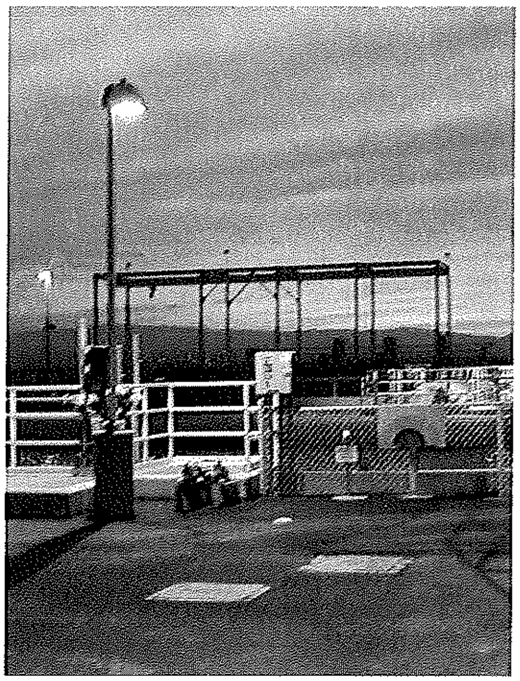
Stantec

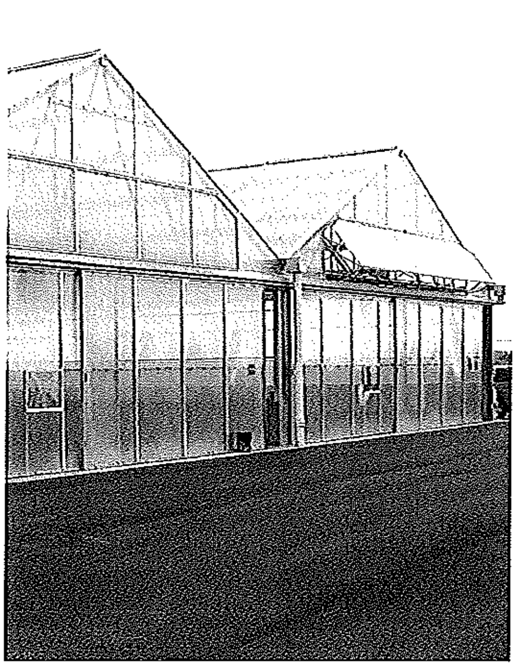
The Town of Discovery Bay Community Services District

Wastewater Treatment Plant Master Plan

FINAL DRAFT

October 2011





Prepared for
The Town of Discovery Bay Community Services District

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Section 1
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 2.1 million gallons per day (Mgal/d). However, the true capacity of a wastewater treatment plant is dependent upon the strength of the wastewater and on the variability of flows and loads (loads are determined as the flow multiplied by the concentrations of key pollutants) and may be different than the permitted capacity.

Wastewater treatment facilities are not systems that are constructed and then remain unchanged for many years. Rather, wastewater treatment facilities must evolve over time in response not only to changing (generally increasing) flows and loads but also to changing regulations that govern the quality and methods of disposal of the final liquid effluent and of the residual solids (mainly sludge or biosolids) that are produced within the treatment plant. Additionally, as new technologies are developed, opportunities to implement more efficient and/or environmentally acceptable solutions may arise. Accordingly, all wastewater treatment plant owners must continually assess their facilities in the light of current and expected conditions and constraints and make changes to those facilities when appropriate.

This Wastewater Master Plan is intended to provide an overall current assessment of the TDBCSD wastewater treatment and disposal facilities and a road map for making improvements to the facilities as flows and loads continue to increase through projected buildout of the community. Also included is an assessment of the sewage collection system pump stations and of the supervisory control and data acquisition (SCADA) system that the District uses to monitor the remote pump stations as well as the wastewater treatment facilities. This Master Plan does not include an assessment of the sewage collection system pipelines throughout the community, which are the subject of a separate investigation.

This Master Plan 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 19.

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. An inventory of the existing collection system pumping stations, including capacities, types of pumps, year of installation, and needed improvements, is 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: Waste Discharge and Treatment Requirements. The requirements of the National Pollution Discharge Elimination System (NPDES) permit that governs discharges from the wastewater treatment plant are reviewed and the plant's ability to meet those requirements is assessed.

Section 9: Influent Pump Station. The capacity, operational issues, and recommended improvements for this key pumping facility are addressed.

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 capacities of these facilities under various normal and abnormal operating conditions are assessed and alternatives for expansion are investigated.

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 disinfection facilities. The capacity of this pumping system and expansion requirements are assessed based on continued pumping to the disinfection facilities and based on pumping to a potential future filtration system.

Section 13: Tertiary Filtration. As regulations for wastewater discharge become more stringent and/or to allow higher-level reuse of the wastewater effluent, it may become necessary or beneficial to filter the secondary effluent prior to disinfection. Alternative filtration systems that potentially could be implemented are evaluated in this section.

Section 14: UV Disinfection. Ultraviolet (UV) radiation is currently used for disinfection of the wastewater effluent. The capacity and required improvements to this system are investigated.

Section 15: Salinity Reduction. The salinity of water supplies in California is a major concern. Therefore, the salinity of wastewater effluents is highly scrutinized and new permit requirements are being implemented for monitoring and control of wastewater salinity. Although specific treatment to remove dissolved salts is not currently required, the potential for such treatment and the costs and issues related thereto are assessed in Section 15.

Section 16: Emergency Storage. The wastewater treatment plant currently includes an unused earthen basin that was part of a previous treatment system. The potential use of this basin for emergency storage is investigated in Section 16.

Section 17: Solids Handling. This section includes an evaluation of facilities for the handling of residual solids (sludge or biosolids) developed within the wastewater treatment plant. Existing facilities are described and recommended improvements are presented.

Section 18: SCADA System. The existing SCADA system was evaluated and improvements recommended prior to this Master Plan investigation. For this study, the previous work was reviewed and alternative recommendations were developed.

Section 19: 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.

Section 2

Executive Summary

Presented below is a section-by-section summary of the key investigations and findings included Sections 3 through 20 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

Development	Number
Residential, Homes	
Approved, But Not Yet Built	600
Undeveloped Lots (Discovery Bay Proper)	55
Pantages	300 [(a)]
Newport Point	70
Villages (Hoffman)	80
Golf Course	13
5-Acre Lots	5
Total	**1,123**
Office and Business Park, Acres	
Bixler Business Park	45
Marsh Creek Office	45
Total	**90**
Commercial, Acres	
Highway 4	5
Discovery Bay / Willow Lake	5
Total	**10**

(a) A portion of this property is outside of the current TDBCSD service area boundary.

2.2 Section 4 – Collection System Pump Stations

There are 15 sewage lift stations within the TDBCSD sewage collection system. Pertinent data on the existing facilities and required improvements are shown in Table 2-2.

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	-	1,3	55,000
C	Beaver Lane and Willow Lake Road	Self Prime	2	300	5	80.s	-	2009	1	35,000
D	Discovery Bay Blvd Near Beaver Lane	Self Prime	2	300	5	70's	2008	-	1, 3	55,000
E	Discovery Bay Blvd and Cabrillo Point	Self Prime	2	680	10	80's	2008	-	1, 3	75,000
F	Willow Lake Road and River Lake Road	Non-Clog, Dry Pit	2	760	10	70's	-	2008 / 9	1, 2, 3	115,000
G	Willow Lake Road and Starboard Drive	Submersible	2	225	3	80's	-	2009	1	35,000
H	Marina Road and Cherry Hills Drive	Submersible	2	225	3	90's	-	-	1,2	55,000
J	Clipper Drive and Windward Point	Submersible	2	690	15	90's	-	-	1,2	95,000
R	Newport Drive and Beacon Place	Submersible	2	170	3	90's	-	2008 / 9	1	35,000
S	Fog Horn Way and Tiller Court	Submersible	2	250	15	1994	-	2009 (1 Pmp)	1,2	55,000
Newport Lift Station	Newport Drive	Submersible	4	1200	100	2002	2006	2011 (2 Pmp)	4	10,000
Lakeshore at Village II	Yosemite Court	Submersible	3	1100	29	2004	-	2009 (2 Pmp)	4	10,000
The Lakes No. 1 at Village III	Fern Ridge Circle	Submersible	3	1000	45	2004	-	2009 (1 Pmp)	4	10,000
The Lakes No. 2 at Village IV	Pinehollow Circle	Submersible	3	450	7.5	2005	-	-	4	10,000
Bixler Rd (School)	Bixler Road North end	Submersible	2	110	3	2008	-	-	None	0
Total Cost										650,000

(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 $ 35,000 for small wet wells / $ 55,000 for large wet wells)
2 Replace or Rehabilitate pumps and valves (Cost $ 20,000 for small pump stations / $ 40,000 for large pump stations)
3 Replace electrical feed panels and field instruments (Cost $ 20,000)
4 General Rehabiliatation of valves & pumps (Cost $ 10,000)

(b) Based on work by District staff (except wet well coatings and pump rehabilitation) with minor engineering advice. First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

2.3 Section 5 – Wastewater Flows and Loads

In June 2008, ECO:LOGIC Engineering, working with Herwit Engineering, submitted a draft of Technical Memorandum No. 1 (TM1) on Design Flows and Loads for the TDBCSD Wastewater Treatment Plant (WWTP). That document, which was based on Data from January 2004 through July 2007 is included herewith as Appendix A. For this Master Plan, data from January 2009 through May 2010 and from a special intensive monitoring effort completed in July 2011 (TM2 in Appendix C) were evaluated also. Because of substantial discrepancies in the data, the existing average influent biochemical oxygen demand (BOD) concentration adopted for use in this Master Plan is based largely on generally accepted typical per-capita BOD loads. Total suspended solids (TSS) and total Kjeldahl nitrogen (TKN) concentrations are based on appropriate ratios to BOD. The historical data and adopted average constituent concentrations were used to establish existing flows and loads, including peaking factors.

Future flows and loads were projected by estimating the values for future development areas and adding them to the existing flows and loads. The existing, incremental and future flows and loads are summarized 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 effluent is recombined at the Secondary Effluent Lift Station within Plant 2, from which it is pumped through a flow metering flume and UV disinfection facilities. The disinfected effluent is then pumped to Old River by the Export Pump Station.

Plant flow schematics, hydraulic profiles and design criteria are presented in Figures 6-1 through 6-3 in Section 6.

The plant is generally successful in meeting most of its permitted effluent limitations most of the time. However, there have been periodic violations of TSS and total coliform limits. Also, in the year 2010, the plant exceeded its annual average limit for effluent electrical conductivity.

2.5 Section 7 – Plant Hydraulic Analysis

To assess the ability of pumping and conveyance facilities in the plant to handle projected peak flows, a spreadsheet-based hydraulic model of the entire treatment plant (Plants 1 and 2) was developed. 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.

Based on the analysis of various future peak flow scenarios, it was determined that the existing plant hydraulic features can accommodate future peak flows with suitable modifications to the main pumping facilities, including the Influent Pump Station, the Secondary Effluent Lift Station, and the Export Pump Station.

Table 2-3
Summary of Existing and Future Flows and Loads

Parameter	Existing	Incremental	Future
Flow, Mgal/d			
Average Dry Weather Flow (ADWF)	1.75	0.55	2.30
Average Annual Flow (AAF)	1.80	0.57	2.37
Average Day Maximum Monthly Flow (ADMMF)	1.98	0.63	2.61
Peak Day Flow (PDF)	3.60	1.14	4.74
Peak Hour Flow (PHF) (a)	5.40	1.71	7.11
Average Constituent Concentrations, mg/L (b)			
BOD	200	200	200
TSS (c)	200	200	200
TKN (d)	40	40	40
Average Annual Load (AAL), lb/d			
BOD	3,002	951	3,953
TSS (c)	3,002	951	3,953
TKN (d)	600	190	791
Average Day Maximum Monthly Load (ADMML), lb/d			
BOD	3,903	1,236	5,139
TSS (c)	3,903	1,236	5,139
TKN (d)	781	247	1,028

(a) Allowance at 3 x AAF. Confirm with future monitoring.
(b) AAF combined with AAL.
(c) Based on 1.0 x BOD. Confirm with future monitoring.
(d) Based on 0.2 x BOD. Confirm with future monitoring.

2.6 Section 8 – 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. Key permit requirements and corresponding existing plant performance and compliance strategies are summarized in Table 2-4.

Looking forward, the key compliance issues that must be resolved are those for total coliform and electrical conductivity, which are considered further in Sections 13 and 15, respectively.

2.7 Section 9 – Influent Pump Station

The Influent Pump Station, which is located within Plant 1, currently includes one large pump and one small pump for Plant 1 and two small pumps and one large pump for Plant 2. The total reliable capacity of this pump station is 4.8 Mgal/d.

Table 2-4
Key NPDES Permit Requirements, Plant Performance and Compliance Strategy

Parameter	Units	Effluent Limits [(a)]	Existing Plant Performance	Compliance Strategy
Flow	Mgal/d	2.1 [(b)]	Generally compliant.	Expand plant and revise permit before limit is reached.
BOD	mg/L	20/40/50	Generally compliant.	Continue current performance or better.
TSS	mg/L	30/40/50	Occasional noncompliance.	Resolve the problem of influent screen bypassing that can lead to clogging of secondary clarifier sludge removal systems and RAS pumps. Operate and maintain the secondary process and design improvements to provide good performance, in general. As a last resort, utilize new provisions for temporary diversion of poor-quality effluent to the sludge lagoons.
pH	Units	6.5 to 8.5 [(c)]	Generally compliant.	Continue current performance or better.
Copper	µg/L	50/–/70	Generally compliant.	Continue current performance or better.
Nitrate-N	mg/L	73/–/126	Generally compliant.	Continue current performance or better.
Ammonia-N	mg/L	10/–/30	Generally compliant	Continue current performance or better.
Total Coliform	MPN/100 mL	23, 240 [(d)]	Occasional noncompliance, prior to recent improvements (2010).	The UV disinfection system has been improved and provisions have been made to divert poor quality effluent to storage. If these improvements are not adequate, effluent filtration could be required.
Electrical Conductivity	µmhos/cm	2,100 [(e)] [(f)]	Noncompliant in 2010	Minimize salinity through source control and minimize or prevent salinity increase during treatment. As a last resort, if required in the future, provide treatment to remove salinity.
Iron (Total Recoverable)	µg/L	300 [(e)]	Generally compliant	Continue current performance or better.
Aluminum (Total Recoverable)	µg/L	200 [(e)]	Generally compliant	Continue current performance or better.

(a) Unless indicated otherwise, limits are Average Monthly/Average Weekly/Maximum Daily.
(b) This is specified as an "Average Daily" limit in the permit. However, the permit indicates that compliance will be assessed based on the "Average Dry Weather Flow", meaning the average flow over three dry weather months.
(c) Range is based on instantaneous minimum and instantaneous maximum.
(d) 23 weekly median, 240 not to be exceeded more than once in 30 days.
(e) Annual average.
(f) The limit decreases to 1,000 µmhos/cm if the District fails to implement a Salinity Plan.

Current issues with the Influent Pump Station include: 1) pump ragging, 2) lack of flow splitting controls, 3) lack of sump mixing that results in different wastewater characteristics for Plants 1 and 2, and 4) inability to take the pump station out of service for needed repairs.

In the future, this pump station must be upgraded to allow pumping peak flows of 2.49 and 4.62 Mgal/d to Plants 1 and 2 respectively.

To mitigate the issue of pump ragging, the option of screening ahead of the pumps was considered, but would not be cost-effective. Instead, pumps that are designed to minimize ragging should be used. Three alternative pump types were considered, including Flygt pumps with N-Series impellers, screw centrifugal pumps, and chopper pumps. Selection of which type of pump to use should be made during final design based on site visits to other facilities with these types of pumps and detailed considerations of pump turndown capabilities.

Recommended improvements to the Influent Pump Station include structural rehabilitation, replacement of all pumps, some piping modifications, installation of a sump mixer and improved flow splitting controls. The total estimated capital cost for these improvements is about $1 million (for cost breakdown, see Table 9-1 in Section 9).

Pump Station W within Plant 1 was the original Influent Pump Station to Plant 1. Pump Station W can be re-activated as a backup to the new Influent Pump Station (allowing it to be taken out of service for repairs) and also to allow pumping raw sewage to an emergency storage basin (see Section 16) within Plant 1. The estimated capital cost for re-activating Pump Station w is $378,000 (for cost breakdown, see Table 9-2 in Section 9).

2.8 Section 10 - Headworks

There are two nearly identical headworks facilities, one located at Plant 1 and one located at Plant 2. Each headworks includes a Parshall flume for influent flow measurement and a mechanical screen to remove rags and other debris and large solids from the sewage flow. Each screen is capable of passing a flow of 6.2 Mgal/d, which exceeds future capacity requirements at the two plants. Therefore, no expansion is required.

At the Plant 2 headworks, there is an automatic sampler that is used to characterize the wastewater into both plants (assuming they would be the same). The sampler does not work properly because its intake tube is located ahead of the screen and gets covered with rags. To mitigate this problem, a new pumped mixing system should be installed to mix the channel both before and after the screen and to provide a screened and well-mixed sample to the automatic sampler. The estimated cost for these improvements, if accomplished by District staff is $10,000.

2.9 Section 11 – Secondary Treatment Facilities

The existing secondary treatment system includes one oxidation ditch and four clarifiers at each plant. Additionally there are return activated sludge (RAS) and waste activated sludge (WAS) pumping systems at each plant. Design criteria for these facilities are summarized in Tables 11-1 and 11-2 in Section 11.

Investigations were conducted to assess the capacities of each plant separately and of both plants combined under various conditions of operation. A summary of the capacity assessment results is presented in Table 2-5. It should be noted that all of the capacities indicated in Table 2-5 have been normalized to the corresponding average annual flow (AAF).

The key result from the capacity evaluations is that the total combined capacity of Plants 1 and 2 is estimated to be about 2.0 Mgal/d AAF (based on Scenarios 1 and 2). Since the current AAF for the combined plant is 1.8 Mgal/d, this analysis would suggest that the plant is currently operating at about 90 percent capacity. However, the ability of the brush rotors to support the 2.0 Mgal/d capacity is marginal. At least one standby rotor should be added to each ditch.

The purpose of Scenarios 5 through 8 was to assess the ability of the plants to operate with key units out of service for maintenance or repairs during warm and dry weather conditions. The combined capacity of the two plants with any clarifier out of service was determined to be at least 2.65 Mgal/d (2.81 Mgal/d with RAS upgrade). Therefore, taking a clarifier out of service under warm and dry weather conditions would not be a problem, even with average annual flows in excess of future requirements (2.37 Mgal/d). However, similar to the condition mentioned above, additional brush rotor capacity would be needed. Taking an oxidation ditch out of service is much more problematical than taking a clarifier out of service. One of the reasons this is so is that taking an oxidation ditch out of service also results in taking both associated clarifiers out of service. Even at current flows and loads, it would not be reasonably possible to take an oxidation ditch and its associated clarifiers out of service at any time of year.

