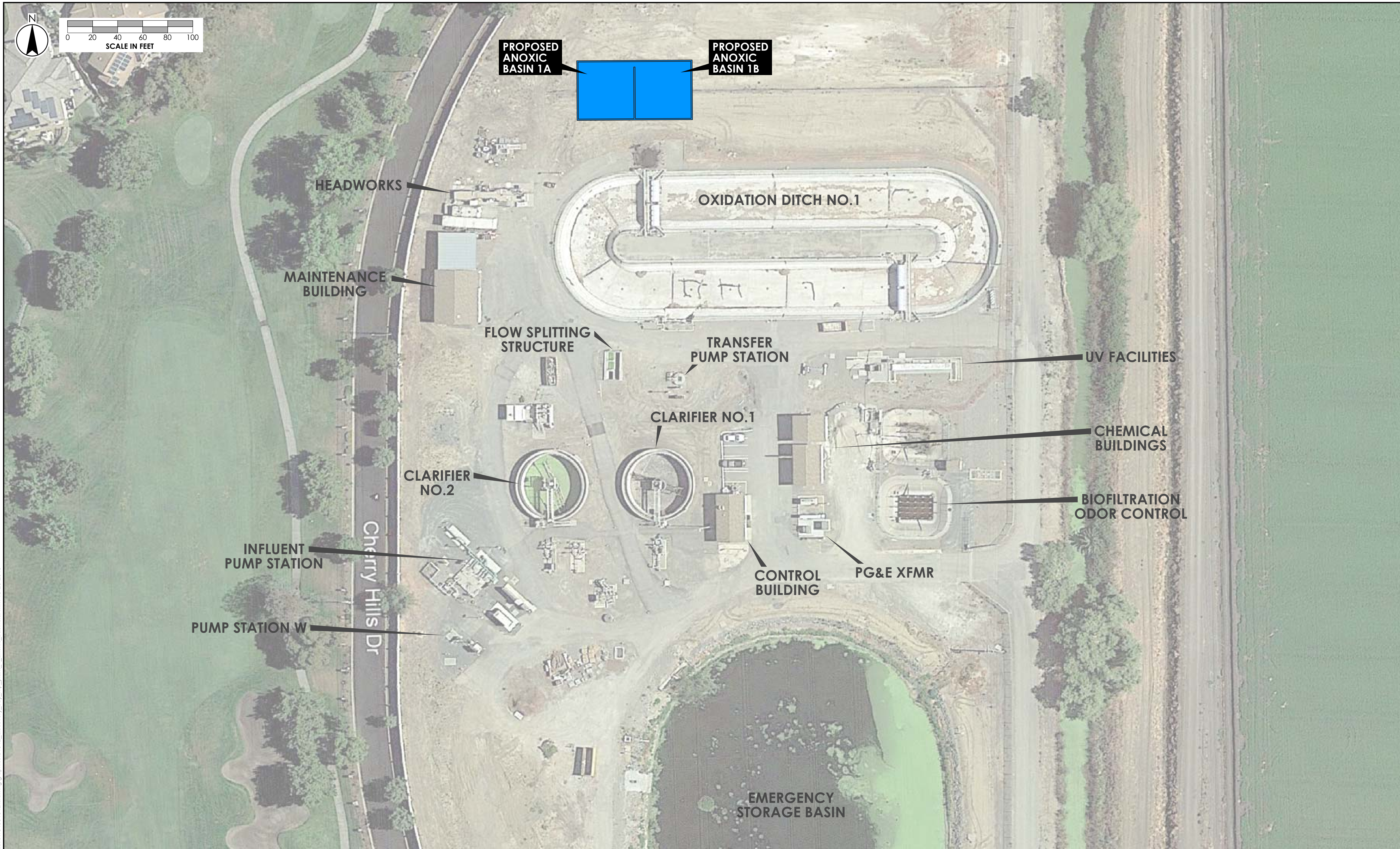
(a) Approximate timing recommendations, where applicable. TBD = To Be Determined.

(b) Total capital cost, including construction, contingencies, engineering, administration and environmental documentation, as applicable. Mid-2019 cost level. ENR 20-Cities CCI = 11,300.

(c) Validation of process design required after routine and intensive influent monitoring data is available from relocated influent sampler.

(d) Actual cost of supplemental aeration must be verified after special field studies to confirm existing rotor capacity and investigation of supplemental aeration alternatives.

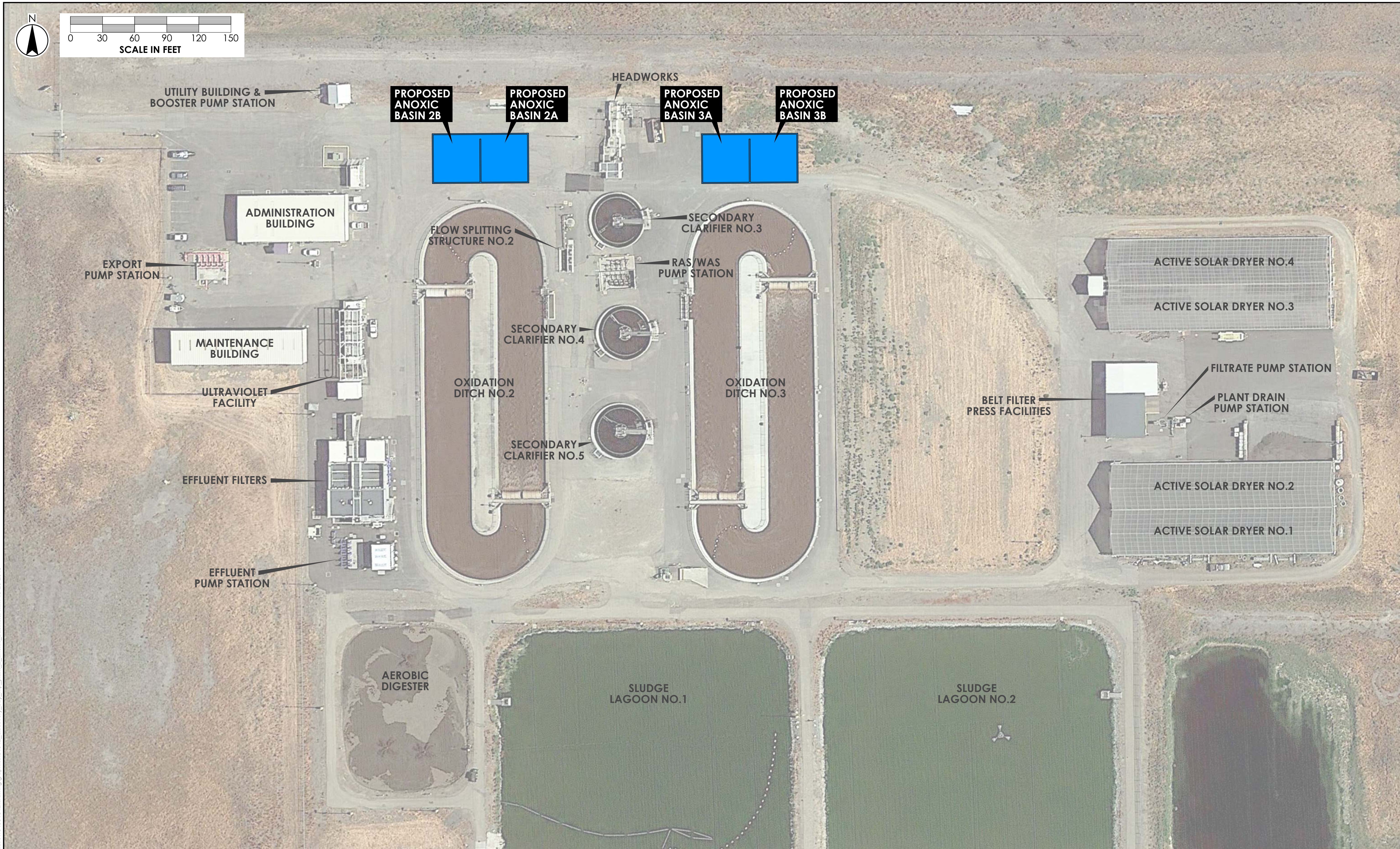
(e) Costs for repair of Old River outfall diffuser are excluded from total due to different funding than other essential Master Plan projects.