Two alternatives were considered for increasing the capacity of the secondary treatment system as needed to accommodate the projected future flows and loads: 1) addition of a third oxidation ditch (with or without additional clarifiers) and 2) use of Salsnes filters.

Under the alternative of adding a third oxidation ditch, evaluations were completed to determine whether zero, one, or two clarifiers should be added with the new oxidation ditch. It was determined that one new clarifier should be added, as this was the minimum requirement to allow a clarifier in either plant to be taken out of service during peak wet weather flow conditions. With the third ditch added, it would also be possible to take any oxidation ditch out of service during warm and dry weather conditions. The estimated cost for the secondary treatment system expansion under this alternative is shown in Table 2-6.

Salsnes filters are mechanical belt filtering devices that can be used to remove a substantial portion of the influent TSS and a lesser amount of BOD from the influent wastewater before it reaches the oxidation ditches, thereby extending the capacity of the ditches. Based on actual pilot testing at the TDBCSD WWTP, it is estimated that the Salsnes filter could remove 65 percent of the TSS and at least 10 percent of the BOD (BOD results were highly variable). The solids removed in the Salsnes filter would be mechanically compacted to a solids content of about 40 percent and then hauled to a landfill.

It was determined that the use of Salsnes filters would not eliminate the need to build a third oxidation ditch and would not be cost-effective. Therefore, it is recommended that future plant expansion be based on Alternative 1.

Table 2-5
Secondary Treatment System Capacity Assessment Results

Scenario	Description	Mixed Liquor Temp, °C	MCRT, days	AAF [a] Capac., Mgal/d	Max Month MLSS, mg/L	Max Month WAS, lb/d	Max Day SOR [b] Per Oxidation Ditch, lb/d		Max Hour SOR [b] Per Oxidation Ditch, lb/d	
							DO = 2 No Denit.	DO = 1 50% Denit.	DO = 2 No Denit.	DO = 1 50% Denit.
1	Existing Plant 1 (RAS = 0.8 Mgal/d per Clarifier)	15	10	1.03	3,000	2,500	9,900	7,500	12,800	9,600
2	Existing Plant 2 (RAS = 0.6 Mgal/d per Clarifier)	15	10	0.97	2,800	2,400	9,300	7,000	12,000	9,000
3	Both Plants Together with RAS Upgrade to 1 Mgal/d per Clarifier	15	10	2.13	3,100	5,200	10,300	7,800	13,200	9,900
4	Both Plants Together with RAS Upgrade to 1 Mgal/d per Clarifier	15	8	2.37	2,900	6,000	11,400	8,600	14,700	11,000
5	Existing Plant 1 (RAS = 0.8 Mgal/d per Clarifier) with One Clarifier Out of Service During Dry Weather Flows	20	8	1.18	2,700	2,900	11,400	8,600	14,600	10,900
6	Existing Plant 2 (RAS = 0.6 Mgal/d per Clarifier) During Dry Weather Flows	20	8	1.47	3,400	3,600	14,100	10,700	18,200	13,600
7	Either Plant with RAS Upgrade to 1 Mgal/d per Clarifier with One Clarifier Out of Service During Dry Weather Flows	20	8	1.22	2,800	3,000	11,700	8,800	15,100	11,300
8	Either Plant with RAS Upgrade to 1 Mgal/d per Clarifier During Dry Weather Flows	20	8	1.59	3,700	3,900	15,300	11,600	19,800	14,800

(a) AAF = Average Annual Flow
(b) SOR = Standard Oxygen Requirement

Table 2-6
Secondary Treatment System Expansion In-Kind Cost Estimate

Item	Cost, $ Millions (a)
New Splitter Box at Plant 2 Headworks	0.05
New Oxidation Ditch at Plant 2	1.10
New Clarifier Splitter Box at Plant 2	0.05
New Clarifier at Plant 2	0.65
New RAS Pump Station at Plant 2	0.25
Replace Existing Plant 2 RAS Pumps	0.12
Standby Floating Brush Aerators in Existing Ditches	0.18
Subtotal 1	2.40
Electrical @ 25% of Subtotal 1	0.60
Site Piping @ 10% of Subtotal 1	0.24
Sitework @ 5% of Subtotal 1	0.12
Subtotal 2	3.36
Contingencies @ 20% of Subtotal 2	0.67
Subtotal 3	4.03
General Conditions, Overhead and Profit @ 20% of Subtotal 3	0.81
Total Construction Cost	4.84
Engineering, Admin. and Environmental @ 25%	1.21
Total Capital Cost	6.05

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

2.10 Section 12 – Secondary Effluent Lift Station

The Secondary Effluent Lift Station currently pumps the combined secondary effluents of Plants 1 and 2 to the Parshall flume ahead of the UV disinfection system. If filters are not added to the plant, this will remain the condition in the future. In this case, the existing pumps may be marginally adequate for the future flows, however, some over-speeding using the variable frequency drives may be required.

If filters are added to the plant, the discharge head for the Secondary Effluent Lift Station will increase for pumping to the filters. In this case, impellers and motors would have to be changed on the existing pumps and some over-speeding using new variable frequency drives would be required. The total capital cost of required improvements is $250,000.

2.11 Section 13 – Tertiary Filtration

The wastewater treatment plant does not currently include effluent filters. However, filters may be needed to improve the performance of the UV disinfection system. Also, filters may be needed in the future to allow reclamation reuse or to meet future more stringent effluent limitations for discharge to Old River.

Three filtration technologies were evaluated, including: 1) continuous backwash sand filters, 2) cloth disk filters, and 3) stainless steel micromesh disk filters. All three options were considered with and without upstream flow equalization. The results of an alternative cost analysis are shown in Table 2-7.

Although the continuous backwash sand filter has a slightly higher cost than the stainless steel micromesh alternative, the continuous backwash sand filter is recommended for implementation because it has an extensive and favorable track record ahead of UV disinfection. The stainless steel micromesh filter is relatively new and unproven, particularly ahead of UV filtration. Flow equalization is recommended and can be justified by savings in filter costs alone. Furthermore, flow equalization will result in substantial cost savings for UV filtration and final effluent pumping.

2.12 Section 14 – UV Disinfection

The existing UV disinfection system includes one channel with TrojanUV3000 equipment and one channel with TrojanUV3000Plus equipment. The capacities of these channels are indicated in Table 2-8. As indicated in the table, the combined reliable capacity of the two channels with one UV bank per channel out of service is estimated to be 4.1 Mgal/d without a safety factor and 3.4 Mgal/d with safety factors. Until on-site viral bioassay testing is completed to validate capacity, the use of safety factors is recommended. The capacities given can be compared to the existing peak day and peak hour flows of 3.6 and 5.4 Mgal/d, respectively.

The capacities indicated above are based on a secondary effluent turbidity generally under 10 NTU, with diversions to the sludge storage basins if the turbidity substantially exceeds 10 NTU. Diversions to the sludge storage basins should also be made to limit peak flows through the UV system; however, this would require modifications to the diversion system, which is currently not configured for peak flow trimming. Also, to realize the combined capacity of the two UV channels, weir modifications are required for flow splitting in proportion to capacity.

Three scenarios for future operation and possible improvement of the UV system were considered:

> <u>Scenario 1:</u> Continuation of existing conditions, including UV disinfection to meet a weekly median total coliform limit of 23 MPN/100 mL after secondary treatment.
>
> <u>Scenario 2:</u> UV disinfection to meet a weekly median total coliform limit of 23 MPN/100 mL, but with effluent filtration provided to improve UV system performance.
>
> <u>Scenario 3:</u> UV disinfection to meet a weekly median total coliform limit of 2.2 MPN/100 mL after effluent filtration. This scenario is based on the possible adoption of more stringent effluent limitations for discharge to Old River or for unrestricted reuse of the wastewater effluent for irrigation.

Table 2-7
Filtration Alternative Cost Analysis

Item	Cost for Indicated Alternative, $ [a]					
	Scenario 1 (With Flow Equalization)			Scenario 2 (Without Flow Equalization)		
	Continuous Backwash	Cloth Disk [b]	SST Mesh Disk	Continuous Backwash	Cloth Disk [b]	SST Mesh Disk
Capital Cost						
Equalization Basin, Piping, Valves and Controls	270,000	270,000	270,000	0	0	0
Concrete structures and canopy (if applicable)	250,000	210,000	180,000	310,000	210,000	225,000
Piping, metals, and ancillaries	330,000	375,000	340,000	440,000	375,000	452,000
Filter Equipment, Installed	1,251,000	1,796,000 [b]	975,000	1,552,000	1,796,000 [b]	1,065,000
Subtotal 1	2,101,000	2,651,000	1,765,000	2,302,000	2,381,000	1,742,000
Elect/Instrum, 25% of Subtotal 1, Unless Noted Otherwise	525,000	663,000	441,000	575,000	595,000	436,000
Sitework, 5% of Subtotal 1 Unless Noted Otherwise	105,000	133,000	88,000	115,000	119,000	87,000
Site Piping, 10% of Subtotal 1, Unless Noted Otherwise	210,000	265,000	177,000	230,000	238,000	174,000
Subtotal 2	2,941,000	3,712,000	2,471,000	3,222,000	3,333,000	2,439,000
General Conditions, Overhead and Profit, 20%	588,000	742,000	494,000	645,000	667,000	488,000
Subtotal 3	3,529,000	4,454,000	2,965,000	3,867,000	4,000,000	2,927,000
Contingencies, 20%	706,000	891,000	593,000	773,000	800,000	585,000
Total Construction Cost	4,235,000	5,345,000	3,558,000	4,640,000	4,800,000	3,512,000
Engineering and Administration, 25%	1,059,000	1,336,000	890,000	1,160,000	1,200,000	878,000
Total Capital Cost	5,294,000	6,681,000	4,448,000	5,800,000	6,000,000	4,390,000
Annual Costs						
Labor	9,360	9,360	9,360	10,920	10,920	10,920
Power	11,040	600	4,440	14,683	840	5,905
Chemicals	11,859	17,789	17,789	17,789	26,684	26,684
Maintenance Materials	3,500	5,200	6,500	5,000	6,500	8,645
Total Annual Cost	35,759	32,949	38,089	48,392	44,944	52,154
Present Worth Costs						
Present Worth of Annual Costs	532,000	490,000	567,000	720,000	669,000	776,000
Total Present Worth Cost	5,826,000	7,171,000	5,015,000	6,520,000	6,669,000	5,166,000

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9000.
(b) Cloth-Disk Filter sizes are same for Scenario 1 (with EQ) and Scenario 2 (without EQ).

Table 2-8
Existing UV System Capacity

Condition	Peak Flow Capacity, Mgal/d [a]		
	TrojanUV3000	TrojanUV3000Plus	Total
All Banks in Service [b] [c]	1.3	4.8	6.1
One Bank in Each Channel Off-Line [c]	0.9	3.2	4.1
One Bank in Each Channel Off-Line, with Dose Safety Factor [d]	0.6	2.8	3.4

(a) Capacities calculated based on UV Dose = 80 mJ/cm^2 (before safety factor), UV Transmittance = 55%, and total coliform = 23 MPN/100 mL. In order to realize these capacities, the turbidity of the secondary effluent should generally be less than 10 NTU (see discussion in Section 14.2).

(b) Total number of banks is 3 for UV3000 and 4 for UV3000Plus.

(c) No safety factor.

(d) Dose safety factor for UV system performance variability = 1.25 for UV3000 and 1.1 for UV3000Plus

To provide reliable disinfection with future flows, both Scenarios 1 and 3 would require conversion of the existing UV3000 channel to a UV3000Plus system at an estimated capital cost of $1.2 million. No improvements to the existing system would be needed for Scenario 2, other than the flow splitting provisions previously mentioned.

It must be noted that reliable UV disinfection without effluent filtration under Scenario 1 may not be possible. The operation and performance of the existing system must be observed for an extended period of time before a conclusion can be reached on this matter. Of particular concern are the frequency and duration of diversions to the sludge storage lagoons.

2.13 Section 15 – Salinity Reduction

Reverse osmosis (RO) as a potential treatment process for removing salinity is investigated in Section 15.

To meet an effluent electrical conductivity goal of 1,000 µmhos/cm, approximately 70 percent of the filtered effluent from the WWTP would have to be routed through a sidestream treatment system including membrane filtration (MF) followed by RO. The concentrated reject water from the RO process would be further concentrated using a Vibratory Shear Enhanced Process (VSEP). The permeate from the RO and VSEP systems would be blended with the filtered effluent that was not treated for salinity removal.

The salinity treatment system would result in a final concentrated reject (brine) flow of about 45,000 gallons per day (about 2% of the total WWTP influent flow) at buildout. Since TDBCSD is remote from the coast, an ocean outfall pipeline would not be practical. Evaporation ponds would require extensive land area and would pose significant ecological risks. No practical brine handling alternative is currently known and it is beyond the scope of this Master Plan to investigate this issue further. For the purposes of this investigation, brine handling costs were developed based on hauling the brine to the East Bay Municipal Utility District for disposal through their outfall.

Estimated capital and annual costs for the MF-RO-VSEP treatment system and brine disposal are shown in Table 2-9. Because of the high costs involved, high energy usage, and other environmental impacts, this type of treatment would only be used as a last resort and if mandated by the State. Before consideration of implementing an MF-RO-VSEP system, all reasonable efforts to control the salinity of the wastewater influent through source control and/or use of alternative water supplies should be investigated.

Table 2-9
MF-RO-VSEP Cost Summary

Item	Cost, $M [(a)]
Capital Costs [(b)]	
MF	4.0
RO	6.8
VSEP	4.9
Total	15.7
Annual Costs	
MF	0.1
RO	0.43
VSEP	0.25
Brine Hauling and Disposal	1.34
Total	2.12

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9000.
(b) Including construction of all required facilities, contingency allowance, engineering and administration.

2.14 Section 16 – Emergency Storage

Within the Plant 1 site, there is an existing 5 Mgal earthen basin that is available for use as an emergency storage basin, but is currently not being used because of lack of permanent pumping and conveyance facilities for filling and draining the basin.

As developed in Section 9, Pump Station W can be re-activated and used to backup the Influent Pump Station or to divert influent wastewater to the emergency storage basin. A new return pump system would be required for draining the basin.

A cost estimate for the improvements necessary to make the emergency storage basin available for use are shown in Table 2-10.

2.15 Section 17 – Wetlands Treatment Potential

In 2007, TDBCSD implemented a wetlands demonstration project to investigate the removal of metals, particularly copper, which was a major issue at that time. The wetlands proved to be effective in accomplishing greater than 90 percent removal of soluble copper. Since that time, however, alternative methods for compliance with water quality objectives for copper have been recognized, eliminating the need for treatment to remove copper.

Table 2-10
Cost Estimate for Emergency Storage Improvements

Item	Cost, $1000s (a)
Re-Grade Basin Bottom and Provide Concrete Pump Intake Sump	30
Self Priming Return Pump System	35
Piping and Valves	30
Misc. Site Improvements	10
Electical and Instrumentation	30
Subtotal 1	135
Contingencies @ 20% of Subtotal 1	27
Subtotal 2	162
General Conditions, Overhead and Profit @ 20% of Subtotal 2	32
Total Construction Cost	194
Engineering, Admin. and Environmental @ 25%	49
Total Capital Cost	243

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

Designed treatment wetlands (DTWs) may have potential for meeting possible future requirements for metals and refractory organics. Also, the possibility of salinity reduction through DTWs could be investigated. Full-scale wetlands have the potential of being a community asset for aesthetic reasons and for providing wildlife habitat as well as for wastewater treatment. Therefore, although there a no current plans to use wetlands, the demonstration wetlands should be retained for possible future use, unless the land area is critically needed for other uses.

2.16 Section 18 – Solids Handling

The solids handling facilities consist of 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), a single belt press dewatering facility, and two active solar sludge dryers.

Sludge dewatering and drying occur mostly during the summer, when the active solar dryers perform best. However, currently, the two active solar dryers cannot be used to their full potential in the summer because the upstream belt press cannot dewater enough sludge to match the capacity of the active solar dryers. During the winter, sludge is wasted directly to the sludge lagoons and no dewatering takes place.

When Plant 2 was constructed (2000 to 2002), the sludge then existing in a lagoon at Plant 1 was transferred to the lagoons at Plant 2. Since then additional sludge has been accumulated in the lagoons at Plant 2 due to winter storage practices and lack of adequate sludge dewatering and drying capacity to remove sludge from the lagoons in the summer. In

January 2007, it was determined that Lagoon No. 1 was full and Lagoon No. 2 was one-quarter full of sludge. Lagoon No. 1 remains full and the level of sludge in Lagoon 2 has not been determined since 2007.

Solids balance calculations were developed for both existing and future conditions. The amount of solids produced is dependent on the influent BOD and TSS loading to the plant. Table 2-11 presents the total solids produced for the facilities at current conditions and at the planned buildout of the facilities. The capacity of the active solar dryers and the number of solar dryers required are also shown in Table 2-11. As indicated in the table, even under existing conditions, three active solar dryers are needed, compared to two existing.

Table 2-11
Summary of Solids Production

Parameter	Existing	Future Buildout
Flow, Mgal/d		
Average Annual Flow (AAF)	1.80	2.37
Average Constituent Concentrations, mg/L		
BOD	200	200
TSS	200	200
TKN	40	40
Solids Wasting (WAS)		
Average Annual, lb/d	3,300	4,300
Maximimum Month, lb/d	4,400	5,800
Volatile Solids (VSS), %	80%	80%
Aerobic Digester and Sludge Lagoon Operation		
VSS detruciton, % (a)	30%	30%
Average Annual TSS Remaining, lb/d	2,500	3,300
Active Solar Dryers		
Annual Capacity per Dryer, lb/d (b)	950	950
Number of Dryers Required	2.6	3.5
Number of Dryers Recommended to Build	3.0	4.0

(a) VSS destruction based on 9 Day HRT in Aerobic Digester and one 1 year sludge storage in existing sludge lagoons.
(b) Capacity at 16% solids feed.

For future flows and loads, two new belt presses and two active solar dryers should be added. Construction of the recommended facilities can be phased. Phase 1 would include the belt presses and one of the active solar dryers. Phase 2 would involve construction of the fourth solar dryer. Cost estimates for Phases 1 and 2 are shown in Tables 2-12 and 2-13, respectively.