Stantec

Town of Discovery Bay
Community Services District
Wastewater Master Plan Update

Figure 22-1
Plant 1 Site Plan with Proposed Improvements



Stantec

Town of Discovery Bay
Community Services District
Wastewater Master Plan Update

Figure 22-2
Plant 2 Site Plan with Proposed Improvements

APPENDIX A

TROJAN UV3000Plus™ SPOT-CHECK BIOASSAY REPORT

DISCOVERY BAY COMMUNITY SERVICES DISTRICT
WASTEWATER TREATMENT PLANT
TOWN OF DISCOVERY BAY, CALIFORNIA

TROJAN UV3000Plus™
ULTRAVIOLET LIGHT DISINFECTION SYSTEM

SPOT-CHECK BIOASSAY REPORT

Prepared for:

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Honolulu, Hawai‘i 96819

October 2017

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EXECUTIVE SUMMARY

On October 2 – 3, 2017, a spot-check bioassay was completed on the Trojan UV3000Plus™ ultraviolet disinfection system installed at the Discovery Bay Community Services District Wastewater Treatment Plant (DBCSDWWTP), Town of Discovery Bay, California. The spot-check bioassay followed the State Water Resources Control Board (SWRCB), Division of Drinking Water (DDW) approved protocol. All testing was conducted in general accordance with Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, National Water Research Institute (NWRI) and American Water Works Association Research Foundation (AWWARF), August 2012, hereafter called the 2012 NWRI Guidelines.

The objectives were 1) demonstrate the Trojan UV3000Plus™ installed at DBCSDWWTP meets the 2012 NWRI Guidelines for delivering a 100 mJ/cm^2 dose, accounting for the appropriate validation factors end of lamp life (EOLL) and fouling factor (FF) and 2) determine the *scaling factor*, the measured dose to predicted dose residuals for the spot-check bioassay data.

Bioassay testing was conducted by adding MS2 phage, provided by GAP EnviroMicrobial Services (GAP), to the filtered effluent and collecting MS2 samples pre-UV and post-UV. The inactivation of the MS2 across a single UV bank was compared to the standard collimated beam curve (SCBC) equation to determine the UV dose (single bank). All collimated beam work and MS2 enumeration was performed by GAP. Collimated beam data was analyzed the same as the Trojan UV3000Plus™ 2012 NWRI validation report.

DBCSDWWTP UV recycled water disinfection system has 2 channels with 4 banks each (1 channel is standby). MS2 was injected in the suction piping for the recirculation flow and the SuperHume™ was batch injected in the suction piping as well. The recirculated flow discharging into the UV Channel Distribution Box upstream from the UV channels. Eight UV dose test runs and one no UV dose test run were completed.

The UV3000Plus™ was validated (ranges and approved factors) per the 2003 NWRI Guidelines and updated per the 2012 NWRI Guidelines as defined by:

$$RED_{NWRI} = f(Q, UVT, BPL, FF, EOLL, CR)$$

where:

RED_{NWRI} = reduction equivalent dose per bank, mJ/cm^2 (75th prediction limit)
Q = flow per lamp per bank, gpm/lamp/bank (range 6.2 to 126.5)
UVT = UV transmittance, percent/cm (range 55 to 77)
BPL = ballast power level per bank, percent (range 60 to 100)
FF = fouling factor (0.95 approved)
EOLL = end of lamp life (0.91 approved)
CR = confidence ratio

The SWRCB DDW communicated that the success for disinfected tertiary recycled water spot-check bioassay is to have 7 of 8 tests (87.5 percent) equal to or greater than the predicted UV dose. Four out of 8 test runs (50.0 percent) passed the scaling factor (SF) equal to or greater than 1.0. This system should use the 2012 NWRI Guidelines equation with a 0.75 multiplier or 133 mJ/cm^2 UV Dose for control.

PROJECT BACKGROUND AND OVERVIEW

This is the spot-check bioassay test report for the Trojan UV3000Plus™ UV disinfection system installed at the Discovery Bay Community Services District Wastewater Treatment Plant (DBCSDWWTP), Town of Discovery Bay, California based on a protocol approved by the State Water Resources Control Board (SWRCB), Division of Drinking Water (DDW). The spot-check bioassay is to demonstrate that the UV system delivers the expected performance under the plant working conditions, as designed, fabricated, installed, and operated. All testing was conducted in general accordance with the Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, National Water Research Institute (NWRI) and American Water Works Association Research Foundation (AWWARF), August 2012, hereafter called the 2012 NWRI Guidelines. The spot-check bioassay was conducted by an experienced, skilled team, with independent microbiological analyses, and an independent third-party witness producing a report.

Purpose

The objectives of this specific spot-check bioassay were:

1) demonstrate the Trojan UV3000Plus™ installed at Discovery Bay Community Services District Wastewater Treatment Plant meets the 2012 NWRI requirements of delivering a validated dose of 100 mJ/cm^2, accounting for appropriate validation factors end of lamp life (EOLL) and fouling factor (FF); and

2) determine the *scaling factor* (SF), the measured dose to predicted dose residuals for the spot-check bioassay data.

The UV3000Plus™ was validated per the 2003 NWRI Guidelines and updated per the 2012 NWRI Guidelines as defined by:

$$RED_{NWRI} = f(Q, UVT, BPL, FF, EOLL, CR) \quad (1)$$

where:

RED_{NWRI} = reduction equivalent dose per bank, mJ/cm^2 (75th prediction limit)
Q = flow per lamp per bank, gpm/lamp/bank (range 6.2 to 126.5)
UVT = UV transmittance, percent/cm (range 55 to 77)
BPL = Ballast power level of the UV system per bank, percent (range 60 to 100)
EOLL = End of lamp life factor (0.91)
FF = Quartz sleeve fouling factor[1]
CR = confidence ratio

Discovery Bay CSD Wastewater Treatment Plant

The facility has both commercial and domestic flow that goes thru screening (preliminary treatment), oxidation ditches (biological process), secondary clarifiers (liquid/solids separation), continuous backwash filtration (advanced treatment) and ultraviolet disinfection. Solids are aerobically digested and/or stored in lagoons, belt filter press dewatered, and dried with active solar drying units before landfill disposal.

[1] FF = 0.95 - conditional CDPH acceptance October 5, 2006.

Discovery Bay CSD WWTP UV Disinfection System

The DBCSDWWTP UV disinfection system is a Trojan Technologies Inc.UV3000Plus™, which uses low-pressure, amalgam lamps. Lamp sleeves and lamps are oriented horizontal and parallel to the filtered flow. Channel 1 UV3000Plus™ suystem was installed in 2010 and Channel 2 UV3000Plus™ was installed this year. Each channel has 4 banks, each with 8 modules and each module containing 8 lamps (64 lamps per bank). The system control center automatically varies the banks on-line and ballast power level to maintain the 100 mJ/cm^2 UV dose based on flow, UVT, and lamp service hours. The disinfection system relies on the CDPH accepted EOLL and FF factors to deliver a conservative UV dose. The system has a fully automatic physical/chemical cleaning system to remove biofilm/chemical fouling on the lamp sleeves.

The existing and newly installed UV3000Plus™ system are the same system that was validated at the Whittier Narrows Water Reclamation Plant in Whittier, California. The validation report was submitted in February 2006 and conditionally approved April 2006 by the California Department of Health Services. A correction factor (CF) was added to the original equation (multiplier) and conditionally approved July 2009. A Validation Report Addendum - 2012 NWRI Analysis was submitted April 2014 and conditionally approved June 16, 2014 by the California Department of Public Health. The UV system design criteria are summarized in Table 1.

Table 1 - DBCSDWWTP UV System Design Parameters

UV System Characteristic	Value
Peak Hour Flow, MGD	4.8
Maximum Month Flow, MGD	2.42
Minimum Continuous Flow, MGD	0.3
Design UVT, percent	65
Design UV Dose (RED_{NWRI}), mJ/cm^2	100
End of Lamp Life (EOLL), decimal	0.91
Lamp Sleeve Fouling Factor (FF), decimal	0.95
Channel/s, number (1 existing and 1 new)	2
Duty Channel/s, number	1
Banks per Channel, number	4
Duty Banks per Channel, number	4
Standby Banks per Channel, number	0
Modules per Bank, number	8
Lamps per Module, number	8
Lamps per Bank, number	64
Duty Lamps per Channel, number	256
Lamps per Channel, number	256
Total System Lamps, number	512

Roles and Responsibilities

Third party oversight was provided by Victor Moreland (Moreland Consulting LLC). All sample preparation and post processing associated with microbial enumeration was done by GAP EnviroMicrobial Services (GAP). Plant flow control was done by the Contractor personnel. UV equipment operation was done by Trojan Technologies Inc.

Bioassay Schedule

MS2 stock was prepared (in accordance with 2012 NWRI) by GAP in London, Ontario, Canada and shipped to the Town of Discovery Bay, California. All microbial analysis was performed by GAP. Testing was done on October 2 and 3. Mixing test was done October 2 and all test runs were done October 3 and 2 collimated beam samples (different UVT's) were taken and prepared. The bioassay schedule with sample numbering is shown in Table 2. The third party after realizing that Channel 1 (existing equipment) was not fully rehabilitated prior to the scheduled Spot-Check Bioassay decided to conduct test runs on Channel 2 for compliance with the 2012 NWRI UV Guidelines. Therefore; this report is only about Channel 2 with the indicated modifications to the schedule.

Table 2 - Spot-Check Bioassay Schedule

Run	Channel	Bank	Flow, MGD	UVT, percent	BPL[1], percent	Reactor Sample	
						Pre-UV[2]	Post-UV
1	2	B	4.60	65	100	1001 1002 1003	2001 2002 2003
2	2	C	4.60	65	100	1004 1005 1006	2004 2005 2006
3	2	B	2.60	65	76	1007 1008 1009	2007 2008 2009
4	2	B	2.60	65	76	1010 1011 1012	2010 2011 2012
5	2	BC	4.60	55	100	1013 1014 1015	2013 2014 2015
6	2	C	2.60	55	100	1016 1017 1018	2016 2017 2018
7		Control (no dose)	2.60	55	0	1019 1020 1021	2019 2020 2021
8	2	C	1.00	55	76	1022 1023 1024	2022 2023 2024
9	2		1.00	55	76	1025 1026 1027	2025 2026 2027

[1]BPL = ballast power level

SYSTEM LAYOUT

The UV system layout at DBCSDWWTP is depicted in Figure 1. After the membranes, the effluent travels to the UV channel via exposed pipe thru the UV disinfection system to a sump downstream from the effluent weir. Figure 2 shows the MS2 injection point going down to the suction piping nozzle (tubing inserted 12 to 16 inches into suction pipe). SuperHume™ was batched into recirculated flow to lower UVT

Figure 1 - Discovery Bay CSD WWTP UV System Layout

Figure 2 - MS2 Injection Point (Recirculation Suction Piping Below)

Pre-UV samples were taken from the open space upstream from Bank A. Post-UV samples were downstream from Bank C.

To confirm proper installation, the UV system banks were checked for lamp spacing and distances to the side walls and channel bottom prior to the spot-check bioassay. The results for Banks B and C were within Trojan Technologies Inc. maximum channel distances. The UV system is designed with 4-inch lamp spacing (horizontal and vertical) and the lamp nearest the side-walls and channel bottom should be approximately 2-inches from lamp center to channel surface. Measurements showed that the channels have variable depth and width (to be expected with concrete channels).

Batch Preparation

MS2 stock (3 L provided by GAP) with a 5 x 10^{11} PFU/mL titer (density) was added to filtered effluent (approximately 90 L) to yield roughly a 3.9 x 10^9 pfu/mL batch density, which was then injected at a rate to yield effluent samples with measurable MS2 densities.

Super Hume™ is provided as 15 percent humic acid in water and was diluted. The injection rate was set to yield the desired UVT percent for each test run that was lower than the ambient UVT.

Flow Measurement

Recirculated flow was measured with magnetic flow meter (Figure 3) in the recirculation piping going to the UV Disinfection System.

Figure 3 - Magnetic Flow Meter

Electrical Testing

Power, current, voltage and power factor for each test bank was monitored during the bioassay testing using a Condura power analyzer, model “EnergyPro” (Certification of Calibration is in Appendix A).

UVT Measurement

For each spot-check bioassay test condition, UVT (254 nm) samples were taken at both pre-UV and post-UV sampling locations. All UVT measurement was done on site immediately before sampling, using a Real Tech Inc. single wave photometer unit (Real UVT Field Meter - Certificate of Analysis is in Appendix B) and manually recorded. Final UVT values used for the calculation were the average measurements from the two locations (upstream and downstream from the on-line bank).

Water Level Measurement

Water level measurement was manually recorded along the length of the channel (upstream from each on-line bank) for all the flow conditions at the WRWRF spot-check bioassay testing. A meter stick was used for the measurement, and the readings were taken from the UV channel bottom to the water surface (Table 3).

Table 3 - Water Levels with All Banks In-Channel

Channel	Bank	Water Level, inches		
		Min Flow (1.0 MGD)	Avg Flow (2.6 MGD)	Peak Flow (4.2 MGD)
2	B	32.0	32.5	32.75
2	C	32.0	32.5	32.875

Channel Mixing Test

Super Hume™ was batched (1liter for 1 minute) into the recirculated flow (inside the suction piping 12 to 16 inches) to lower UVT. Samples were taken at intervals from upstream and downstream sampling locations (Channel 2). The mixing study showed that adequate mixing was achieved from the Super Hume™ Table 4 shows the mixing test results. This illustrates that mixing had been achieved.