Table 2-12
Cost Estimate for Solids Handling Phase 1 Improvements

Item	Cost, $ [(a)]
Dewatering Building Improvements (2 Presses)	844,000
1 New Solar Dryer	1,150,000
Civil	140,000
Electrical and Instrumentation	450,000
Subtotal 1	2,584,000
Contingencies @ 20% of Subtotal 1	517,000
Subtotal 2	3,101,000
General Condition, Overhead and Profit @ 20% of Subtotal 2	620,000
Total Construction Cost	3,721,000
Engineering, Admin, and Environmental @ 25%	930,000
Total Capital Cost	4,651,000

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

Table 2-13
Cost Estimate for Solids Handling Phase 2 Improvements

Item	Cost, $ [(a)]
1 New Solar Dryer	900,000
Civil	30,000
Electrical and Instrumentation	200,000
Subtotal 1	1,130,000
Contingencies @ 20% of Subtotal 1	226,000
Subtotal 2	1,356,000
General Condition, Overhead and Profit @ 20% of Subtotal 2	271,000
Total Construction Cost	1,627,000
Engineering, Admin, and Environmental @ 25%	407,000
Total Capital Cost	2,034,000

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

2.17 Section 19 – SCADA System

The Town of Discovery Bay Community Services District owns and operates (including operation by contract) water supply, treatment and distribution systems and wastewater collection and treatment systems. Critical facilities associated with these systems are scattered throughout the District. To allow District staff and contract operators to monitor, log data from, receive alarms from and, in many cases, control the operation of the remote facilities from centralized locations, a supervisory control and data acquisition (SCADA) system is used. Of course, the District's water and wastewater facilities have evolved over many years and, therefore, the SCADA system hardware and software at the various sites range from old and obsolete to new and modern. In recent years, investigations have been undertaken to determine the best means for upgrading the SCADA system to provide the level of functionality and reliability desired by the District and its contract operators.

As part of this Master Plan, previous investigations and recommendations regarding the SCADA system were reviewed, a tour of the facilities was conducted, and revised recommendations were developed as follows:

1. Add a new redundant radio master RTU with a Modicon Unity based Programmable Automation Controller (PAC) at Wastewater Treatment Plant 2 as the new Master Data Concentrator.

2. Add the features desired to update the programs at the sewage lift station RTUs, including runtimes, number of starts, average run times and associated alarms as well as adding an analog level-based control to RTUs that do not have them (this item is similar to Veolia Projects 3 and 4, except that it does not require changing PLC hardware.)

3. Add a separate backup float / alarm system with appropriate intrinsic barriers to allow the lift stations to continue operations in auto if the level transmitter or PLC became inoperable.

4. Start a SCADA Replacement Design Project that will investigate the replacement of the obsolete Modicon 612 PLCs with a legacy migration plan to replace the PLCs in an orderly fashion starting at the most critical PLCs to the least critical. This will allow the District to schedule a multi-year capital plan, or if funds become available, accelerate the upgrade of more sites, as desired.

The estimated cost for all of the improvements indicated above, including eventual replacement of all the obsolete Modicon 612 PLCs (Item 4 above) is $350,000. However, as noted under Item 4, the recommendations have been developed to allow gradual replacement over several years, if desired by the District. Therefore, after establishing priorities, the District can budget portions of the work each year, as needed.

2.18 Section 20 – Summary of Improvements

A list of all the recommended improvements developed in this Master Plan is presented in Table 2-14. 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 five separate columns to distinguish those improvements that should be undertaken immediately, those that are critical and should be completed as soon as possible, those that are certain or likely to be required (but not immediate or critical), those that are reasonably possible, and those that are unlikely to be required.

A site plan indicating where the future improvements could be located is shown in Figure 20-1 in Section 20.

Table 2-14
Recommended Improvements

Item	Description	Rept. Sect.	Reason for Improvement	Trigger for Implementation	Possible Timing (a)			Budgetary Cost, $ (b)				
					Begin Design	Begin Const.	Begin Operation	Immediate Improvements	Critical Improvements	Other Certain or Likely Improvements	Reasonably Possible or Optional Improvements	Unlikely Improvements
1	Influent Pump Station Modifications, Upgrade	9	Mitigate Ragging, Increase Capacity, Change Flow Splitting	Desired for Improved Reliability. Needed with Plant Expansion	2012	2013	2014		1,044,000			
2	Re-Activate Pump Station W	9	Backup to Influent Pump Station and Use for Emergency Storage	Desired to Facilitate Influent PS Mod's. Needed if Emergency Storage is to be Provided.	2012	2012	2012	378,000				
3	Emergency Storage Facilities	16	Facilitate Possible Emergency Full or Partial Plant Shutdown	Desired for Overall Reliability. Provide When Funds Available.	TBD	TBD	TBD	243,000				
4	Splitter Box, Oxidation Ditch, Clarifier, and RAS Pumps at Plant 2 and Standby Aerators for Existing Oxidation Ditches	11	Facilitate Taking an Oxidation Ditch Out of Service and Plant Expansion	Splitter Box, Oxidation Ditch, and Standby Aerators Needed Now for Reliability. Clarifier and RAS Pumps Needed Before Average Annual Flow Exceeds 2.0 Mgal/d.	2012	2013	2014		6,050,000			
5	Secondary Effluent Pump Station Modifications	12	Increase Pumping Head to Filters	Needed with Effluent Filters	TBD	TBD	TBD				250,000	
6	Secondary Effluent Equalization (c)	13	Limit Peak Flows to Filters, UV and Export Pump Station	When Peak Flows to UV Cannot be Trimmed to Sludge Lagoons or When Filters Required	TBD	TBD	TBD			680,000		
7	Effluent Filtration (c)	13	UV Performance or More Strigent Requirements or Reclamation	Upon Determination of Need	TBD	TBD	TBD				4,614,000	
8	Revise UV Disinfection Weirs	14	Flow Split to UV Channels	Desired Now	2011	2012	2012	10,000				
9	Conduct UV Disinfection Viral Bioassay Tests	14	Verify Existing Capacity	Desired Now	2011	2012	2012	50,000				
10	Upgrade UV Disinfection	14	Plant Expansion or More Stringent Total Coliform Limits	When Peak Day Flow Exceeds Peak Flow Capacity of UV Disinfection System (d)	TBD	TBD	TBD			1,200,000		
11	Reverse Osmosis Facilities	15	Reduce Effluent Salinity, Last Resort	If Required by Regulation - Very Unlikely	TBD	TBD	TBD					15,700,000
12	Add Pump to Export Pump Station	7	Plant Expansion	When Peak Day Flow Exceeds 4.0 Mgal/d (e)	TBD	TBD	TBD			100,000		
13	Solids Improvements, Phase 1: One New Solar Dryers and 2 Belt Presses	18	Correct Current Capacity Deficiency	Needed Now to Process Stored Sludge and Prevent Further Storage	2011	2012	2012	4,651,000				
14	Solids Improvements, Phase 2: One New Solar Dryer	18	Plant Expansion	To Be Determined Based on Operational Experience with Phase 1 Solids Improvements	TBD	TBD	TBD			2,034,000		
15	Collection System Pump Station Improvements	4	Needed for Reliable Performance	When Funds Available	Various (f)	Various (f)	Various (f)		100,000	550,000		
16	SCADA Improvements	19	Improved Monitoring and Control	When Funds Available	Various (f)	Various (f)	Various (f)		100,000	250,000		
	Total							5,332,000	7,294,000	4,814,000	4,864,000	15,700,000

(a) Approximate timing recommendations, where applicable. TBD = To Be Determined.
(b) Total capital cost, including construction, contingencies, engineering, administration and environmental documentation, as applicable. First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.
(c) Total cost of $5,294,000 for equalization and filtration broken down to $680,000 for flow equalization and $4,614,000 for filters. Filter cost includes coagulation and flocculation.
(d) Peak flow capacity of UV disinfection system to be verified by viral bioassay testing. Capacity estimated at 3.4 to 4.1 Mgal/d. Existing peak day flow is 3.6 Mgal/d.
(e) Subject to confirmation of reliable capacity of Export Pump Station and possible increased capacity with pump over-speeding.
(f) Project can be phased over multiple years, based on priorities and available funding, to be determined by the District.

Section 3

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 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 Manager and are as shown in Table 3-1.

Table 3-1
Projected Growth within TDBCSD

Development	Number
Residential, Homes	
Approved, But Not Yet Built	600
Undeveloped Lots (Discovery Bay Proper)	55
Pantages	300 [(a)]
Newport Point	70
Villages (Hoffman)	80
Golf Course	13
5-Acre Lots	5
Total	**1,123**
Office and Business Park, Acres	
Bixler Business Park	45
Marsh Creek Office	45
Total	**90**
Commercial, Acres	
Highway 4	5
Discovery Bay / Willow Lake	5
Total	**10**

(a) A portion of this property is outside of the current TDBCSD service area boundary.

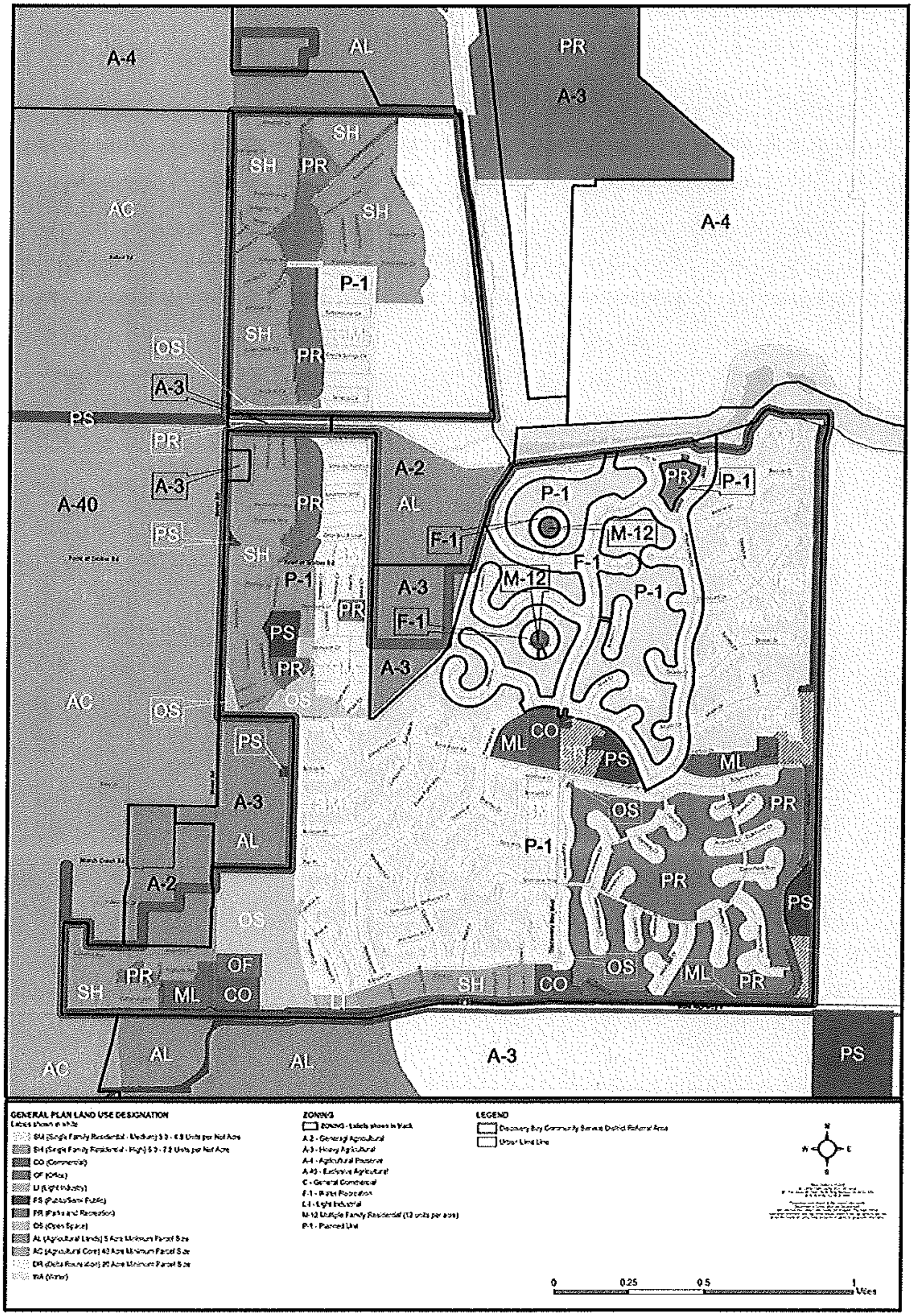


Figure 3-1
Discovery Bay Area Community Service District Referral Area

Stantec

Section 4

Collection System Pump Stations

There are fifteen sewage pumping stations within the Discovery Bay sewage collection system. The pump stations are listed in Table 4-1, which includes information on the type, number, and size of pumps. Also shown in the table are the year that the pump station was constructed, the year that pumps were last replaced or rehabilitated and currently recommended improvements, together with budgetary costs.

As indicated in Table 4-1, the total budgetary cost for all pump stations combined is $650,000, assuming that all work will be done by District Staff, except specialty work like wet well coatings and pump rehabilitation. Only minor consultation with the District Engineer is presumed. It is recommended that the District establish appropriate priorities for this work and then budget to accomplish certain portions of the work each year until completed.

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	-	1,3	55,000
C	Beaver Lane and Willow Lake Road	Self Prime	2	300	5	80,s	-	2009	1	35,000
D	Discovery Bay Blvd Near Beaver Lane	Self Prime	2	300	5	70's	2008	-	1, 3	55,000
E	Discovery Bay Blvd and Cabrillo Point	Self Prime	2	680	10	80's	2008	-	1, 3	75,000
F	Willow Lake Road and River Lake Road	Non-Clog, Dry Pit	2	760	10	70's	-	2008 / 9	1, 2, 3	115,000
G	Willow Lake Road and Starboard Drive	Submersible	2	225	3	80's	-	2009	1	35,000
H	Marina Road and Cherry Hills Drive	Submersible	2	225	3	90's	-	-	1,2	55,000
J	Clipper Drive and Windward Point	Submersible	2	690	15	90's	-	-	1,2	95,000
R	Newport Drive and Beacon Place	Submersible	2	170	3	90's	-	2008 / 9	1	35,000
S	Fog Horn Way and Tiller Court	Submersible	2	250	15	1994	-	2009 (1 Pmp)	1,2	55,000
Newport Lift Station	Newport Drive	Submersible	4	1200	100	2002	2006	2011 (2 Pmp)	4	10,000
Lakeshore at Village II	Yosemite Court	Submersible	3	1100	29	2004	-	2009 (2 Pmp)	4	10,000
The Lakes No. 1 at Village III	Fern Ridge Circle	Submersible	3	1000	45	2004	-	2009 (1 Pmp)	4	10,000
The Lakes No. 2 at Village IV	Pinehollow Circle	Submersible	3	450	7.5	2005	-	-	4	10,000
Bixler Rd (School)	Bixler Road North end	Submersible	2	110	3	2008	-	-	None	0
Total Cost										650,000

(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 $ 35,000 for small wet wells / $ 55,000 for large wet wells)
2 Replace or Rehabilitate pumps and valves (Cost $ 20,000 for small pump stations / $ 40,000 for large pump stations)
3 Replace electrical feed panels and field instruments (Cost $ 20,000)
4 General Rehabiliatation of valves & pumps (Cost $ 10,000)

(b) Based on work by District staff (except wet well coatings and pump rehabilitation) with minor engineering advice. First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

Section 5
Wastewater Flows and Loads

In this section, various investigations that have been completed to evaluate influent wastewater characteristics are discussed and used as the basis for establishing existing flows and loads. Future flows and loads are then determined based on existing criteria and allowances for future growth within the service area.

5.1 Technical Memorandum No. 1

In June 2008, ECO:LOGIC Engineering, working with Herwit Engineering, submitted a draft of Technical Memorandum No. 1 (TM1) on Design Flows and Loads for the Town of Discovery Bay Community Services District Wastewater Treatment Plant (TDBCSD WWTP). In that memorandum, routine plant data from January 2004 through July 2007 were analyzed for the purpose of establishing flows and loads existing in those years. Additionally, a special intensive monitoring program was conducted for two weeks in December 2007 to provide more detailed data from a carefully controlled plant sampling campaign. After establishing existing flows and loads, allowances were made for residential and commercial growth within the District to determine future design flows and loads. Although TM1 was never officially adopted by TDBCSD and remains in draft form, the information on existing flows and loads provided therein is very pertinent to this investigation. Therefore, the previously completed draft TM1 is included herewith as Appendix A. The reader is referred to Table 1-6 in TM1 for a summary of existing and then projected future flows and loads.

A key finding of TM1 was that the historical plant data (2004-2007) on influent BOD and TSS concentrations was unreliable; therefore, the average influent BOD concentration of 240 mg/L developed in the December 2007 special monitoring effort was adopted as an appropriate planning value. Similarly, the average influent TSS was established at 312 mg/L based on a TSS/BOD ratio of 1.3 developed in the special monitoring effort. In TM1, it was recognized that the apparent TSS/BOD ratio of 1.3 was unusually high and that there were questions regarding unusual values for other constituent concentration ratios also (e.g., COD/BOD, TKN/BOD, and COD/VSS). Therefore, TM1 included a recommendation for future additional monitoring to check the results.

5.2 Analysis of Recent Plant Data

Plant influent flows and biochemical oxygen demand (BOD_5 or simply BOD) concentrations and loads from January 2009 through May 2010 were obtained for this analysis and are discussed below.

5.2.1 Influent Flows

Daily and rolling 30-day average influent flows are shown in Figure 5-1. As indicated in the Figure, flows are typically within the range from about 1 to 2.5 Mgal/d. The average flow for the entire period was 1.75 Mgal/d, which is nearly the same as the average annual flow (AAF) of

1.80 Mgal/d established in TM1. Therefore, the existing average annual flow of 1.80 Mgal/d is confirmed. Additionally, noting that the rolling 30-day average flow reached almost 2.0 Mgal/d on several occasions (Figure 5-1), the average day maximum monthly flow (ADMMF) of 1.98 Mgal/d (equals 1.1 x AAF) is confirmed.

On five days over the period analyzed, flows were near or just above 3.0 Mgal/d (May 2009 and February 2010). The flow of 3.37 Mgal/d recorded on May 26, 2009 is 1.93 times the average flow recorded over the entire period shown in Figure 5-1. Therefore, the peak daily design flow of 3.6 Mgal/d (equals 2.0 x AAF) previously established in TM1 remains valid.

No data on peak hourly flows were available for this study. A reasonable allowance, based on data from other areas, is 1.5 times the peak day flow, which would be 3.0 times the average annual flow, or 5.4 Mgal/d.

Since the flow limit given in the District's National Pollution Discharge Elimination System (NPDES) permit is based on the average dry weather flow (ADWF), which is generally taken as the average flow for the months of July through September, data from recent years was reviewed to determine the ratio between the average flow for July through September (ADWF) and the AAF. It was found that the ADWF varies from about 95 to 98 percent of the AAF, with an average of about 97 percent. Therefore, the existing ADWF is estimated to be 1.75 Mgal/d.