Time, min	Flow, MGD	UVT, percent/cm	
		Pre-UV Sample Point	Post-UV Sample Point
0	4.53	64.5	
5	4.54	55.1	51.7
10	4.52	56.3	56.0
15	4.51	56.8	56.7

BIOASSAY RESULTS

Spot-check bioassay MS2 results are presented in this section. All collimated beam testing and all bioassay culturing was done by GAP (same as the 2003 NWRI validation testing).

Collimated Beam Analysis

Collimated beam sample was taken after the last test run each day and two collimated beam analysis sets (raw data in Appendix C) were conducted for each day. The UV doses ranged from 20 to 80 mJ/cm^2, with a UV dose interval approximately every 20 mJ/cm^2. The target doses during the bioassay were all less than 50 mJ/cm^2.

Collimated beam data was treated in the same manner to the method used for the Trojan UV3000Plus™ 2003 NWRI validation report, which was to use linear regression analysis to generate the dose response equation using all data points. The CB dose response curves are plotted in Figure 7. The linear regression equation is shown below.

$$\text{Log Inactivation} = \text{Slope} \times \text{UV Dose} + \text{Intercept} \tag{2}$$

Using regression analysis, the following equation was determined for evaluating UV system performance.

CB 66.1 %/cm CB Log Inactivation (LI) = (0.038591 x UV Dose) + 0.463809 (3)
CB 58.5 %/cm CB Log Inactivation (LI) = (0.039373 x UV Dose) + 0.382191 (4)

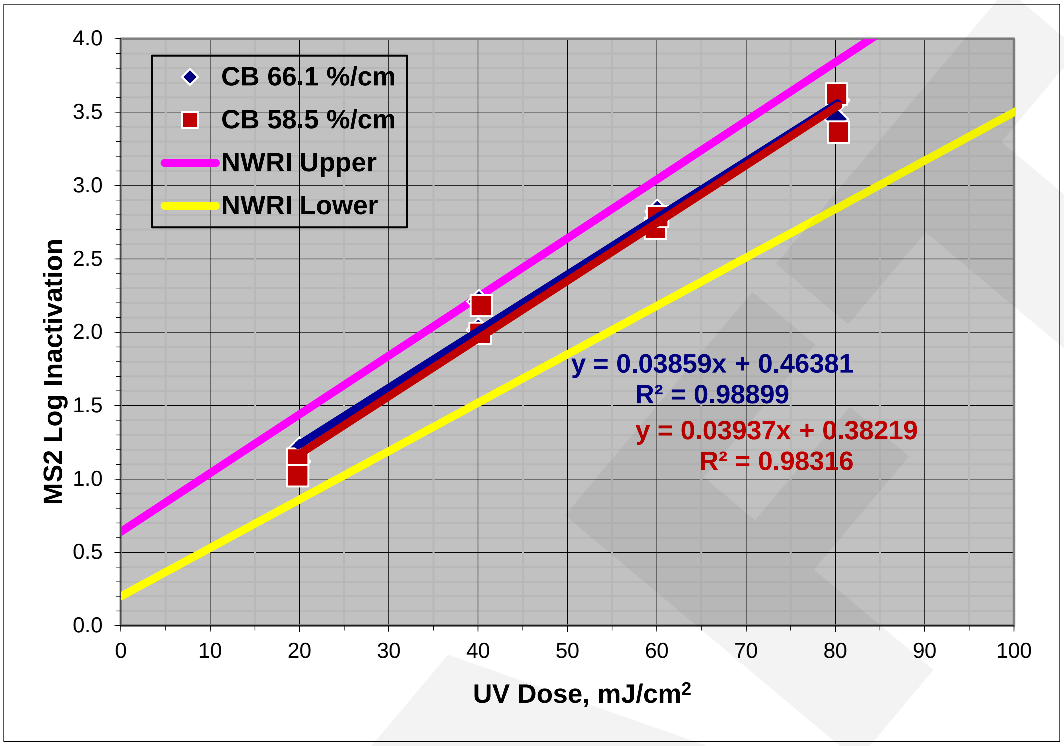

Figure 4 - Collimated Beam Analyzes

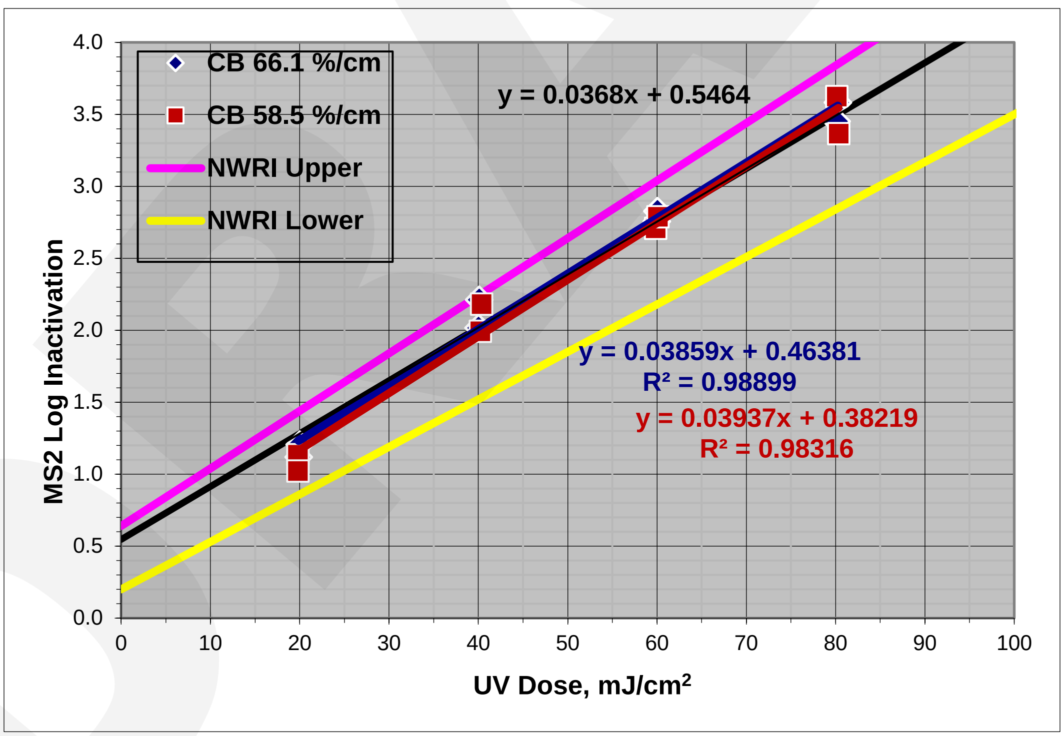

Figure 5 - Collimated Beam Analyzes with Standard Collimated Beam Curve

The collimated beam results were used to compare with the 2012 NWRI upper and lower boundaries, and were well within those boundaries. 2012 NWRI guidelines require that the standard collimated beam equation be used to convert the measured log inactivation values across the reactor to delivered UV dose for the collected samples. That equation is shown below, and the linear regression equation is shown in Figure 5:

$$x\ (\text{UV dose}) = \frac{y\ (\text{log inactivation}) - 0.5464}{0.0368} \tag{5}$$

UV Reactor Performance Results

For each spot-check bioassay test, three pre-UV and three post-UV samples were collected. Pre-UV samples were collected across the channel width at mid depth upstream from Bank A. Post-UV samples were downstream from Bank D and upstream from the weirs. All samples were collected in pre-labeled sterile sample tubes, and immediately placed in a sample ice chest. After all samples were collected they were place in refrigerator overnight and then shipped (in a sealed cooler with blue ice packs) via overnight courier to GAP for MS2 analysis (raw data in Appendix C). The testing was conducted well after a 100-hour burn-in period for the UV lamps. Using the standard collimated beam analysis and field and laboratory data results, the lower 75th prediction limit delivered UV dose values were calculated for each test condition during the spot-check bioassay test. The spot-check bioassay results are summarized in Table 4. The test run reactor UV Dose results were determined using the standard collimated beam curve detailed in the 2012 NWRI Guidelines.