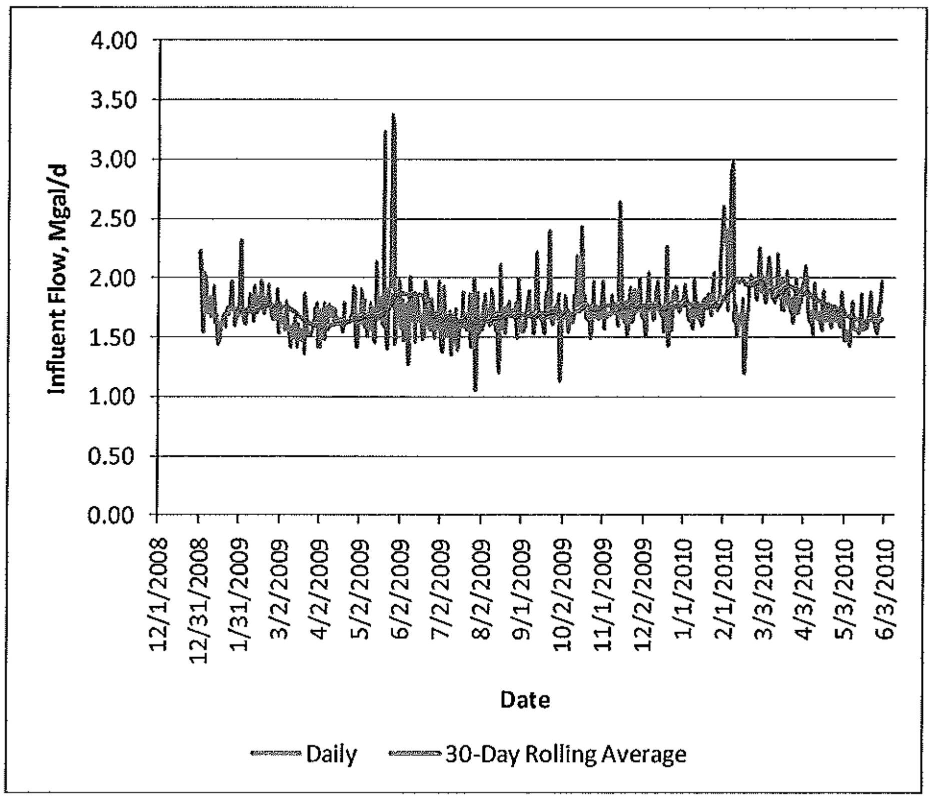


Figure 5-1
Influent Flows

5.2.2 Influent BOD

Daily and rolling 30-day average influent BOD loads are shown in Figure 5-2. As shown, except for two apparent anomalous excursions, the influent BOD load is typically just near or just over 2000 lb/d, which is much lower than the average annual BOD load of 3603 lb/d established in TM1. Influent BOD concentrations are shown in Figure 5-3 and were typically in the range of 100 to 200 mg/L, which is much less than the average annual concentration of 240 mg/L established in TM1.

A possible explanation for the generally low BOD concentrations and loads indicated by the 2009/2010 data is that influent samples may have been inadvertently partially filtered by rags and paper wrapping around the influent sampler intake tube. This problem was discussed in TM1 with regard to the data analyzed therein. For the intensive monitoring effort conducted in December 2007, the sampler intake tube was cleaned daily.

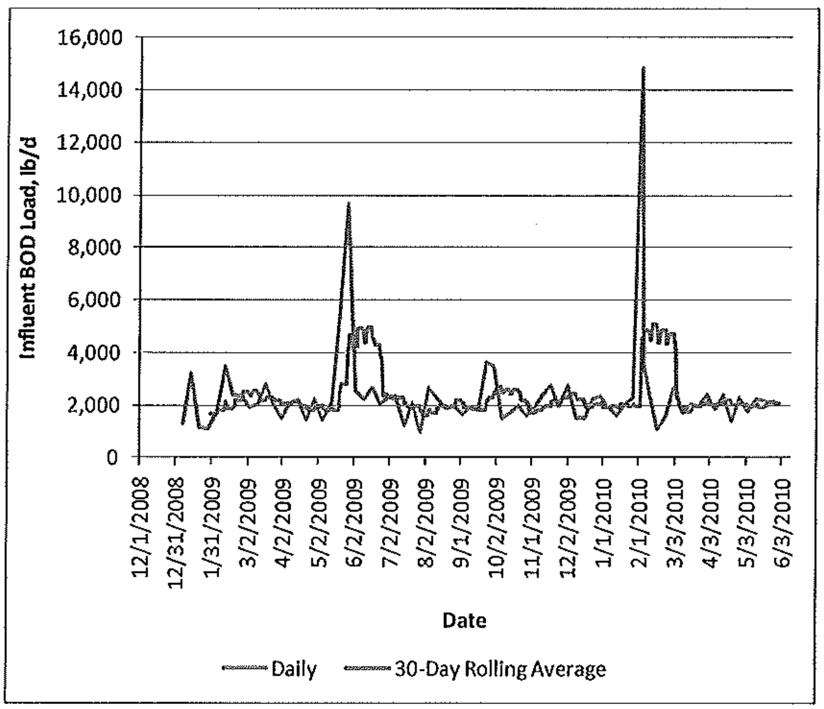


Figure 5-2
Influent BOD Load

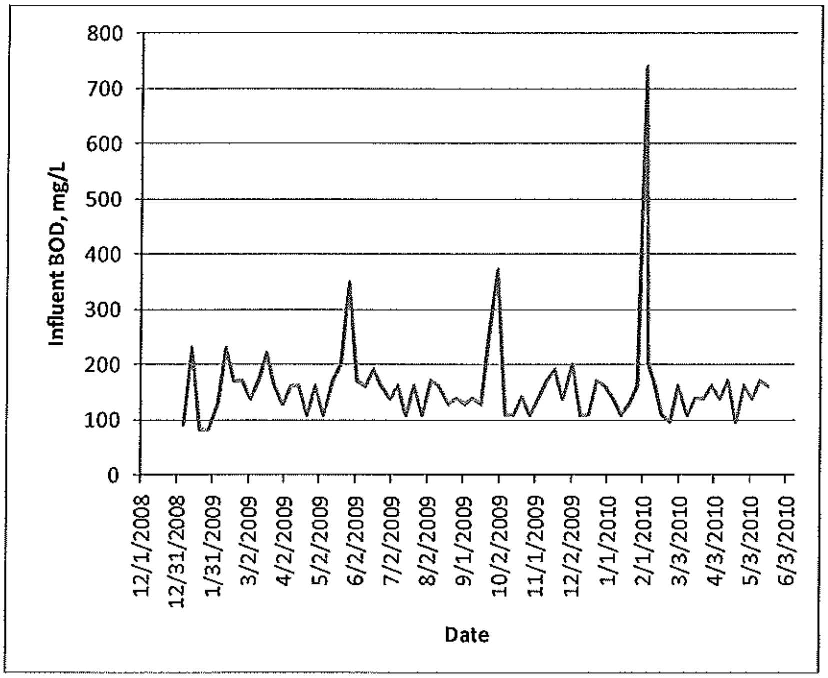


Figure 5-3
Influent BOD Concentrations

5.2.3 Influent Total Suspended Solids and Total Kjeldahl Nitrogen

Because of the influent sampling issues discussed for BOD, recent influent total suspended solids (TSS) data were not evaluated. Influent total Kjeldahl nitrogen (TKN) is not routinely monitored.

5.3 Special Monitoring Effort in July 2011

Because the strength of the influent wastewater directly impacts the sizing and cost of treatment facilities and because of lingering uncertainties regarding the wastewater strength, TDBCSD authorized a second special influent monitoring effort, which was conducted in July 2011. A complete description of the monitoring program and discussions of the results are presented in Technical Memorandum No. 2 (TM2), which is in Appendix C.

As a general summary, the July 2011 special monitoring results, like the 2009/2010 plant data, indicate a relatively low strength wastewater. The flow weighted average influent BOD concentration during the July 2011 special monitoring effort was about 160 mg/L. Average influent constituent concentration ratios from the July 2011 special monitoring effort were generally in line with expectations for typical domestic sewage, which are as follows: COD/BOD = 2.0, TSS/BOD = 1.0, TKN/BOD = 0.20, VSS/TSS = 0.90, and NH_3-N/TKN = 0.67.

5.4 Overall Assessment of Monitoring Data and Establishment of Existing Wastewater Flows and Loads to be used for Planning

In the following paragraphs, an overall assessment of the historical data discussed above is presented and additional relevant factors are considered to develop existing flows and loads to be used for completion of the Master Plan.

5.4.1 Flows

As previously discussed, recent plant influent flow data are generally consistent with the assessment of existing flows presented in TM1. Therefore, the existing flows indicated in TM1 and the average dry weather flow developed previously in this section are adopted for this Master Plan and are as follows:

Average Dry Weather Flow (ADWF) = 1.75 Mgal/d
Average Annual Flow (AAF) = 1.8 Mgal/d
Average Day Maximum Monthly Flow (ADMMF) = 1.98 Mgal/d
Peak Day Flow (PDF) = 3.60 Mgal/d
Peak Hour Flow (PHF) = 5.4 Mgal/d

At the time of the 2010 census, the population of Discovery Bay was 13,352. Therefore, the annual average flow of 1.8 Mgal/d implies an average flow of 135 gallons per capita per day (gpcd). Similarly, the average dry weather flow of 1.75 Mgal/d corresponds to 131 gpcd. These per-capita flows are quite high. It would generally be expected that the average annual flow would be 100 gpcd or less. The high flows could be indicative of persistent year-round infiltration of groundwater into the sewage collection system.

5.4.2 BOD Concentrations and Loads

Historical plant data and data from the two special monitoring efforts are not consistent with regard to influent BOD concentrations, as summarized below:

1. The data for the years 2004 through mid-2007 considered in TM1 included separate periods when the reported BOD concentrations generally ranged from 500 to 2000 mg/L, 50 to 500 mg/L, and 100 to 300 mg/L (see Figure 1-3 in TM1 [Appendix A]).

2. Results from the special monitoring effort completed in December 2007 and reported in TM1 (Appendix A) indicate an average BOD of about 240 mg/L.

3. Plant data for 2009 through May 2010 indicate BOD concentrations generally between 100 and 200 mg/L, with occasional excursions to much higher values (see Figure 5-3).

4. Results from the special monitoring effort completed in July 2011 and reported in TM2 (Appendix C) indicate an average BOD of about 160 mg/L.

It is noted that plant flows at the time of the December 2007 special monitoring effort and at the time of the July 2011 special monitoring effort were nearly the same at 1.61 and 1.57 Mgal/d, respectively, based on plant effluent flow. Therefore, differences in infiltration and inflow quantities are not believed to be a factor in the differing BOD concentrations.

In view of the uncertainties resulting from the data presented above, it is appropriate to consider per-capita BOD loads as a primary basis for establishing influent BOD loads and concentrations to be used for this Master Plan. In particular, the "Recommended Standards for Wastewater Facilities" developed by the Great Lakes – Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (commonly referred to as the "Ten States Standards") indicates an average per capita BOD load of 0.22 lb/d for communities with garbage grinders. This value has been recognized in engineering textbooks and is considered reasonable based on various evaluations for agencies in California. This criterion combined with the District population of 13,352 results in an existing average BOD load of 2,937 lb/d. With an average annual flow of 1.8 Mgal/d, the corresponding BOD concentration would be about 196 mg/L. Therefore, with rounding, the average annual BOD concentration adopted for this Master Plan is 200 mg/L. The existing average annual BOD load, with this rounded concentration, is 3,002 lb/d.

The average day maximum monthly BOD load is estimated to be 1.3 times the average annual BOD load. This is consistent with typical textbook values and with actual data from other facilities in Northern California. Similarly, the peak day load is estimated to be 2.0 times the average annual load.

5.4.3 TSS Concentrations and Loads

In the July 2011 special monitoring effort, the TSS/BOD ratio was found to be about 1.0, which is consistent with typical domestic sewage (see TM2 [Appendix C]). Therefore, existing TSS concentrations and loads are estimated to be the same as for BOD. The TSS/BOD ratio should be confirmed based on future monitoring.

5.4.4 TKN Concentrations and Loads

In the July 2011 special monitoring effort, the TKN/BOD ratio was found to be about 0.20, which is consistent with typical domestic sewage (see TM2 [Appendix C]). Therefore, existing TKN concentrations and loads are estimated to be 0.2 times those for BOD. The TKN/BOD ratio should be confirmed based on future monitoring.

5.5 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. According to the District Manager, there are 5172 single family homes and 222 condominium/townhouse units existing within the District. Assuming an equivalency factor of 0.75 for the condominium/townhouse units gives a total of 5339 equivalent dwelling units

(EDUs, where 1 EDU is equivalent to a typical single family home) for existing residential development. According to the District Manager the existing commercial connections within the District are roughly estimated to be equivalent to about 28 EDUs, resulting in a combined total of 5367 equivalent dwelling units (EDUs) for all existing development. Therefore, the average annual flow of 1.8 Mgal/d is equivalent to 335 gpd/EDU.

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 5-1.

Table 5-1
Average Annual Flows from Projected Growth

Development Type	Units	Number	Sewage Generation Rate, gpd/unit	Projected Flow, gpd
Residential	Homes	1,123	335	376,205
Commercial	Acres	10	1,600	16,000
Business Park / Office	Acres	90	2,000	180,000
Total				572,205 round to 570,000

5.6 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 5-2. It is assumed that wastewater constituent concentrations and flow and load variability for future growth will be the same as existing.

Table 5-2
Summary of Existing and Future Flows and Loads

Parameter	Existing	Incremental	Future
Flow, Mgal/d			
Average Dry Weather Flow (ADWF)	1.75	0.55	2.30
Average Annual Flow (AAF)	1.80	0.57	2.37
Average Day Maximum Monthly Flow (ADMMF)	1.98	0.63	2.61
Peak Day Flow (PDF)	3.60	1.14	4.74
Peak Hour Flow (PHF) (a)	5.40	1.71	7.11
Average Constituent Concentrations, mg/L (b)			
BOD	200	200	200
TSS (c)	200	200	200
TKN (d)	40	40	40
Average Annual Load (AAL), lb/d			
BOD	3,002	951	3,953
TSS (c)	3,002	951	3,953
TKN (d)	600	190	791
Average Day Maximum Monthly Load (ADMML), lb/d			
BOD	3,903	1,236	5,139
TSS (c)	3,903	1,236	5,139
TKN (d)	781	247	1,028

(a) Allowance at 3 x AAF. Confirm with future monitoring.
(b) AAF combined with AAL.
(c) Based on 1.0 x BOD. Confirm with future monitoring.
(d) Based on 0.2 x BOD. Confirm with future monitoring.

Section 6

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, and ultraviolet (UV) disinfection prior to export pumping for discharge into Old River. Waste sludge is aerobically digested and/or stored in lagoons, dewatered using a belt filter press, and dried in active solar drying units before landfill disposal.

The overall treatment system is located in two distinct geographical 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 through 2002 and has undergone several upgrades since then.

The influent pump station that serves both plants is located on the Plant 1 site. The discharge from the influent pump station is split approximately evenly to Plants 1 and 2 for treatment in screening and secondary treatment facilities. The secondary effluent from both plants is then combined within Plant 2 for UV disinfection and export pumping for discharge to Old River. All of the sludge handling facilities for both plants are located at Plant 2.

Copies of Construction Drawings G-2 through G-4 from the 2.0 MGD Expansion Project (when Plant 2 was added) are presented in Figures 6-1 through 6-3 to show plant flow schematics, hydraulic profiles, and design criteria, respectively. Clarifier 4, which is indicated as a future facility in these drawings has since been constructed. The drawings shown in Figures 6-1 through 6-3 do not include the sludge dewatering and drying facilities nor the Export Pump Station and discharge to Old River, which were subsequently added. Plant 2 was laid out to facilitate the future addition of effluent filtration facilities ahead of the UV disinfection system.

The Export Pump Station at Plant 2 currently includes four 20 horsepower vertical turbine pumps, each rated at 1.6 Mgal/d at 45 feet of head. There is space for a fifth pump to be added.

Sludge dewatering and drying facilities at Plant 2 include a 1.5 meter monobelt belt filter press and two active solar drying beds, each measuring 40 feet by 204 feet. The active solar drying beds are covered by greenhouse structures and include automated tilling machines and ventilation systems to promote sludge drying.

6.2 Existing Plant Performance

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

Biochemical Oxygen Demand (BOD_5, average monthly) ≤ 20 mg/L
Total Suspended Solids (average monthly) ≤ 30 mg/L
Ammonia Nitrogen (average monthly) ≤ 10 mg/L
Nitrate Nitrogen (average monthly) ≤ 73 mg/L
Total Coliform Organisms (weekly median) ≤ 23 per100 mL Most Probable Number
Electrical Conductivity (annual average) ≤ 2,100 µmhos/cm

In general, the plant is successful in meeting the discharge requirements indicated above, with the exception of occasional historical violations of the Total Suspended Solids and Total Coliform limits and violation of the electrical conductivity limit in 2010, all of which are discussed further in Section 8.