Table 4 - Spot-Check Bioassay Results

Test Run	Channel	Bank	UVT, percent/cm	Power, percent	Flow, MGD	Flow, gpm/lamp/bank	SCBC UV Dose, mJ/cm^2
1	2	C	66.35	100	4.232	45.92	24.43
2	2	B	66.20	100	4.198	45.55	20.77
3	2	C	66.35	76	2.593	28.14	41.47
4	2	B	66.30	76	2.601	28.22	34.13
5	2	BC	54.60	100	4.221	45.80	32.41
6	2	B	55.80	100	2.650	28.75	25.67
7	2		55.50	0	2.615	28.38	Control
8	2	B	56.75	68	1.007	10.93	52.49
9	2	C	56.95	68	1.025	11.12	49.34

Nine UV dose test runs were completed for Channel 2 during the test day. The control test run showed only a minor change in the MS2 log density (4.0489 - 4.0537 = -0.0048 increase).

The scaling factors (SF) for the reactor UV doses are summarized in Table 5.

Table 5 - Scaling Factor (Measured UV Dose/Predicted UV Dose)

Test Run	Channel	Bank	UVT, percent/cm	Power, percent	Flow, gpm/lamp/bank	UV Dose, mJ/cm^2 Measured	UV Dose, mJ/cm^2 Predicted	SF
1	2	C	66.35	100	45.92	24.43	32.64	0.75
2	2	B	66.20	100	45.55	20.77	32.60	0.64
3	2	C	66.35	76	28.14	41.47	34.99	1.19
4	2	B	66.30	76	28.22	34.13	34.81	0.98
5	2	BC	54.60	100	45.80	32.41	31.75	1.02
6	2	B	55.80	100	28.75	25.67	25.75	0.997
7	2		55.50	0	28.38	Control		
8	2	B	56.75	68	10.93	52.49	38.49	1.36
9	2	C	56.95	68	11.12	49.34	38.43	1.28

The results shown used to determine the SF shows that the UV equipment at Discovery Bay CSD WWTP passed 4 of 8 test runs (50.0 percent). SWRCB requires passing 7 of 8 test runs (87.5 percent). When the spot-check bioassay results are less than required, a de-rating factor must be applied to the control logic. This system should use the 2012 NWRI Guidelines equation with a 0.75 multiplier or 133 mJ/cm^2 UV Dose for control.

Channel flow conditions observed during the test runs sampling showed that at the high flow (~ 4.2 MGD) the Pre-UV sample point (downstream from the channel isolation slide gate and lateral spillway channel (refer to Figure 6)) was very turbulent. At the mid flow (~ 2.6 MGD) there was mildly turbulent and at the low flow (~ 1.0 MGD) there was no noticeable turbulence.

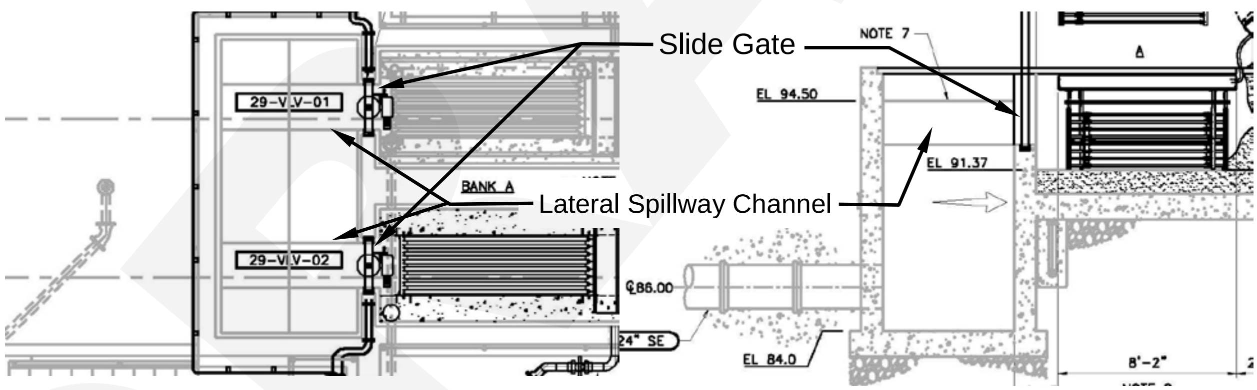

Figure 6 - Distribution Box Plan and Section

Looking at the results in Figures 7 and 8, it is easy to see the impact of the very turbulent condition on test runs 1 and 2 and even the mildly turbulent condition on test runs 3, 4, and 6. The low flow condition (test runs 8 and 9) shows no hydraulic influence on the bank results. Bank B the closest tested bank to the slide gate inlet was the effected the most during most test runs. Evaluating these results, indicates that flows above 2.6 MGD are beginning to be significantly impacts by the channel hydraulics (caused by the inlet conditions thru the slide gate and lateral spillway channel.

While, the channel flows are at 2.6 MGD and below the setpoint UV Dose should be 102 mJ/cm^2 or a 0.98 multiplier. When the flow exceeds 2.6 MGD the setpoint UV Dose should be 133 mJ/cm^2 or a 0.75 multiplier.