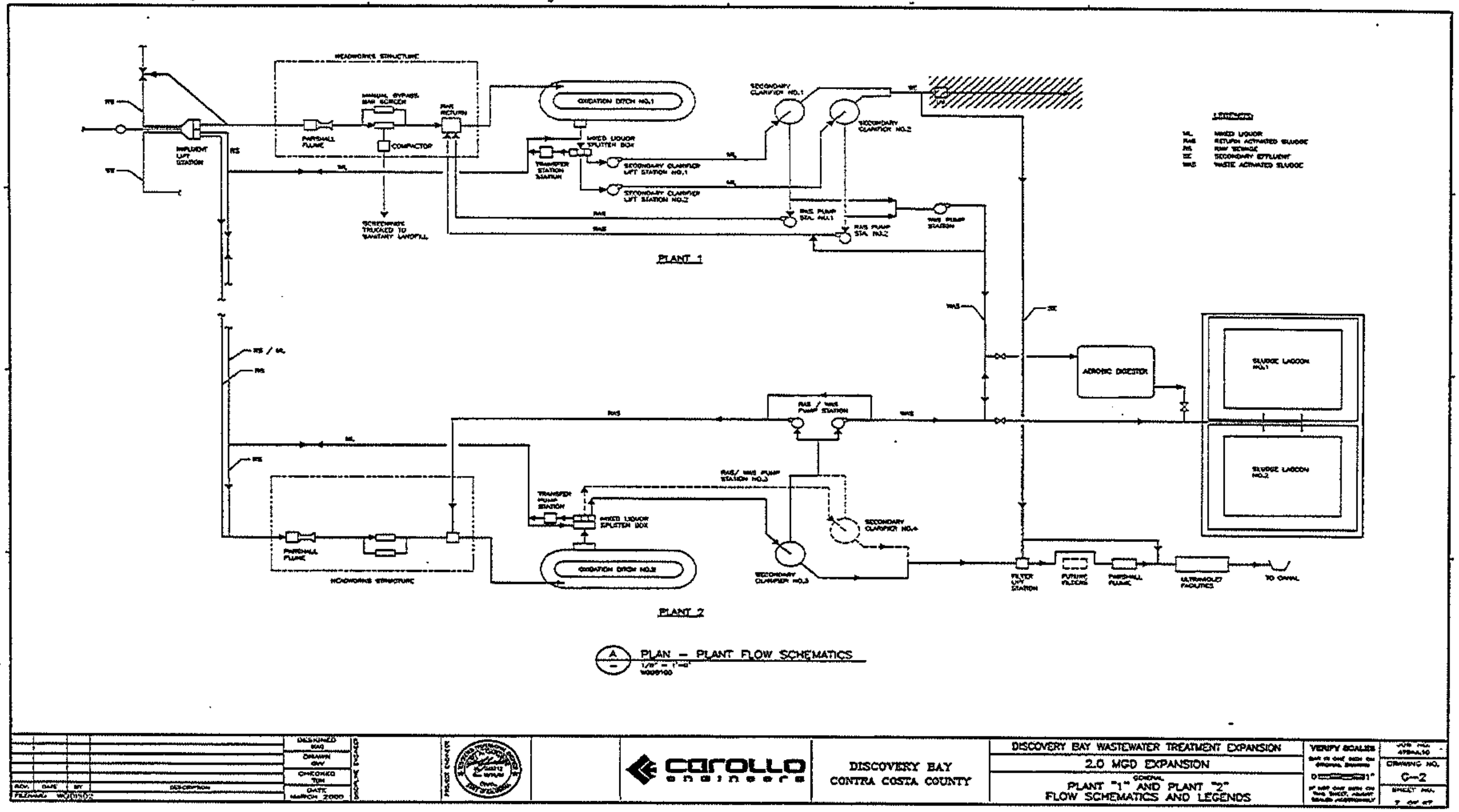


Figure 6-1
Flow Diagram

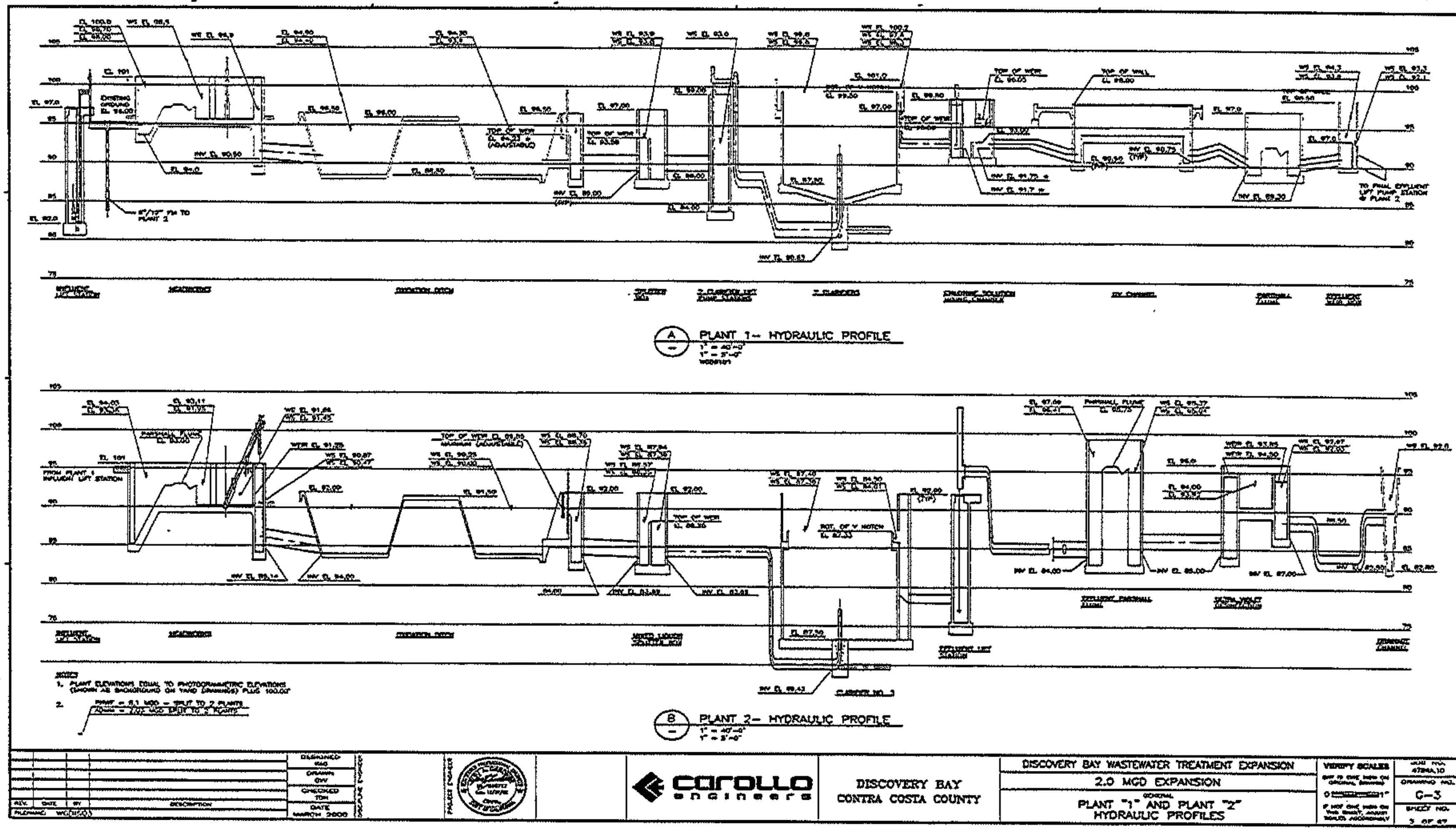


Figure 6-2
Hydraulic Profile

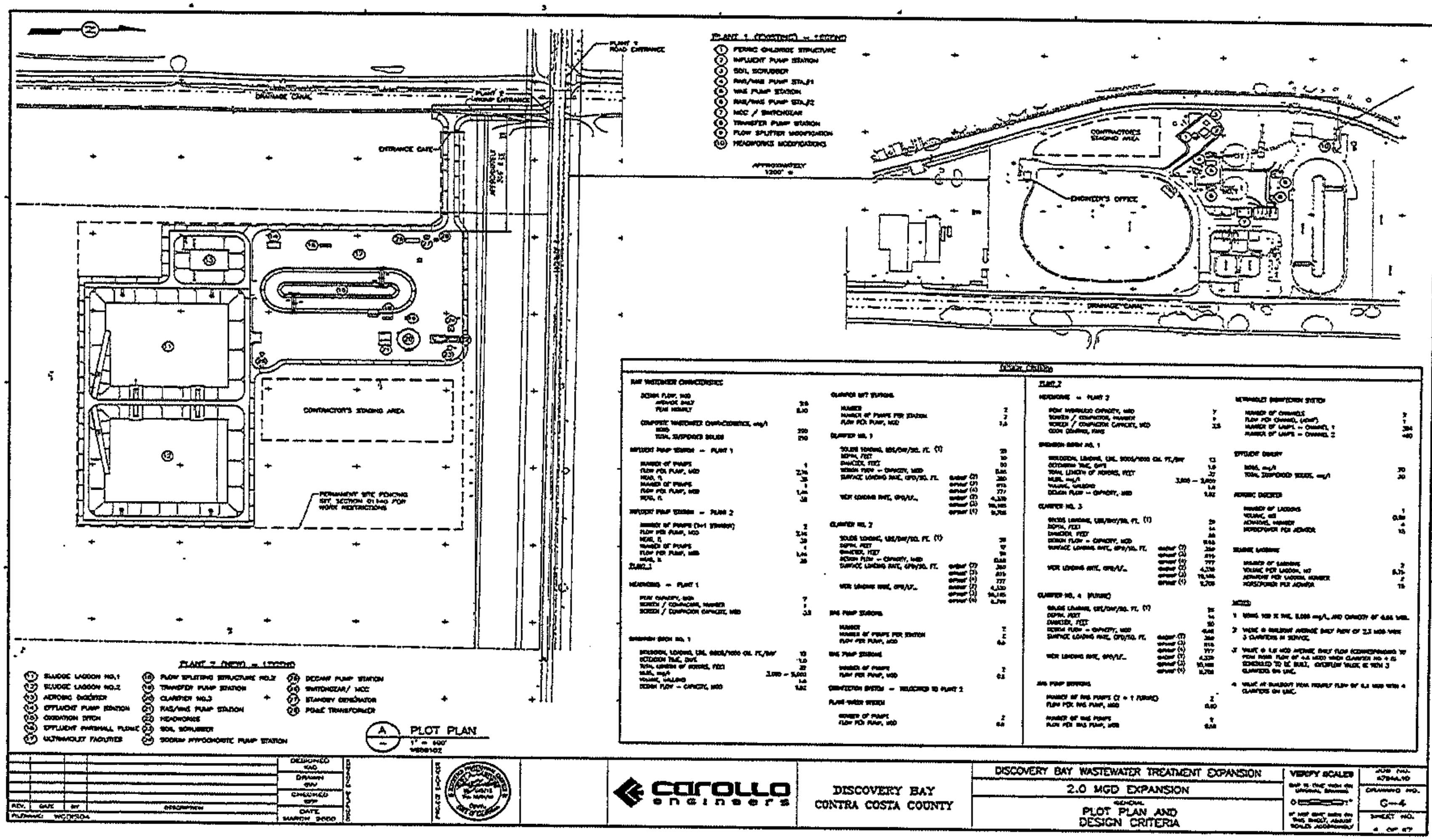


Figure 6-3
Overall Layout and Design Criteria

Section 7

Plant Hydraulic Capacity Analysis

To assess the ability of pumping and conveyance facilities in the plant to handle projected peak flows, a spreadsheet-based hydraulic model of the entire treatment plant (Plants 1 and 2) was developed. 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.

As a worst-case scenario, the hydraulic model was used to simulate existing facilities while handling the future peak hour flow of 7.11 Mgal/d, split equally to Plants 1 and 2. Another scenario including flow equalization after the secondary treatment facilities, resulting in a flow through downstream facilities of 4.74 Mgal/d (the future peak day flow) also was analyzed. A modification of the hydraulic model was also developed to assess conditions that would result if approximately two-thirds of the influent flow were routed to Plant 2 as the result of adding a new oxidation ditch treatment train at that location. The purpose of these analyses was to locate any hydraulic bottlenecks in the system so that future improvements can be planned to mitigate these bottlenecks.

7.1 Future Peak Hour Flow Split Equally To Plants 1 and 2, Without Equalization

In this scenario, the future peak hour flow of 7.11 Mgal/d was assumed to be split equally to the screening and secondary treatment systems in Plants 1 and 2 and then recombined for UV disinfection and export pumping at Plant 2, all without flow equalization or peak flow attenuation of any kind. Hydraulic bottlenecks identified from this analysis are discussed below.

7.1.1 Influent Pump Station

The Influent Pump Station has a total reliable pumping capacity of about 4.8 Mgal/d with one large pump out of service. Therefore, this pump station must be upgraded for the future peak hour flow of 7.11 Mgal/d. This topic is considered in Section 9.

7.1.2 Plant 2, Flow Splitting Structure 2 and Clarifiers 3 and 4

A hydraulic bottleneck exists between Flow Splitting Structure 2 and Clarifiers 3 and 4. The splitter box weirs are at elevation 88.25 feet and the clarifier launder v-notch weirs that set the water surface elevation in the clarifiers are at elevation 87.33 feet, a difference of only 0.92 feet. When allowing for a desired maximum return activated sludge flow of about 1 Mgal/d per clarifier (gives underflow rate of about 500 gpd/ft2), the maximum total plant influent flow (split equally to Plants 1 and 2) that can be accommodated without submerging the weirs in the splitter box is approximately 3.2 Mgal/d, which gives 1.6 Mgal/d to Oxidation Ditch 2. Even with the weirs submerged under the 7.11 Mgal/d scenario (3.56 Mgal/d to Oxidation Ditch 2), however, the flow should split equally to Clarifiers 3 and 4, since the piping to each clarifier and the clarifier internals that establish head losses are nearly identical.

With extreme peak flows, such as the worst-case 7.11 Mgal/d considered in this analysis, the submergence of the weirs in Flow Splitting Structure 2 is such that the hydraulic grade line is impacted further upstream at the Oxidation Ditch 2 outlet weir. The ditch outlet weir is adjustable and can be set to obtain the desired submergence of the oxidation ditch rotors, which determines the amount of oxygen transfer in the ditch. Typically, the rotor submergence is adjustable from about 6" to 14". However, with the extreme peak flow of 7.11 Mgal/d, the hydraulic grade backup from Flow Splitting Structure 2, would be such that the oxidation ditch outlet weir would become submerged and it would be impossible to attain rotor submergences less than about 10 inches. However, this is not considered to be a problem, because it is likely that submergence greater than 10 inches would be desired and, if not, providing more aeration than needed during the peak flow event is not a problem.

Based on the above, even though a hydraulic bottleneck exists between Flow Splitting Structure 2 and Clarifiers 3 and 4, there are no apparent negative consequences, even up to the extreme peak flow of 7.11 Mgal/d.

7.1.3 Secondary Effluent Lift Station

The Secondary Effluent Lift Station is currently used to lift the secondary effluent from both Plants 1 and 2 into the Parshall flume ahead of the UV disinfection system. The reliable pumping capacity of this lift station, with one large pump out of service is about 6.9 Mgal/d. This is almost equal to the worst-case future plant influent flow of 7.11 Mgal/d, so it is possible that no modification would be needed for continued pumping to the Parshall flume. This should be confirmed by observing actual peak flows in future years. If needed, the existing pumps can be operated at slightly increased speeds on the existing variable frequency drives to increase capacity.

Revised requirements for the Secondary Effluent Lift Station in the event that flow equalization and filters are added downstream are discussed in Section 7.2, below. The same requirements would apply if filters were added without flow equalization.

7.1.4 Export Pumping and Outfall to Old River

The Export Pump Station and Pipeline and river diffuser were designed to accommodate a flow of up to 6.2 Mgal/d. Currently, however, only four of five pump positions are used and the pumps were sized for initial flows, with plans to replace the pumps to accommodate future flows when needed. The current reliable capacity of the pump station is estimated to be about 4.0 Mgal/d, with one pump out of service. This is an approximate value; the actual value should be determined based on field testing.

It is theoretically possible to install export pumps large enough to accommodate the 7.11 Mgal/d future peak hour flow considered herein. With one of five pumps out of service, the pumps would have to be sized for about 1.8 Mgal/d at approximately 95 feet of head and would probably require 50 horsepower motors. This compares to the existing pumps, which are rated at 1.6 Mgal/d at 45 feet of head and have 20 horsepower motors. Therefore, the pumps would have to be replaced to accommodate a design flow of 7.11 Mgal/d. However, in consideration of possible future filtration and UV disinfection system improvements in Sections 13 and 14, secondary effluent flow equalization facilities are recommended to limit the peak flow to 4.74 Mgal/d, which would also apply to the Export Pump Station. This is considered under Section 7.2, below.

7.2 Future Peak Hour Flow Split Equally To Plants 1 and 2, With Equalization after the Secondary Effluent Lift Station

Under this scenario, the flows through all facilities upstream of the Secondary Effluent Lift Station were the same as in the previous scenario. Therefore, the hydraulic bottlenecks identified above for the Influent Pump Station and for Plant 2 Flow Splitting Structure 2 and Clarifiers 3 and 4 remain unchanged. For this scenario, all secondary effluent flows in excess of the future peak day average flow of 4.74 Mgal/d were assumed to be diverted from the discharge of the Secondary Effluent Lift Station to an equalization storage basin. The implications of this operation on the Secondary Effluent Lift Station and the Export Pump Station are considered below.

7.2.1 Secondary Effluent Lift Station

With flow equalization, there are two possible scenarios for the Secondary Effluent Lift Station: 1) continuing to pump to the Parshall flume if filters are not implemented, and 2) pumping to a future filtration system.

Without future filters, part of the flow that would otherwise be pumped to the Parshall flume would be diverted to the new equalization basin. Since the hydraulic grade line at the entrance to the Parshall flume (while 4.74 Mgal/d is passed through the flume) would be at about elevation 96.9 feet and the water level in the Secondary Effluent Lift Station sump would be at a maximum elevation of 82.5 feet, flow could be diverted from the pump discharge to an equalization basin and then drained by gravity back to the Secondary Effluent Lift Station. Of course, the Secondary Effluent Lift Station would have to pump the total flow passed ahead through the flume as well as the diverted flow, or the entire peak hour flow at this point in the process. As described in Section 7.1.3 above, however, it is possible that the existing reliable capacity of 6.9 Mgal/d for the Secondary Effluent Lift Station would be adequate or that the pump speeds could be increased slightly to accommodate a higher flow.

With future filters added, it is estimated that the Secondary Effluent Lift Station would have to pump the peak hour flow to a water surface elevation of about 102 feet (allows gravity flow through coagulation, flocculation and filtration facilities to the existing Parshall flume). Under this scenario, the Secondary Effluent Lift Station pumps would need to be upgraded or replaced to enable pumping the peak hour flow to this higher elevation. This topic is considered in Section 12.

7.2.2 Export Pump Station

With flow equalization, the Export Pump Station, export pipeline, and river diffuser system would have to handle a peak flow of only 4.74 Mgal/d. To meet a design capacity of 4.74 Mgal/d using four pumps (a fifth pump would be added as a standby unit), each pump would need to produce about 830 gpm at 58 feet of head. The existing pumps are capable of this operating condition if they are operated at a 107 percent over-speed condition using the existing variable frequency drives (vfds). This would still be within the motor horsepower rating.

7.3 Future Peak Hour Flow Split 1/3 to Plant 1 and 2/3 to Plant 2

If a new oxidation ditch treatment train with two clarifiers is added to Plant 2, then the flow split between Plants 1 and 2 will be 1/3 and 2/3, respectively. The Influent Pump Station modifications would have to be designed accordingly, which is discussed in Section 9. If only one clarifier is added with the new oxidation ditch at Plant 2, slightly less than 2/3 (about 65 percent) of the flow would go to Plant 2. If no new clarifiers were added with the new oxidation ditch at Plant 2, approximately 61 percent of the flow would normally go to Plant 2.

With only one-third of the flow going to Plant 1, there would be no hydraulic bottlenecks in the facilities there. Since all of the flow sent to Plants 1 and 2 would re-combine at the Secondary Effluent Lift Station, conditions from that lift station and downstream would be the same as considered in Sections 7.1 (without equalization) and 7.2 (with equalization), above.

The key differences of concern in hydraulic conditions between this scenario and the previous two scenarios would occur from the headworks to the Secondary Effluent Lift Station in Plant 2 and are discussed below.

The headworks at Plant 2 would need to handle two-thirds of the 7.11 Mgal/d peak hour flow, or 4.74 Mgal/d. Since the existing screen was designed to handle up to 6.2 Mgal/d, this is not a problem.

A new splitter box would have to be added between the headworks and the oxidation ditches. Since the floor elevation at the headworks screen is about the same as the maximum water surface elevation in the existing oxidation ditch, there is less hydraulic gradient available for insertion of a splitter box than is desirable. The splitter box weirs will have to be above the floor elevation at the screen, which will not allow the screen channel to drain down, even at low flows. Although this could result in low velocities that would allow some solids to settle in the screen channel during low flows, this should not be a significant problem. At high flows, the depth of the channel downstream from the screen would be within allowable limits.

With the second oxidation ditch and additional clarifier(s) added at Plant 2, the flow through each oxidation ditch would be two-thirds or less of the flow considered under the previous two scenarios. If two clarifiers are added, the flow per clarifier would be two-thirds of the flow considered under the previous two scenarios. Accordingly, the amount of submergence of the clarifier splitter box weirs would be substantially reduced and there would be no submergence of the oxidation ditch outlet weir, allowing a full range of rotor submergence. If only one clarifier is added at Plant 2, the flow per clarifier will be slightly less than under the previous two scenarios, resulting in slightly less submergence of the clarifier splitter box weirs and the oxidation ditch outlet weirs. If no new clarifiers are added at Plant 2, the flow per clarifier could go up more than 20 percent compared to the previous two scenarios, exacerbating the weir submergence problems. However, depending on sludge settleability (SVI) at the time, it may be possible to mitigate the weir submergence at Plant 2 by forcing more than 39 percent of the flow to go to Plant 1 during these extreme peak flow events. If a 50/50 flow split was forced during the peak event, the flow per clarifier and the clarifier weir submergence would be the same as the scenario considered in Section 7.1.

7.4 Summary

Based on the results and discussion presented above, the existing plant hydraulic features can accommodate the future peak flows with suitable modifications to the main pumping facilities, including the Influent Pump Station, the Secondary Effluent Lift Station, and the Export Pump Station. This conclusion is applicable whether the flow is split equally to Plants 1 and 2 or whether approximately 2/3 of the total flow is routed through secondary treatment facilities at Plant 2 as the result of adding another oxidation ditch and one or two clarifiers at Plant 2.

Section 8
Waste Discharge Requirements

The Discovery Bay wastewater treatment plant effluent is discharged to Old River at a location approximately one-half mile southeast of Plant 2. The discharge is regulated under a National Pollution Discharge Elimination System (NPDES) permit and waste discharge requirements adopted by the California Regional Water Quality Control Board, Central Valley Region. The permit is updated approximately every five years. The current permit was adopted on December 4, 2008 (Order No. R5-2008-0179, NPDES No. CA0078590).

In this section, key provisions of the existing permit are summarized and compliance issues are assessed. Finally, potential future permit and treatment requirements are discussed.

8.1 Existing Permit Requirements and Compliance Assessment

Key effluent limitations contained in the NPDES permit are summarized in Table 8-1. For each parameter, an assessment of the existing plant performance and compliance strategies are indicated. 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 conditions in Old River. Included, for example, are limitations on bacteria, dissolved oxygen, pH, turbidity and biostimulatory substances (as well as others). No receiving water limitation compliance issues are known to exist or are anticipated.

As indicated in Table 8-1, the plant is generally compliant with most of the effluent limitations contained in the permit. Historically, there have been occasional violations of the total suspended solids (TSS) and total coliform limits. Additionally, the yearly average electrical conductivity limit was exceeded in 2010.

8.2 Recent Permit Violations

Each of the permit compliance issues noted above is discussed briefly below.

8.2.1 Total Suspended Solids

There have been several violations of effluent TSS limits in the past few years, including three violations of the weekly average limit of 40 mg/L (actual values were 43, 44, and 54 mg/L) and two violations of the daily maximum limit of 50 mg/L (actual values were 63 and 66 mg/L), which occurred between December 31, 2008 and August 8, 2009, and were listed in a Civil Liability Complaint issued by the California Regional Water Quality Control Board in December 2009. Since then, however, the plant operator reports that performance has been improved and that TSS violations have been mitigated, despite ongoing operational difficulties as noted below.

According to the plant operator, TSS compliance has been challenging at times, in part due to problems with clogging at the secondary clarifiers and return activated sludge (RAS) pumps at Plant 1. Reportedly, the clarifier sludge removal tubes and the RAS pumps are prone to clogging with rags and balls of stringy materials. With clogging, the sludge cannot be removed properly from the clarifiers, leading to TSS violations. Apparently, frequent action is required to remove rags and to clear or prevent clogging. This situation is surprising, since both Plants 1 and 2 have headworks with fine screens that are specifically designed to remove rags and stringy materials. Apparently, the screens have not been functioning properly, allowing raw sewage to overflow into a screen bypass channel, which has only a coarse bar rack and does not adequately remove rags and stringy materials. This phenomenon was confirmed by the District Engineer who noted clear evidence of the screen bypasses upon inspecting the headworks on multiple occasions. It is believed that these problems can be mitigated by repair and maintenance of the screens and related controls.

Two other issues reported by the operator are that effluent TSS can be elevated when the launder channels in the secondary clarifiers are cleaned and when pump cycling in the secondary effluent pump station stirs up solids that may have settled in the pump sump. However, these problems should be transient and of short duration, such that a 24-hour effluent composite sample should not be substantially impacted. Also, recent plant improvements include provisions for temporary diversions of poor quality plant effluent to the sludge lagoons that can be used to mitigate these problems.

8.2.2 Total Coliform

There have been several violations of effluent total coliform limits in the past few years, including five violations of the weekly median limit of 23 MPN/100 mL, which occurred in December 2008 (one violation at 840 MPN/100 mL) and July 2009 (four violations, all at 27 MPN/100 mL) and were listed in a Civil Liability Complaint issued by the California Regional Water Quality Control Board in December 2009. Although recent UV disinfection system improvements should enhance total coliform compliance, violations have occurred during startup and shakedown of the improvements. It is hoped that the recent problems will be resolved after construction-related impacts have ceased and with operational adjustments to the new UV disinfection system.

The efficacy of the UV disinfection system is affected by the solids content and turbidity of the secondary effluent. With high turbidity (substantially over 10 NTU), adequate disinfection can be problematical. As a safeguard against such conditions, the UV disinfection system improvements included provisions for automatic diversions of plant effluent to the sludge storage lagoons in the event of secondary effluent turbidity over an adjustable setpoint limit.

At this time, it is not known whether the UV disinfection system improvements described above will provide an acceptable level of reliability in meeting the total coliform limits. If the maximum turbidity needed for reliable disinfection is such that automatic diversions of secondary effluent to the sludge lagoons would occur more frequently than desired, effluent filtration could be required prior to UV disinfection. This topic is addressed in Sections 13 and 14.

8.2.3 Electrical Conductivity

Electrical conductivity is a measure of the salinity of the wastewater effluent.

From January 14, 2004, through October 10, 2007, the average effluent electrical conductivity was 1921 µmhos/cm and the range was 724 to 2,280 µmhos/cm, based on 91 samples. These values far exceed the goal of 1,000 µmhos/cm for agricultural use. At the current time, however, treatment for salinity reduction is infeasible. Therefore, the permit requirement of 2,100 µmhos/cm was established to prevent further degradation above the previous highest annual average value. However, that limit was exceeded in 2010, when the average annual electrical conductivity was 2,192 µmhos/cm.

Recent monitoring efforts conducted by the District indicate that the electrical conductivity in the sewage from new development is substantially greater than the average electrical conductivity in sewage from the District as a whole. It is believed that this is due to the general use of water softeners in the new homes. Future monitoring efforts are planned to assess the actual impact of the water softeners.

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.

In Section 15 of this Master Plan, the possibility of future wastewater treatment to reduce salinity is considered.

8.3 Possible Future Permit Requirements

The general trend in permitting is to become more and more stringent over the years and wastewater reclamation is becoming more and more important as a means of supplementing scarce water resources. Accordingly, the potential of providing effluent filtration and improved disinfection to meet more stringent effluent standards and/or to allow reclamation must be considered in this master plan. Even without such changes, effluent filtration could be required for more reliable UV disinfection, as discussed above. Effluent filtration is considered in Section 13.

Salinity in water supplies is an increasing concern throughout the state and regulations and permitting language relating to salinity in wastewater are evolving. As mentioned previously, the possibility of future requirements for salinity reduction is briefly considered in Section 15 of this Master Plan.

Table 8-1
Key NPDES Permit Requirements, Plant Performance and Compliance Strategy

Parameter	Units	Effluent Limits [a]	Existing Plant Performance	Compliance Strategy
Flow	Mgal/d	2.1 [b]	Generally compliant.	Expand plant and revise permit before limit is reached.
BOD	mg/L	20/40/50	Generally compliant.	Continue current performance or better.
TSS	mg/L	30/40/50	Occasional noncompliance.	Resolve the problem of influent screen bypassing that can lead to clogging of secondary clarifier sludge removal systems and RAS pumps. Operate and maintain the secondary process and design improvements to provide good performance, in general. As a last resort, utilize new provisions for temporary diversion of poor-quality effluent to the sludge lagoons.
pH	Units	6.5 to 8.5 [c]	Generally compliant.	Continue current performance or better.
Copper	µg/L	50/–/70	Generally compliant.	Continue current performance or better.
Nitrate-N	mg/L	73/–/126	Generally compliant.	Continue current performance or better.
Ammonia-N	mg/L	10/–/30	Generally compliant	Continue current performance or better.
Total Coliform	MPN/100 mL	23, 240 [d]	Occasional noncompliance, prior to recent improvements (2010).	The UV disinfection system has been improved and provisions have been made to divert poor quality effluent to storage. If these improvements are not adequate, effluent filtration could be required.
Electrical Conductivity	µmhos/cm	2,100 [e] [f]	Noncompliant in 2010	Minimize salinity through source control and minimize or prevent salinity increase during treatment. As a last resort, if required in the future, provide treatment to remove salinity.
Iron (Total Recoverable)	µg/L	300 [e]	Generally compliant	Continue current performance or better.
Aluminum (Total Recoverable)	µg/L	200 [e]	Generally compliant	Continue current performance or better.

(a) Unless indicated otherwise, limits are Average Monthly/Average Weekly/Maximum Daily.
(b) This is specified as an "Average Daily" limit in the permit. However, the permit indicates that compliance will be assessed based on the "Average Dry Weather Flow", meaning the average flow over three dry weather months.
(c) Range is based on instantaneous minimum and instantaneous maximum.
(d) 23 weekly median, 240 not to be exceeded more than once in 30 days.
(e) Annual average.
(f) The limit decreases to 1,000 µmhos/cm if the District fails to implement a Salinity Plan.

Section 9
Influent Pump Station

The existing Influent Pump Station, although located at Plant 1, serves both Plants 1 and 2. In this section, a description of the pump station is provided, current operating issues are discussed and alternatives for improvement and expansion are considered. The rehabilitation and use of Pump Station W as a backup to the Influent Pump Station is also considered.

9.1 Description of Existing Facilities

Plan and section views of the existing Influent Pump Station, taken from the original construction drawings, are shown in Figure 9-1. As shown, there is a main sump compartment that receives influent raw sewage from the community via a 12-inch gravity sewer and a 12-inch forcemain (from Pump Station F). The sump also receives drainage from the chemical pump station and sewage from sources within Plant 1 through 4 and 6-inch pipelines.

From the main sump compartment, the raw sewage flows over manually adjustable weir gates into two separate pump sumps for pumping to Plants 1 and 2, respectively. There is an opening in the dividing wall so that each sump can overflow into the other, if the water level should rise substantially above the normal operating level.

The sump serving Plant 1 is currently fitted with one large pump and one small pump, rated at 2.0 and 1.15 Mgal/d, respectively, when both pumps are running at the same time. Therefore, the total pumping capacity to Plant 1 is about 3.15 Mgal/d. The reliable pumping capacity with the large pump out of service is 1.5 Mgal/d (the small pump running alone produces more flow than when running together with the large pump).

The sump serving Plant 2 is fitted with one large pump and two small pumps, which are identical to the corresponding units serving Plant 1. While pumping to Plant 2, the total capacity with all pumps in service is about 3.3 Mgal/d. The reliable pumping capacity with one large pump out of service is about 2.5 Mgal/d. There are parallel 8-inch and 12-inch forcemains from the influent pump station to Plant 2. The capacities listed here are based on using both forcemains.

Based on the capacities indicated above, the total reliable capacity of the Influent Pump Station can be based on the lowest capacity that would occur with one large pump out of service from either the Plant 1 or Plant 2 side. Accordingly, the total reliable capacity is estimated to be about 4.8 Mgal/d with the large pump on the Plant 1 side out of service. In this case, the flows to Plants 1 and 2 would be about 1.5 and 3.3 Mgal/d, respectively. If this condition should occur, the Plant 1 sump level would rise, submerging the weir gate on that side and forcing more flow to the Plant 2 pumps.

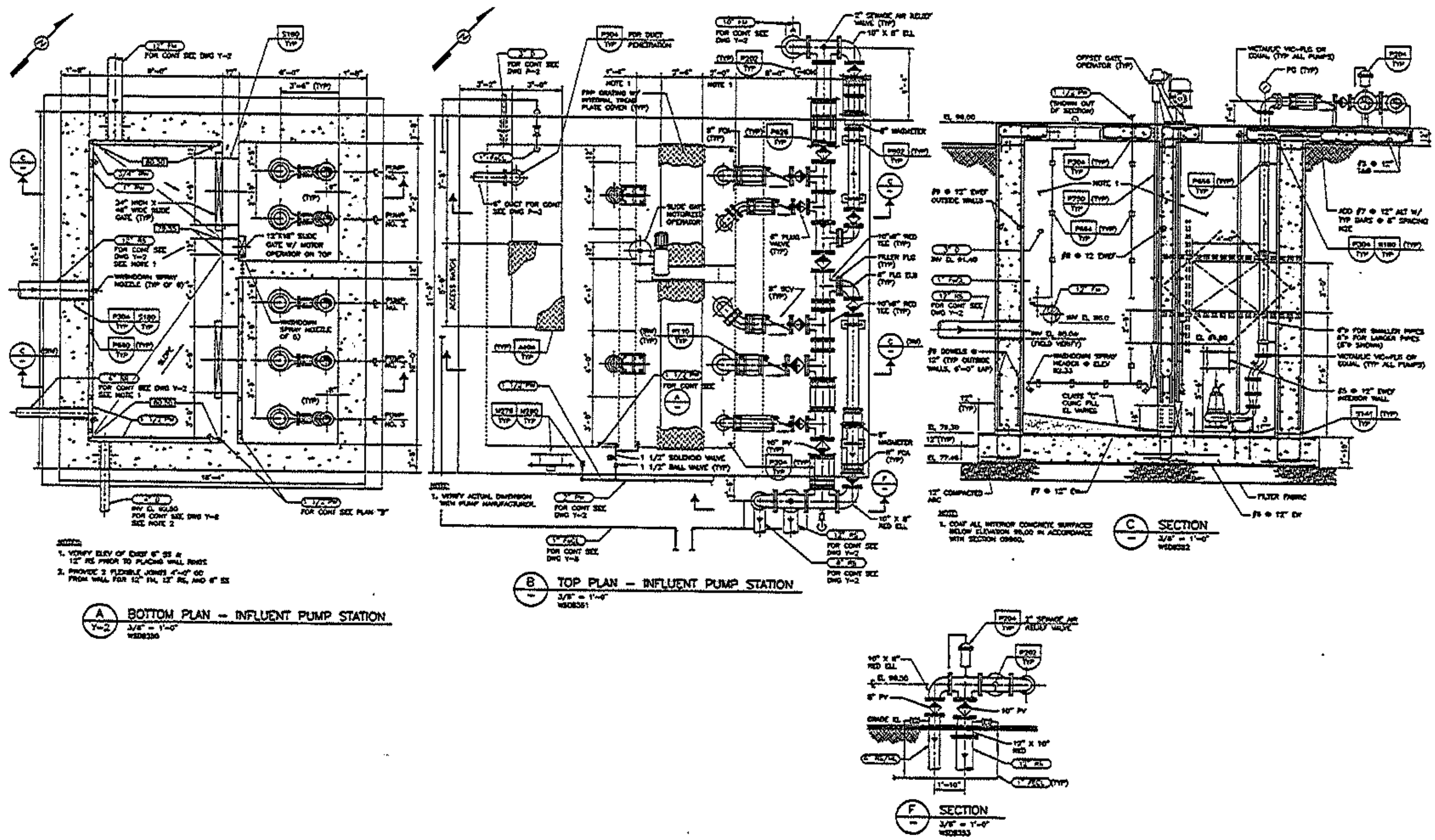


Figure 9-1
Existing Influent Pump Station

9.2 Existing Operational Issues

There are four main operational issues associated with the Influent Pump Station:

- Pump ragging
- It is difficult to adjust for unequal flow splits to the two plants, when desired
- The characteristics of the wastewater routed to Plant 1 are apparently different than the characteristics of the wastewater routed to Plant 2
- There are no provisions for taking this pump station completely out of service for repairs or maintenance

Each of these issues is discussed further below. Mitigation measures are discussed later in this section.

9.2.1 Pump Ragging

Based on discussions with District staff, the pumps at the Influent Pump Station have historically had a problem with clogging with rags or other stringy materials (referred to as ragging), resulting in the repeated need to remove pumps from the sump to clear the obstruction. The ragging problem is exacerbated when the pumps are operated at low speed to match low influent flows. Because of this issue, the control system limits on minimum speed have been adjusted upward such that the pumps operate intermittently at higher speeds, rather than continuously at lower speeds, during low flow conditions. With the higher speeds, the ragging problem is somewhat mitigated, but further improvement is desirable.

9.2.2 Lack of Flow Splitting Controls

Occasionally, due to maintenance or other issues, it is desirable to send more flow to one plant than the other. The only existing method for controlling the flow split is to adjust the weir gates leading to the sump compartments serving Plants 1 and 2. When the weir gates are set at the same elevation, the flow will split equally to the two plants over the full range of influent flows from minimum to maximum. However, when it is desired to route more flow to one plant or the other, the weir gates can be adjusted to attain the desired flow split at any given time, but as the total influent flow varies, the desired flow split is no longer maintained. Theoretically, to maintain a nearly constant percentage flow split to each plant with variable total flow, it is the length of the weirs that should be adjusted (and the weirs should be shaped differently), not the elevation; however, it is impractical to adjust the weir length.