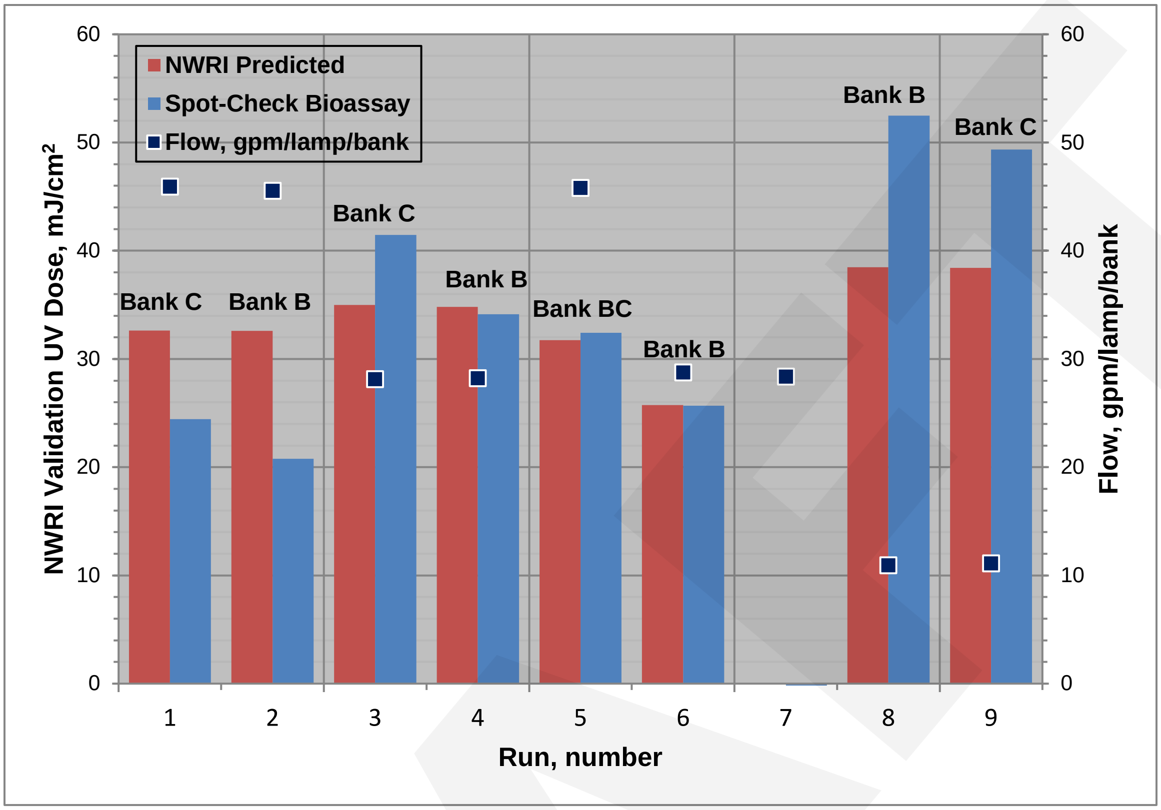

Figure 7 - Predicted UV Doses and Measured UV Doses

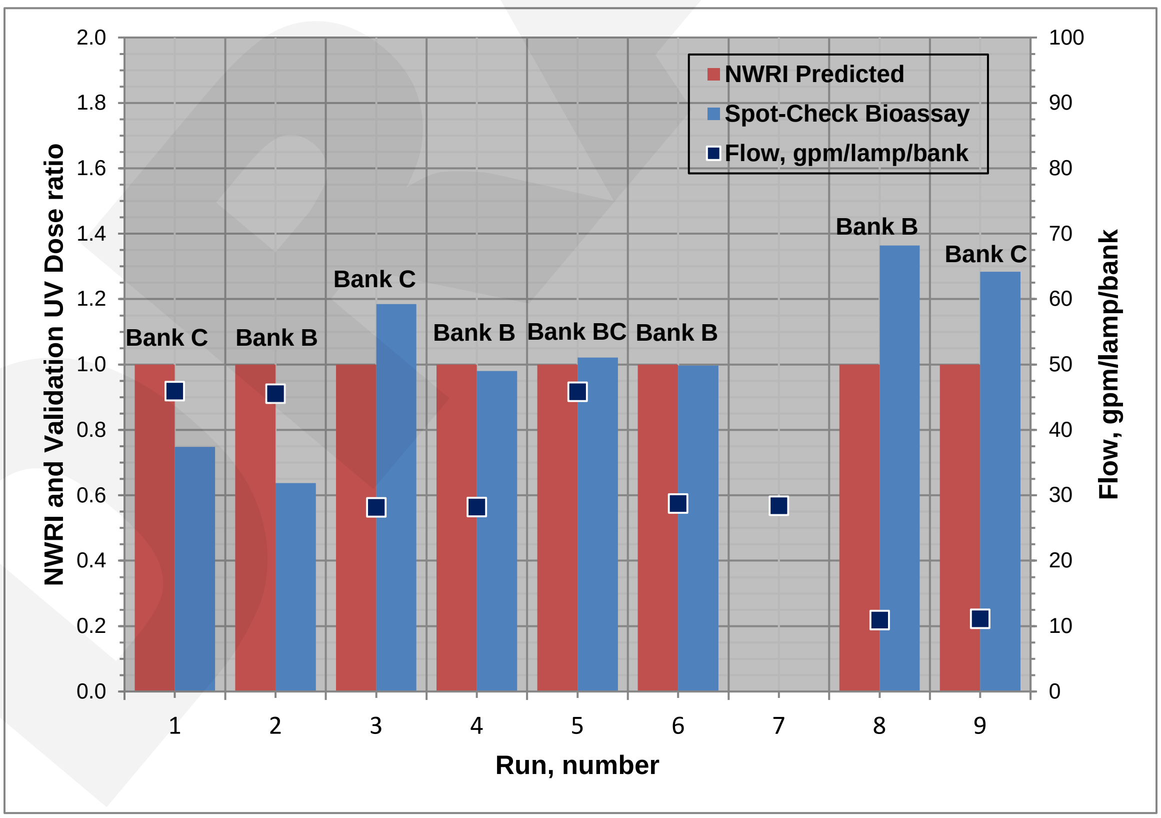

Figure 8 - Predicted and Measured UV Dose Scaling Factors

CONCLUSIONS AND RECOMMENDATIONS

The spot-check bioassay testing goal was to verify that the full-scale UV system, under operating conditions up to 4.20 MGD maximum flow at Discovery Bay Community Services District Wastewater Treatment Plant. The UV system operated less than expected results and UV Dose 2012 NWRI updated algorithm should be operated with a 0.75 multiplier or 133 mJ/cm^2.

While, the channel flows are at 2.6 MGD and below the setpoint UV Dose should be 102 mJ/cm^2 or a 0.98 multiplier. When the flow exceeds 2.6 MGD the setpoint UV Dose should be 133 mJ/cm^2 or a 0.75 multiplier.

REFERENCES

Collimated Beam Analysis, GAP EnviroMicrobial Services Ltd., SOP CODE: SOP53, April 14, 2008.

Quantitative Recovery of Bacteriophage Used for Disinfection Equipment Validation, GAP EnviroMicrobial Services Ltd., Method Code: Bactphage-0001, January 28, 2009

APPENDIX A - Current Meter Certificate of Calibration

CANDURA instruments
Rugged ▪ Reliable ▪ Weatherproof
775 Pacific Rd, Unit #26
Oakville, Ontario L6L 6M4
Tel: 905-847-6799
Fax: 905-847-0306

Certificate of Calibration

Certificate No:	EPCAL00335-2016-06-16
Manufacturer:	CANDURA Instruments
Model:	EP600i
Serial No:	EP60000335
Firmware Version:	6.028 06032012
Date Calibrated:	June 16, 2016

This instrument has been calibrated in accordance with the manufacturer's instructions using standards that are directly traceable to the National Institute of Standards and Technology (NIST) or the National Research Council of Canada (NRC). The specific path of traceability for the reported measurement results is maintained at the Candura Instruments facility and is available there for review.

See following pages for calibration data.