9.2.3 Differing Wastewater Characteristics to Plants 1 and 2

Based on input from plant operations personnel, the wastewater that is pumped to Plant 1 is typically higher in strength than the wastewater that is pumped to Plant 2. This is somewhat surprising, since the pump sumps for both plants have a common inlet compartment. However, in reviewing Figure 9-1, it can be noticed that 12-inch gravity sewer coming into the pump station on the southwest side enters the facility at an approximate equal distance from the weir gate leading to Plant 1 and the weir gate leading to Plant 2. However, the 12-inch forcemain from Pump Station F enters the inlet compartment on the northwest side, near the weir gate leading to Plant 1. Accordingly, it is likely that disproportionate amounts of flow from the two sources are routed to Plants 1 and 2. If there are differences in the wastewater characteristics from the gravity sewer versus the forcemain, these would be reflected as differing loading conditions to Plants 1 and 2.

9.2.4 Inability to Take the Influent Pump Station Out of Service

Although it is possible to isolate and take out of service the individual sumps and pumps leading to Plant 1 or to Plant 2, there are no current provisions for taking the whole pump station out of service for repairs or maintenance in the common sump influent chamber. This is of concern since it is known that the coating system has failed and concrete repairs are required in this sump.

9.3 Future Capacity Requirements and Pump and Piping Modifications

As developed in Section 5, the future peak hour design flow is 7.11 Mgal/d. Since the existing pump station reliable capacity is 4.8 Mgal/d, substantial modifications are required. Furthermore, as developed in Section 11, it is planned to add another oxidation ditch treatment train to Plant 2. In that case, the normal flow split between Plants 1 and 2 will be approximately 1/3 and 2/3, respectively, depending on the number of clarifiers added at Plant 2. Therefore, the peak design flows to Plants 1 and 2 will be approximately 2.37 Mgal/d and 4.74 Mgal/d, respectively. The analysis presented herein is based on a 1/3 - 2/3 flow split between the two plants, but the overall conclusions and recommendations would not change significantly if the flow split were slightly different.

With the high flows going to Plant 2 and the long forcemain to Plant 2, the design head for the pumps serving Plant 2 will be much different than for Plant 1. For Plant 1, it is recommended to provide one duty and one standby pump, each rated for 2.37 Mgal/d at 40 feet of head. For Plant 2, it is recommended to provide two duty and one standby pump, each rated for 2.37 Mgal/d at 95 feet of head. The pump head requirements were developed from the plant hydraulic model discussed in Section 7, modified as discussed below.

The existing Influent Pump Station includes 6-inch pump discharge piping at two positions (one each for Plant 1 and Plant 2) and 8-inch pump discharge piping in three positions (one for Plant 1 and two for Plant 2). Currently, there are large pumps at two of the three 8-inch piping positions and small pumps at the 6-inch piping positions and at the remaining 8-inch piping position. The original design intent was to someday replace the small pump at the 8-inch piping position with a large pump.

Since all five future pumps will have a capacity of 2.37 Mgal/d, the existing 6-inch pump discharge piping existing at two pump locations will have to be replaced with 8-inch piping. Additionally, to accommodate the high flow being routed to Plant 2, the existing 8-inch magnetic flow meter and associated piping for flow to Plant 2 should be replaced with 10-inch diameter facilities.

9.4 Pump Station Improvement and Expansion Alternatives

To effectively eliminate or drastically reduce the occurrence of pump ragging two main alternatives are considered: 1) install a new influent screen system ahead of the Influent Pump Station, and 2) replace the pumps with pumps that are less likely to clog. Each of these alternatives is considered below.

9.4.1 Influent Screening Ahead of the Influent Pump Station

Under this alternative, a new headworks facility with screens would be constructed ahead of the Influent Pump Station. This facility would replace the individual headworks screens at the two plants.

Since the gravity sewer coming into the existing Influent Pump Station is approximately 12 feet below grade, the new screening channels would have to be below that elevation. It is estimated that the complete headworks could cost around $1 million. It is believed that this cost is not warranted, since there are options to use pumps that are less prone to ragging than the current pumps. Also, it is noted that it is common practice to have raw sewage pump stations in collection systems and treatment plants that are not protected by screens. Even if the District were to consider screens ahead of the Influent Pump Station, it would still have 15 collection system pump stations not protected by screens.

Besides the issues mentioned above, it is noted that it may be impossible to accommodate the head losses resulting from the new headworks, while still continuing to use the existing Influent Pump Station. The resulting depth in the pump sumps would likely be inadequate. No investigations were developed to see if this issue could be mitigated.

Based on the above considerations, screening ahead of the Influent Pump Station is not recommended.

9.4.2 Pump Replacement Alternatives

The existing Influent Pump Station was originally provided with Flygt non-clog submersible pumps with standard "C-Series" impellers. Since that time, Flygt has developed "N-Series" impellers, which were specifically designed to mitigate ragging. Recently one of the existing influent pumps was fitted with a new Flygt "N-Series" impeller. However, the unit has not been in service long enough to make a judgment on the degree to which ragging has been mitigated.

To increase the capacity of the Influent Pump Station, the existing pumps will have to be replaced. Three alternative pump types were considered for the replacements as follows:

- Flygt pumps with N-Series impellers.
- Pumps with screw centrifugal impellers, such as Wemco Hidrostal
- Chopper pumps, such as Vaughan

Pumps with screw centrifugal impellers have been used extensively in wastewater collection system pump stations and in wastewater treatment plants. Although generally more expensive than standard non-clog pumps they are much less prone to ragging and are frequently higher in efficiency. There are several manufacturers of screw centrifugal pumps.

Chopper pumps are wastewater pumps that are fitted with a mechanism for cutting into small pieces any rags or stringy materials that should enter the pump. Chopper pumps are used extensively in wastewater and sludge applications where standard non-clog pumps would be prone to clogging.

Proposals were requested and received from manufacturers of the three pump types being considered. In general, budgetary pricing (not including contractor markups and installation costs) for the Plant 1 pumps ranged from about $20,000 to $30,000 each. Budgetary pricing for the Plant 2 pumps ranged from about $35,000 to $45,000 each. The most efficient pumps would be the screw centrifugal type with efficiencies in the 75 to 80 percent range, followed by the Flygt N-Series pumps with efficiencies in the 70 to 75 percent range and chopper pumps in the 60 to 65 percent range. For all pump types, turndown to 0.33 Mgal/d and 0.67 Mgal/d for the Plant 1 and Plant 2 pumps, respectively, should not be a problem. It is likely that further turndown would be possible based on more detailed analysis during design. The respective manufacturers do not anticipate ragging problems even at turndown.

For this Master Plan, a final pump selection is not made. It is recommended that District staff and engineers evaluate the three pump types in more detail as the initial step of design. This should include contacting references and visiting facilities where the pumps of interest are already installed and have been in service for at least one year to confirm performance, reliability, freedom from ragging, maintenance requirements, manufacturer support and other issues of concern. Turndown capabilities should be confirmed in more detail and life cycle cost analyses performed. The costs presented herein for Influent Pump Station Modifications should be adequate to cover all three options.

9.5 Recommended Improvements

Recommended improvements to the Influent Pump Station include the following:

- Replace all pumps with pumping units designed for future flows and to avoid ragging, even at turndown.
- Replace the 6-inch pump discharge piping and valves at two pump positions with 8-inch facilities.
- Replace the 8-inch magnetic flow meter and associated header piping that leads to Plant 2 with 10-inch diameter facilities.
- Provide new controls for flow splitting between Plants 1 and 2.
- Install a mixer in the sump inlet compartment.
- Rehabilitate concrete and coatings as needed (after Pump Station W is activated to allow the Influent Pump Station to be taken out of service).

- Further discussion regarding flow splitting and the sump mixer are presented below, followed by a cost estimate for all improvements.

9.5.1 Flow Splitting and Controls

With 1/3 of the flow normally going to Plant 1 and 2/3 of the flow normally going to Plant 2, the existing equal-sized weir gates in the Influent Pump Station will no longer be appropriate for flow splitting. It would be possible to replace the weir gates such that the effective weir length for Plant 1 would be one-half of the effective weir length for Plant 2. Then, with the weirs at the same elevation a 1/3-2/3 flow split would occur. However, similar to the existing situation as previously discussed, such a solution would not provide a means for adjusting the flow split between the two plants, such as could be desired during maintenance and repair activities.

To allow variable flow splitting, it is recommended to automatically control the speed of the pumps such that the flow rate to Plant 2 is two times (or other desired ratio) the flow rate to Plant 1, as indicated on the magnetic flow meters at the Influent Pump Station used to monitor the flow to each plant. In this case, the weir gates to each sump would be left in their lowest position and the sump level on the Plant 1 side (or the side receiving the lowest flow) would be allowed to submerge the weir, forcing most of the flow to the Plant 2 side (or the side receiving the highest flow). The pumps on the Plant 2 side (or the side taking the most flow) would be controlled to maintain sump level, similar to the existing practice. The pumps on the Plant 1 side (or the side taking the least flow) would be controlled to produce one half (or other desired fraction) of the flow of the pumps on the other side. In the case that the total influent flow was below desired pump turndown for continuous operation, one pump on each side would be cycled on and off together at speeds that would provide the desired flow split.

To allow more turndown than would be possible by operating the Plant 1 and 2 pumps at minimum allowable flow rates, consideration could be given during design to providing a new interconnection with a magnetic flow meter and motorized pinch valve between the Plant 1 and Plant 2 pump discharge manifolds. Then, at low flows, the Plant 1 pumps could be operated to pump to Plants 1 and 2 at the same time. The amount of flow discharged to Plant 2 would be controlled by the pinch valve and monitored by the new magnetic flow meter. For the cost estimate presented herein, it is presumed that the new interconnection will not be provided.

9.5.2 Sump Mixing

In section 9.2, above, it was noted that the wastewater routed to Plant 1 is different than that routed to Plant 2 and that a possible cause for this condition is that the forcemain entering the sump inlet compartment is near to the weir gate leading to the Plant 1 pumps.

To assure that the wastewater routed to each of the two plants is generally the same, a submersible mixer could be installed in the sump inlet compartment. The mixer would have the added benefit of preventing accumulations of settling and floating solids, which would keep this sump inlet compartment much cleaner and reduce maintenance requirements.

9.5.3 Cost Estimate

A cost estimate for the recommended improvements to the Influent Pump Station is presented in Table 9-1.

Table 9-1
Cost Estimate for Improvements to the Influent Pump Station

Item	Cost, $1000s (a)
Replace all Five Pumps	330
Install Mixer In Sump Inlet Compartment	15
Piping Modifications	35
Misc. Demolition, Rehabilitation	50
Electical and Instrumentation	150
Subtotal 1	580
Contingencies @ 20% of Subtotal 1	116
Subtotal 2	696
General Conditions, Overhead and Profit @ 20% of Subtotal 2	139
Total Construction Cost	835
Engineering, Admin. and Environmental @ 25%	209
Total Capital Cost	1044

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

9.6 Pump Station W as a Backup to the Influent Pump Station

Pump Station W was the original Influent Pump Station at Plant 1. It includes a circular sump with three submersible pumps. This pump station was decommissioned when the current Influent Pump Station was built and put into service. However, the 12-inch gravity sewer that has now been re-routed to the new Influent Pump Station is still connected to Pump Station W and can be routed to Pump Station W by opening a slide gate in an upstream manhole. However, there is no slide gate or valve to allow stopping flow to the new Influent Pump Station.

The discharge piping from Pump Station W was left in place. The piping allowed Pump Station W to pump to the Plant 1 headworks or to an existing earthen basin on the Plant 1 site that was originally an aerated lagoon, was later a waste sludge holding basin, and was then abandoned. This earthen basin is indicated to be an emergency storage basin in the existing NPDES permit, however, permanent pumping and conveyance features to permit emergency storage use have not be installed. Full implementation of this emergency storage facility involving the use of Pump Station W is considered in Section 16.

Pump Station W could be reactivated as a backup to the Influent Pump Station (and for emergency storage use) and the Influent Pump Station could be taken completely out of service for repairs or maintenance by accomplishing the following:

- Install two new submersible pumps, each rated at about 2.5 Mgal/d, in Pump Station W.

- Provide new electrical supply and controls for Pump Station W.
- Provide a sluice gate at the Influent Pump Station to shut-off the 12-inch gravity sewer flow at that location.
- Interconnect the discharge forcemain from Pump Station W to the forcemain from the Influent Pump Station to Plant 2.

With the improvements listed above, the influent sewage coming to the Plant 1 site in the 12-inch gravity sewer would be handled by Pump Station W and would normally be pumped to Plant 2. However, by adjusting manual valves on the pump station discharge piping, a portion or all of the flow could be routed to Plant 1 or to the emergency storage basin. All of the influent flow coming to the Plant 1 site via the 12-inch forcemain from Pump Station F would be directed into the Plant 1 headworks using existing valves and interconnecting piping on that forcemain.

A cost estimate for re-activating Pump Station W as described above is presented in Table 9-2.

Table 9-2
Cost Estimate for Re-Activating Pump Station W

Item	Cost, $1000s (a)
Install Two New 2.5 Mgal/d Pumps	100
Interconnect Piping to Plant 2 Forcemain	30
Sluice Gate on 12-Inch Gravity Line at Influent Pump Station	10
Misc. Demolition, Rehabilitation	20
Electical and Instrumentation	50
Subtotal 1	210
Contingencies @ 20% of Subtotal 1	42
Subtotal 2	252
General Conditions, Overhead and Profit @ 20% of Subtotal 2	50
Total Construction Cost	302
Engineering, Admin. and Environmental @ 25%	76
Total Capital Cost	378

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

9.7 Consideration of Direct Pumping from the Newport Pump Station to Plant 2

The analysis and recommendations presented above are based on the continued routing of all wastewater from the community to the Plant 1 site. Within the Plant 1 site, the wastewater is then routed to Plant 1, Plant 2, or the emergency storage basin.

It is noted, that the forcemain from the Newport Pump Station in the collection system currently terminates at the Golf Course Valve Station, from which point the discharge then flows by gravity sewers to the Plant 1 site. The Golf Course Valve Station is only about 300 feet from the point where the forcemains from the Influent Pump Station to Plant 2 cross Highway 4. If the

Newport Pump Station forcemain were directly connected to one of the forcemains to Plant 2, this would avoid the need for re-pumping this flow at the Influent Pump Station. In this case, the design flow and head for the Influent Pump Station pumping to Plant 2 could be reduced accordingly. This alternative was not considered in further detail as part of the current Master Plan, but should be evaluated prior to final design of improvements to the Influent Pump Station.

If the direct tie from the Newport Pump Station to Plant 2 were implemented, valves could be provided to allow routing the Newport Pump Station flow either to Plant 1 or to Plant 2. Although the normal discharge point for the Newport Pump Station would be to Plant 2, it would be possible to route the Newport Pump Station flow through Plant 1 or to the emergency storage basin at Plant 1, if desired.

Section 10
Headworks

There are currently separate headworks systems at Plant 1 and at Plant 2. In this section, the existing facilities are described, known operating issues are considered, capacities are evaluated, and recommended improvements are discussed.

10.1 Description of Existing Facilities

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.

10.2 Existing Operational Issues

There are two key operational issues with the existing headworks systems: 1) bypassing of the mechanical screening units, and 2) unrepresentative sampling at the Plant 2 headworks. Each of these issues is discussed below.

As discussed in Section 8, bypassing of the screening units is the probable cause of rag accumulations in the downstream treatment facilities, particularly at Plant 1. These rag accumulations lead to pump and clarifier sludge suction tube clogging, possibly even leading to effluent permit violations for total suspended solids. As mentioned in Section 8, the District Engineer has confirmed that the mechanical screening unit has not been functioning properly, on occasion, leading to clogging of the mechanical screen, backups in the flow channel and overflow around the mechanical screen and through the manual backup bar screen. Such failures can be caused by the control system not calling for screen cleaning operations when needed or by mechanical problems with the mechanism used to clean the screen. In any case, it is believed that the problems can be resolved by appropriate repairs and maintenance.

The unrepresentative sampling issue is discussed in Section 5. As noted, it has been observed that the sampler intake tube accumulates rags and paper that may effectively filter the wastewater being sampled. It is necessary that the sampler intake be installed at a well-mixed location. The hydraulic jump at the exit of the Parshall flume is ideal for being well mixed and the sampler intake has been positioned there. Unfortunately, this is upstream of the influent screen, which exposes the sampler intake to the rag and paper accumulations. A resolution for this issue is discussed in Section 9.4.

10.3 Existing Capacity and Future Requirements

As developed in Section 5, the future peak hour design flow is 7.11 Mgal/d. With the proposed plant expansion, the normal flow split between Plants 1 and 2 will be about 35 and 65 percent, respectively. Therefore, the peak design flows to the headworks at Plants 1 and 2 will be about 2.49 Mgal/d and 4.62 Mgal/d, respectively.

The existing screening system at each plant has a maximum design capacity of 6.2 Mgal/d. Therefore, no modifications to increase the capacities of the screens should be needed.

10.4 Recommended Improvements

Consistent screening is necessary to protect downstream treatment facilities from clogging or being entangled with rags and stringy materials. As mentioned above, the existing screens have failed to perform in the past. The District should confirm that the screens are maintained and in good operating condition. If the screens repeatedly fail to perform, even with proper maintenance, the District should consider replacing the units with more reliable equipment. For this Master Plan, it is assumed that replacement is not necessary; however, this must be confirmed.

The long-term solution to the problem of unrepresentative sampling at the Plant 2 headworks is to implement a new sampling system downstream from the screen in the drop box leading to the headworks effluent pipe. A small mechanical mixer could be installed to keep this compartment well mixed and the sampler intake tube relocated to this position. However, this solution cannot be implemented unless the RAS discharge that is currently upstream from the drop box is moved somewhere downstream. In the future, presuming a second oxidation ditch treatment train is constructed, a new splitter box will be required downstream from the headworks and the RAS discharge could be relocated to the new splitter box structure at that time. Although it would be possible now to directly connect the RAS pipeline to the 24-inch oxidation ditch influent pipeline where they cross, that would be a disproportionately expensive and temporary solution.