Calibration Meter:	Fluke 5080A
Model No:	5080A
Serial No:	1482120

Environmental Conditions

Temperature:	24°C
Relative Humidity	50%

Technician: R.B

Page 1 of 2

CANDURA instruments
Rugged ▪ Reliable ▪ Weatherproof
775 Pacific Rd, Unit #26
Oakville, Ontario L6L 6M4
Tel: 905-847-6799
Fax: 905-847-0306

Certificate No: EPCAL00335-2016-06-16
Serial No: EP60000335
Date: June 16, 2016

AC Voltage: Acceptable Error 0.1% of Range + 0.1% of Reading

Range (V)	Ch	600+/- 1.2		300+/- 0.9		60+/- 0.66	
600	1	600.1	Pass	299.9	Pass	60.0	Pass
	2	600.2	Pass	299.8	Pass	59.8	Pass
	3	600.2	Pass	299.9	Pass	59.8	Pass

Frequency (Ch1): Acceptable Error +/- 0.05 Hz

Range (Hz)	Ch	60+/-0.05	
60	1	60.00	Pass

Current Ch1,2,3 FlexCT: Acceptable Error 0.2% of Range + 0.2% of Reading

Range (A)	Ch	20+/- 0.08		10+/- 0.06		2+/- 0.044	
20	1	20.00	Pass	10.00	Pass	1.99	Pass
	2	20.00	Pass	9.99	Pass	1.98	Pass
	3	19.99	Pass	9.99	Pass	1.99	Pass
Range (A)	Ch	200+/- 0.8		100+/- 0.6		25+/- 0.45	
200	1	200.0	Pass	99.9	Pass	24.8	Pass
	2	200.1	Pass	99.9	Pass	24.9	Pass
	3	200.0	Pass	99.9	Pass	24.8	Pass
Range (A)	Ch	2000+/- 8		1000+/- 6		250+/- 4.5	
2000	1	2000	Pass	999	Pass	248	Pass
	2	2000	Pass	999	Pass	248	Pass
	3	2000	Pass	999	Pass	248	Pass

FlexCT kW reading @ 600V, 2000A

Ch	Reading (kW)
1	1200
2	1200
3	1200

Page 2 of 2

APPENDIX B - Photometer Certificate of Performance

SGS

SGS Lakefield Research Limited
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Real Tech Inc.

701 Rossland Rd. E, Unit #358
Whitby, ON
L1N 9K3,

Phone: 905-579-2888
Fax:pdf format

Thursday, February 14, 2008

Date Rec. : 13 February 2008
LR Report: **CA10231-FEB08**

Copy: Final # 1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	UV Transmittance (%) Real Tech Inc.	UV Transmittance (%) SGS
1: sample 1	90.0	90.1
2: sample 2	80.5	80.7
3: sample 3	69.0	69.2
4: sample 4	61.3	61.4
5: sample 5	51.5	51.5
6: sample 6	44.9	44.9
7: sample 7	36.3	36.3
8: sample 8	28.8	28.8
9: sample 9	19.5	19.4
10: sample 10	11.3	11.1

Chris Sullivan, B.Sc., C.Chem
Project Specialist
Environmental Services, Analytical

OnLine LIMS

Page 1 of 1

APPENDIX C - GAP EnviroMicrobial Services MS2 Results

1020 Hargrieve Road, Unit 14
London, ON N6E 1P5

FINAL RESULTS FORM

Ph # 519-681-0571
Fax# 519-681-7150

GAPLAB Environmental Microbiology

GAP JOB #: A12592	DATE SAMPLED: 03-Oct-17	REPORT TO: Trojan Technologies	
CLIENT: Trojan Technologies	COLLECTED BY: Steve McDermid	ATTENTION: Steve McDemid	
PROJECT: Discovery Bay	DATE RECEIVED: 05-Oct-17	ADDRESS: 3020 Gore Rd. London, ON N5V 4T7	
PAGE #: 1 of 3	RECEIVED BY: K. Shuty		
	ANALYSIS START: 05-Oct-17	TEL: 519-457-3400	FAX: 519-457-3030
	ANALYSIS FINISH: 11-Oct-17	EMAIL: smcdermid@trojanuv.com	EMAIL:

TEST RESULTS *-relate only to the samples submitted and the analyses requested.*

Accredited Method Code: BACTPHAGE-0001

LAB #	SENDERS #	SAMPLE DESCRIPTION	ASSAY DATE	MS2 - Replicate 1			MS2 - Replicate 2			MS2 - Replicate 3			Calculated Concentration (pfu/mL)		
				Dilution log	Aliquot (mL)	PFU	Dilution log	Aliquot (mL)	PFU	Dilution log	Aliqot (mL)	PFU	Replicate 1	Replicate 2	Replicate 3
7208	1001		06-Oct-17	-2	1	109	-2	1	99				1.09E+04	9.9E+03	
7209	1002		06-Oct-17	-2	1	85							8.5E+03		
7210	1003		06-Oct-17	-2	1	110							1.10E+04		
7211	1004		06-Oct-17	-2	1	102							1.02E+04		
7212	1005		06-Oct-17	-2	1	86							8.6E+03		
7213	1006		06-Oct-17	-2	1	97							9.7E+03		
7214	1007		06-Oct-17	-2	1	86							8.6E+03		
7215	1008		06-Oct-17	-2	1	74							7.4E+03		
7216	1009		06-Oct-17	-2	1	94							9.4E+03		
7217	1010		06-Oct-17	-2	1	63							6.3E+03		
7218	1011		06-Oct-17	-2	1	78							7.8E+03		
7219	1012		06-Oct-17	-2	1	62							6.2E+03		
7220	1013		06-Oct-17	-2	1	65							6.5E+03		
7221	1014		06-Oct-17	-2	1	77							7.7E+03		
7222	1015		06-Oct-17	-2	1	107							1.07E+04		
7223	1016		06-Oct-17	-2	0.5	26							5.2E+03		
7224	1017		06-Oct-17	-2	0.5	74							1.5E+04		
7225	1018		06-Oct-17	-2	0.5	67							1.3E+04		
7226	1019		06-Oct-17	-2	1	115							1.15E+04		
7227	1020		06-Oct-17	-2	1	115							1.15E+04		
7228	1021		06-Oct-17	-2	1	113	-2	1	105				1.13E+04	1.05E+04	
7229	1022		06-Oct-17	-3	2	67							3.4E+04		
7230	1023		06-Oct-17	-3	2	84							4.2E+04		
7231	1024		06-Oct-17	-3	2	50							2.5E+04		
7232	1025		06-Oct-17	-3	1	30							3.0E+04		
7233	1026		06-Oct-17	-3	1	35							3.5E+04		
7234	1027		06-Oct-17	-3	1	21							2.1E+04		
7235	2001		06-Oct-17	0	0.5	195							3.90E+02		
7236	2002		06-Oct-17	0	0.5	177							3.54E+02		
7237	2003		06-Oct-17	0	0.5	168							3.36E+02		
7238	2004		06-Oct-17	-1	1	39							3.9E+02		
7239	2005		06-Oct-17	-1	1	55							5.5E+02		
7240	2006		06-Oct-17	-1	1	93							9.3E+02		
7241	2007		06-Oct-17	0	1	65							6.5E+01		
7242	2008		06-Oct-17	0	1	93							9.3E+01		
7243	2009		06-Oct-17	0	1	60							6.0E+01		

CALCULATED BY: S. Verhoeven	POSITION: Lab Manager	MANAGER APPROVAL: C. Odegaard (Technical Manager)	
SIGNATURE: S. Verh		SIGNATURE: Cgl	DATE: 11-Oct-17

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< = Less Than; > = Greater Than; TNTC = Too Numerous To Count; OBSC = Obscured; NR = No Result; LA = Laboratory Accident
CFU = Colony Forming Unit; PFU = Plaque Forming Unit; MF = Membrane Filtration; MPN = Most Probable Number; SP = Spread Plate

CALA Testing Accreditation No. A 2914

1020 Hargrieve Road, Unit 14
London, ON N6E 1P5

FINAL RESULTS FORM

Ph # 519-681-0571
Fax# 519-681-7150

GAPLAB Environmental Microbiology

GAP JOB #:	A12592	DATE SAMPLED:	03-Oct-17
CLIENT:	Trojan Technologies	COLLECTED BY:	Steve McDermid
PROJECT:	Discovery Bay	DATE RECEIVED:	05-Oct-17
PAGE #:	2 of 3	RECEIVED BY:	K. Shuty
		ANALYSIS START:	05-Oct-17
		ANALYSIS FINISH:	11-Oct-17

REPORT TO:	Trojan Technologies		
ATTENTION:	Steve McDermid		
ADDRESS:	3020 Gore Rd. London, ON N5V 4T7		
TEL:	519-457-3400	FAX:	519-457-3030
EMAIL:	smcdermid@trojanuv.com	EMAIL:	

TEST RESULTS *-relate only to the samples submitted and the analyses requested.*

Accredited Method Code: BACTPHAGE-0001

LAB #	SENDERS #	SAMPLE DESCRIPTION	ASSAY DATE	MS2 - Replicate 1			MS2 - Replicate 2			MS2 - Replicate 3			Calculated Concentration (pfu/mL)		
				Dilution log	Aliquot (mL)	PFU	Dilution log	Aliquot (mL)	PFU	Dilution log	Aliqot (mL)	PFU	Replicate 1	Replicate 2	Replicate 3
7244	2010		06-Oct-17	0	1	88							8.8E+01		
7245	2011		06-Oct-17	0	1	111							1.11E+02		
7246	2012		06-Oct-17	0	1	122							1.22E+02		
7247	2013		06-Oct-17	0	0.5	51							1.0E+02		
7248	2014		06-Oct-17	0	0.5	75							1.5E+02		
7249	2015		10-Oct-17	0	0.5	106							2.12E+02		
7250	2016		06-Oct-17	-1	2	58							2.9E+02		
7251	2017		06-Oct-17	-1	2	52							2.6E+02		
7252	2018		06-Oct-17	-1	2	92							4.6E+02		
7253	2019		06-Oct-17	-2	0.5	49							9.8E+03		
7254	2020		06-Oct-17	-2	0.5	56							1.1E+04		
7255	2021		06-Oct-17	-2	0.5	66							1.3E+04		
7256	2022		06-Oct-17	0	0.5	83							1.7E+02		
7257	2023		06-Oct-17	0	0.5	61							1.2E+02		
7258	2024		06-Oct-17	0	0.5	32							6.4E+01		
7259	2025		06-Oct-17	0	0.5	62							1.2E+02		
7260	2026		06-Oct-17	0	0.5	65							1.3E+02		
7261	2027		06-Oct-17	0	0.5	56							1.1E+02		
7292		Batch 1	05-Oct-17	-7	1	131							1.31E+09		
Q1728		Method Blank	05-Oct-17	0	2	0							0E+00		
Q1731		Method Blank	06-Oct-17	0	2	0							0E+00		
Q1745		Method Blank	10-Oct-17	0	2	0							0E+00		

CALCULATED BY: S. Verhoeven	POSITION: Lab Manager	MANAGER APPROVAL: C. Odegaard (Technical Manager)	
SIGNATURE: S. Verh		SIGNATURE: Cyd	DATE: 11-Oct-17

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ilac-MRA CALA Testing Accreditation No. A 2914

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London, ON N6E 1P5

FINAL RESULTS FORM

Ph # 519-681-0571
Fax# 519-681-7150

GAPLAB Environmental Microbiology

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PAGE #:	3 of 3	**RECEIVED BY:**	K. Shuty
		ANALYSIS START:	05-Oct-17
		ANALYSIS FINISH:	11-Oct-17

REPORT TO:	Trojan Technologies		
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TEST RESULTS *-relate only to the samples submitted and the analyses requested.*

Accredited Method Code: BACTPHAGE-0001

LAB #	SENDERS #	CB DOSE (mJ/cm^2)	ASSAY DATE	MS2 - Replicate 1			MS2 - Replicate 2			MS2 - Replicate 3			Calculated Concentration (pfu/mL)		
				Dilution log	Aliquot (mL)	PFU	Dilution log	Aliquot (mL)	PFU	Dilution log	Aliqot (mL)	PFU	Replicate 1	Replicate 2	Replicate 3
7293	CB Oct 3	0.00	06-Oct-17	-2	1	109	-2	1	90				1.09E+04	9.0E+03	
7294	CB Oct 3	9.95	06-Oct-17	-1	0.5	139							2.78E+03		
7295	CB Oct 3	19.99	06-Oct-17	-1	1	62							6.2E+02		
7296	CB Oct 3	40.11	06-Oct-17	0	1	61							6.1E+01		
7297	CB Oct 3	60.05	06-Oct-17	0	5	74							1.5E+01		
7298	CB Oct 3	80.24	06-Oct-17	0	10	26							2.6E+00		
7299	CB Oct 3	0.00	06-Oct-17	-2	1	99	-2	1	88				9.9E+03	8.8E+03	
7300	CB Oct 3	9.98	06-Oct-17	-1	0.5	96							1.9E+03		
7301	CB Oct 3	19.87	06-Oct-17	-1	1	71							7.1E+02		
7413	CB Oct 3	40.02	06-Oct-17	0	1	90							9.0E+01		
7414	CB Oct 3	59.99	06-Oct-17	0	5	82							1.6E+01		
7415	CB Oct 3	80.10	06-Oct-17	0	10	33							3.3E+00		
7302	CB Oct 3 55%	0.00	06-Oct-17	-2	0.5	134	-2	0.5	140				2.68E+04	2.80E+04	
7303	CB Oct 3 55%	9.99	06-Oct-17	-2	2	121							6.05E+03		
7304	CB Oct 3 55%	19.78	06-Oct-17	-1	0.5	100							2.00E+03		
7305	CB Oct 3 55%	40.36	10-Oct-17	0	0.5	90							1.8E+02		
7306	CB Oct 3 55%	59.83	06-Oct-17	0	2	107							5.35E+01		
7307	CB Oct 3 55%	80.10	10-Oct-17	0	10	65							6.5E+00		
7308	CB Oct 3 55%	0.00	06-Oct-17	-2	0.5	125	-2	0.5	148				2.50E+04	2.96E+04	
7309	CB Oct 3 55%	10.02	06-Oct-17	-2	2	156							7.80E+03		
7310	CB Oct 3 55%	19.79	06-Oct-17	-1	0.5	129							2.58E+03		
7311	CB Oct 3 55%	40.20	06-Oct-17	0	0.5	138							2.76E+02		
7312	CB Oct 3 55%	60.09	06-Oct-17	0	5	221							4.42E+01		
7313	CB Oct 3 55%	80.32	06-Oct-17	0	10	117							1.17E+01		

CALCULATED BY: S. Verhoeven **POSITION:** Lab Manager **MANAGER APPROVAL:** C. Odegaard (Technical Manager)

SIGNATURE: S. Verh **SIGNATURE:** Cgd **DATE:** 11-Oct-17

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< = Less Than; > = Greater Than; TNTC = Too Numerous To Count; OBSC = Obscured; NR = No Result; LA = Laboratory Accident
CFU = Colony Forming Unit; PFU = Plaque Forming Unit; MF = Membrane Filtration; MPN = Most Probable Number; SP = Spread Plate

ilac-MRA CALA Testing Accreditation No. A 2914