For now, the best solution to the problem of sample tube intake clogging and unrepresentative sampling may be to create a mixed sampling pool immediately downstream from the screen. This could be done by installing a weir plate, perhaps six inches high, in the stop plate slot at the end of the screen channel. Then, a self priming pump could be installed to take suction out of the sample pool and discharge at two locations: 1) back into the sample pool and 2) into the pool that would be created between the Parshall flume and the screen. Both discharges would have a nozzle arranged horizontally under the water surface to create mixing in the areas of discharge. The automatic sampler intake could be connected to a sample tee in the pump discharge piping or could be placed directly in the mixed pool downstream from the screen. It is estimated that this solution could be implemented for about $10,000.

Section 11

Secondary Treatment Facilities

In this section, the existing secondary treatment system is described and the capacity of the system is evaluated based on normal operations and operation with key elements out of service. Alternatives for future expansion are considered and a recommended plan for expansion is presented.

11.1 Existing Facilities

The existing secondary treatment facilities are divided between Plant 1 and Plant 2. At each plant, there is one oxidation ditch, two secondary clarifiers and other ancillary facilities as described 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. 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.

At each plant, the mixed liquor from the oxidation ditch flows to a splitter box that is used to divide the flow equally to two secondary clarifiers. 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 ditch using the return activated sludge (RAS) pumps. A portion of the settled solids are wasted from the system and are pumped (using 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)	2,200 [(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
Mixed Liquor Transfer Pumps	No. Pumps	1 + 1 Standby
Mixed Liquor Transfer Pumps	Capacity per Pump, Mgal/d	0.58

(a) Estimated value, same as rotors in Oxidation Ditch 2, per District Engineer.

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

Component	Parameter	Value
Oxidation Ditch 2	Volume, Mgal	1.0
Oxidation Ditch 2	Number of Brush Rotors	4
Oxidation Ditch 2	Brush Rotor Horsepower, ea	30
Oxidation Ditch 2	Capacity per Brush Rotor, lb O_2 / d (Standard)	2,200
Clarifier 3	Diameter, ft	50
Clarifier 3	Depth, ft	14
Clarifier 4	Diameter, ft	50
Clarifier 4	Depth, ft	14
RAS Pumps (Serving Clarifiers 3 and 4)	No. Pumps	2 + 1 Standby
RAS Pumps (Serving Clarifiers 3 and 4)	Capacity per Pump, Mgal/d	0.60
WAS Pumps	No. Pumps	1 + 1 Standby
WAS Pumps	Capacity per Pump, Mgal/d	0.58
Mixed Liquor Transfer Pumps	No. Pumps	1 + 1 Standby
Mixed Liquor Transfer Pumps	Capacity per Pump, Mgal/d	0.58

Although there are only two secondary clarifiers at each plant, the splitter box ahead of these clarifiers has three outlet compartments – one for each clarifier and a third compartment that can be used to transfer mixed liquor to the other plant, in the event that one of the clarifiers for the plant in question is out of service. Any splitter box outlet not being used is blocked with stop plates. When the transfer provisions are used, the mixed liquor that exits the transfer section of the splitter box flows to a mixed liquor transfer pump station (there is one at each plant) for pumping to the splitter box of the other plant. Ideally, this transfer system would allow the two ditches to share the three clarifiers remaining in service when one clarifier is taken out of service. However, that is not currently possible, because there are no provisions for returning settled mixed liquor (RAS) back to the oxidation ditch from which the solids originated after the mixed liquor is transferred for settling in the other plant. Modifications needed to take full advantage of the mixed liquor transfer system are discussed in Section 11.3.

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. 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.

11.2 Capacity Assessment

The capacity of the existing secondary treatment system was assessed using a spreadsheet model to simultaneously solve biological process design equations for the oxidation ditches, secondary clarifiers and RAS pumping systems. In the paragraphs below, key parameter values used in the model are discussed, followed by consideration of modeling results for various plant operating scenarios.

11.2.1 Key Parameters used in Process Analyses

Key parameter values used in all of the process analyses considered herein, unless noted otherwise, are listed below:

- Average influent BOD = 200 mg/L
- Average influent TSS = 200 mg/L
- Average influent TKN = 40 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 = 2.0 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) = 200 mL/g

As noted above, sludge yields were based on values shown in Figure 11.7b of MOP8. This is because reliable plant influent load and sludge production data, which would be needed to calculate site-specific sludge yields, are not available. The MOP8 sludge yields are known to be conservatively high for most plants. For example, with a 10 day mean cell residence time (MCRT) and a temperature of 15 °C, the sludge yield would be estimated to be about 1.06 pounds of total suspended solids (TSS) per pound of BOD removed. Typical values would perhaps be around 80% of the MOP8 values. However, the MOP8 values are based on TSS:BOD ratios of 0.9 to 1.1. With higher TSS/BOD ratios, sludge yields would be higher than typical. Considering the uncertainties indicated in Section 5 with regard to the TSS/BOD ratios, it is prudent to be conservative and not reduce the MOP8 values. Based on the uncertainty of actual sludge yields, the capacity assessments presented herein are approximate, but believed to be reasonably conservative.

Several different plant operating scenarios were analyzed in the capacity assessments that are described in this section. For most of the scenarios, a mixed liquor temperature of 15 °C and a mean cell residence time (MCRT) of 10 days were used. The temperature of 15 °C is a typical minimum monthly effluent temperature, as determined from plant records. The low temperature condition is the most critical for plant design. The MCRT of 10 days should give reliable plant performance with nearly complete nitrification (ammonia conversion to nitrate) and the ability to do substantial simultaneous denitrification (conversion of nitrate to nitrogen gas) at temperatures at least as low as 15 °C. Although an MCRT of 10 days was used under critical low temperature and high load conditions, operation at substantially higher MCRT values would be possible most of the year with higher temperatures and lower loads. Additionally, if actual sludge yields are substantially lower than those assumed for this analysis, higher MCRT values would be possible at all times.

The degree to which nitrification and denitrification can be accomplished in the oxidation ditches is dependent on the temperature, the MCRT and the dissolved oxygen (DO) concentration. If the DO concentration is maintained at or above 2 mg/L and the MCRT is adequate, depending on temperature, essentially complete nitrification can be assured. If the DO is reduced substantially below 2 mg/L, nitrification can be limited, depending on the temperature and MCRT. Denitrification can only occur in the absence of dissolved oxygen. However, even when the DO in the bulk liquid is significantly above zero, the DO inside bacterial flocs can be zero, such that significant denitrification can still be achieved. It is important to assure reliable nitrification to meet the monthly average effluent permit limit for ammonia-nitrogen of 10 mg/L. Although the plant does have an effluent nitrate limit of 73 mg/L (monthly average), this limit is sufficiently high that essentially no denitrification is required. However, even if denitrification is not required, it is beneficial to provide some denitrification, because this reduces the demand for oxygen. Also, operating at low dissolved oxygen concentrations to promote denitrification increases the efficiency of oxygen transfer. Each of these factors results in lower power requirements. For this analysis, it was assumed that essentially no denitrification would be obtained with a DO concentration of 2 mg/L and that 50 percent denitrification could be obtained at a DO concentration of 1 mg/L. With the temperature and MCRT values used in this analysis, essentially complete nitrification should be possible, even at DO concentrations down to 1 mg/L.

For all of the analyses, a sludge volume index of 200 mL/g was assumed. This is a relatively conservative (high) value, indicating somewhat poor sludge settling characteristics in the secondary clarifiers. High SVI values can be caused by frequent or continuous operation at low dissolved oxygen concentrations. It is expected that the actual SVI should be below 200 mL/g most of the time, even when operating at DO concentrations as low as 1 mg/L, in which case the allowable plant capacity would be increased above the values indicated. However, actual desirable DO concentrations to avoid sludge bulking should be confirmed by the plant operators.

11.2.2 Scenarios Considered and Results

The various scenarios analyzed and key results are indicated in Table 11-3 and discussed below. Scenarios representing peak flows and loads and scenarios representing lower flow and load conditions are included in the analysis. In all cases, the capacity indicated in Table 11-3 is the average annual flow (AAF) corresponding to the scenario in question. As noted in Section 5, the average dry weather flow (ADWF), which is the basis of the flow limit given in the plant's NPDES permit, would be about 97 percent of the AAF.

Scenarios 1 and 2: Existing Plants, MCRT = 10 Days

Scenarios 1 and 2 are evaluations of Plants 1 and 2, respectively. As indicated in the Table, the average annual flow capacities of the plants are estimated at 1.03 and 0.97 Mgal/d, respectively, for a total of 2.0 Mgal/d. The slight difference in capacities for the two plants is the result of differing RAS pumping rates. Since the current AAF for the combined plant is 1.8 Mgal/d, this analysis would suggest that the plant is currently operating at about 90 percent capacity. However, the ability of the brush rotors to support the 2.0 Mgal/d capacity is marginal, as discussed below.

With four existing brush rotors in each oxidation ditch, the total standard oxygen delivery capacity is estimated at about 8,800 lb/d per ditch. Based on the standard oxygen requirements shown in the last four columns of Table 11-3, the existing brush rotors would not be able to meet either the peak day average or the peak hour oxygen requirements, while maintaining a DO concentration of 2 mg/L, with no denitrification. However, this should not be a problem, because depressed DO concentrations, which will promote some denitrification, are tolerable and are probably desirable, particularly during peak load conditions. With a DO concentration of 1 mg/L and assuming 50% denitrification, the existing brush rotors would be adequate to meet peak day average demands, but would not be able to meet peak hour demands. Although marginal, this condition is probably acceptable, because it would occur only on the peak hour of the peak day in the peak month. Under such rare conditions, depression of the DO below 1 mg/L and some ammonia breakthrough (caused by inadequate oxygen supply) can be tolerated. It should be noted, however, that this analysis presumes that all four brush rotors in both ditches would be in service. Since brush rotors can be out of service for maintenance or repairs, it would be beneficial to have a standby rotor in each ditch. Floating brush aerators could be used for this purpose. One 30 horsepower unit in each ditch would be recommended. When all aerators are in service, the standby unit would allow maintaining higher dissolved oxygen concentrations than would otherwise be possible during peak loading conditions, if desired. As an alternative to adding a floating brush aerator, a blower and a lift-out diffuser assembly can be evaluated before final implementation.

To summarize the results of Scenarios 1 and 2, the existing oxidation ditches, clarifiers, and RAS pumps can support an average annual flow capacity of about 2.0 Mgal/d, but aeration capacity is marginal and standby aeration equipment should be provided.

Scenario 3: Existing Plants with Upgraded RAS Pumping Capacity, MCRT = 10 Days

The capacity of a secondary clarifier is maximized when the RAS pumping rate produces a clarifier underflow rate (RAS flow divided by clarifier area) of at least 500 gpd/ft^2. For the existing 50-foot diameter clarifiers, that requires a RAS pumping rate of about 1 Mgal/d per clarifier. In Scenario 3, a RAS pumping rate of 1 Mgal/d per clarifier was assumed, resulting in a total combined capacity for the two plants of 2.13 Mgal/d. This is slightly greater than the 2.0 Mgal/d combined capacity without the RAS upgrade.

With the slightly increased capacity allowed by the RAS pump upgrade, the existing rotor capacity is even more challenged than indicated for Scenarios 1 and 2. At least one

30 horsepower floating brush aerator (or the equivalent) should be added to each oxidation ditch, as noted above.

Scenario 4: Existing Plants with Upgraded RAS Pumping Capacity, MCRT = 8 Days

Scenario 4 was developed to indicate the increase in capacity allowed by operating at a lower MCRT. Using a lower MCRT requires more careful operator attention and results in somewhat less reliable performance. However, it is believed that the 8 day MCRT should be adequate for temperatures as low as 15 °C. At the reduced MCRT, however, it may be difficult to assure reliable nitrification during peak loading conditions combined with minimum temperatures, particularly if the dissolved oxygen concentration is significantly below 2 mg/L.

As indicated in Table 11-3, lowering the MCRT from 10 days to 8 days increases the average annual flow capacity from 2.13 to 2.37 Mgal/d. At the higher capacity, it would be necessary to provide supplemental aeration capacity, beyond that allowed by the existing brush rotors and additional standby rotor capacity would be highly recommended. Two 30-horsepower floating brush aerators (or the equivalent) would be recommended for each ditch.

Scenarios 5 and 6: Existing Plants, Dry Weather Flows, Units Out of Service

The purpose of these scenarios is to evaluate the capacity of the existing plants (without RAS pumping upgrade and without mixed liquor transfers between plants) during dry weather flow conditions, while taking a clarifier or oxidation ditch out of service for maintenance or repairs. It is presumed that such maintenance or repair work could be scheduled at times of dry weather flows. The maximum dry weather flow during the peak flow hours of the day was assumed to be 1.5 times the average annual flow. It is presumed that peak loading conditions could occur during an extended shut down of an oxidation ditch or clarifier in the dry weather months. Therefore, peak month loading conditions were used for these scenarios. A mixed liquor temperature of 20 °C and a MCRT of 8 days were used in these scenarios to represent warm weather such as might occur in the spring or fall. Temperatures in the summer would be higher, resulting in more capacity than indicated for these scenarios.

Scenario 5 is based on Plant 1, with one clarifier out of service. The average annual flow capacity of this plant under the modeled conditions is 1.18 Mgal/d. Thus, even with one clarifier out of service, the AAF capacity of the plant with dry weather flows is greater than the AAF capacity of the plant with both clarifiers in service and with high wet weather flows (1.03 Mgal/d in Scenario 1).

Scenario 6 is based on Plant 2, with all facilities in service, under the same flow and load conditions as considered for Plant 1 in Scenario 5. The capacity of Plant 2 in this case would be 1.47 Mgal/d, resulting in a total combined AAF capacity for the two plants of 2.65 Mgal/d. Obviously, this exceeds the existing AAF of 1.8 Mgal/d and the future AAF of 2.37 Mgal/d. Therefore, except for rotor capacity, which is discussed below, there should be no problem taking a clarifier out of service during dry weather conditions. This same conclusion would apply to taking a clarifier out of service in either plant.

As noted in the last column of Table 11-3, the standard oxygen requirement (based on max. hour, DO = 1, 50% denitrification) for Scenarios 5 and 6 are 10,900 and 13,600 lb/d, respectively. However, these are based on a total capacity of 2.65 Mgal/d AAF, which is not needed. Under existing flow conditions (1.8 Mgal/d AAF), the oxygen requirements for Plant 1 and Plant 2 would be about 7,400 and 9,200 lb/d, respectively. Although the 9,200 lb/d requirement for Plant 2 slightly exceeds existing rotor capacity (8,800 lb/d), it is close enough that acceptable performance should be attained. For future conditions (2.37 Mgal/d AAF), the oxygen requirements for Plant 1 and Plant 2 would be about 9,800 and 12,200 lb/d, respectively. Therefore, additional aeration capacity equivalent to 0.45 and 1.5 existing rotors, respectively, would be needed.

Scenario 6 can also be considered to assess the impact of taking an oxidation ditch out of service during dry weather flows. Taking an oxidation ditch out of service would require taking the associated clarifiers out of service also. Thus, if the oxidation ditch in Plant 1 were taken out of service, all of the influent flow to the two plants would be routed through Plant 2. As mentioned above, the AAF capacity of the Plant 2 in this scenario would be 1.47 Mgal/d, which is less than the existing and future AAF. Therefore, it would not be possible to take the Plant 1 oxidation ditch out of service under the modeled conditions. Although not shown in Table 11-3 the Plant 1 capacity with all units in service under the same conditions would be 1.55 Mgal/d (higher because of higher RAS flows); therefore, it would not be possible to take the Plant 2 oxidation ditch out of service either.

Scenarios 7 and 8: Existing Plants with Upgraded RAS Pumping Capacity, Units Out of Service

Scenarios 7 and 8 are the same as Scenarios 5 and 6, respectively, except that RAS pumping rates are increased to 1.0 Mgal/d per clarifier. As indicated in the Table, the capacities would be increased somewhat, but it still would not be possible to take an oxidation ditch out of service.

Consideration of Peak Flow Trimming

Although not specifically included in the scenarios shown in Table 11-3, consideration can be given to trimming peak hour flows to the plant. Specifically, flows greater than the peak day average flow would be diverted to a storage basin and then returned for treatment after influent flows subside. The benefit of peak flow trimming would be to limit the peak overflow rate and solids flux on the secondary clarifiers. However, with peak flow trimming, the critical flow and loading conditions on the secondary clarifiers would be sustained for one or more days, as compared to one or more hours without peak flow trimming. Because of the sustained nature of critical conditions with peak flow trimming, it would be appropriate to apply additional safety factors for clarifier sizing, as compared to the case without flow trimming. The net result would be that the capacity with peak flow trimming would not be substantially greater than without peak flow trimming, but the reliability would be improved.

Table 11-3
Secondary Treatment System Capacity Assessment Results

Scenario	Description	Mixed Liquor Temp, °C	MCRT, days	AAF [a] Capac., Mgal/d	Max Month MLSS, mg/L	Max Month WAS, lb/d	Max Day SOR(b) Per Oxidation Ditch, lb/d		Max Hour SOR [b] Per Oxidation Ditch, lb/d	
							DO = 2 No Denit.	DO = 1 50% Denit.	DO = 2 No Denit.	DO = 1 50% Denit.
1	Existing Plant 1 (RAS = 0.8 Mgal/d per Clarifier)	15	10	1.03	3,000	2,500	9,900	7,500	12,800	9,600
2	Existing Plant 2 (RAS = 0.6 Mgal/d per Clarifier)	15	10	0.97	2,800	2,400	9,300	7,000	12,000	9,000
3	Both Plants Together with RAS Upgrade to 1 Mgal/d per Clarifier	15	10	2.13	3,100	5,200	10,300	7,800	13,200	9,900
4	Both Plants Together with RAS Upgrade to 1 Mgal/d per Clarifier	15	8	2.37	2,900	6,000	11,400	8,600	14,700	11,000
5	Existing Plant 1 (RAS = 0.8 Mgal/d per Clarifier) with One Clarifier Out of Service During Dry Weather Flows	20	8	1.18	2,700	2,900	11,400	8,600	14,600	10,900
6	Existing Plant 2 (RAS = 0.6 Mgal/d per Clarifier) During Dry Weather Flows	20	8	1.47	3,400	3,600	14,100	10,700	18,200	13,600
7	Either Plant with RAS Upgrade to 1 Mgal/d per Clarifier with One Clarifier Out of Service During Dry Weather Flows	20	8	1.22	2,800	3,000	11,700	8,800	15,100	11,300
8	Either Plant with RAS Upgrade to 1 Mgal/d per Clarifier During Dry Weather Flows	20	8	1.59	3,700	3,900	15,300	11,600	19,800	14,800

(a) AAF = Average Annual Flow
(b) SOR = Standard Oxygen Requirement

11.3 Future Improvements

As noted in Section 11.2, the capacity of the existing treatment facilities is about 2.0 Mgal/d AAF. To accommodate the projected increase in the average annual flow from 1.8 to 2.37 Mgal/d, together with the associated increase in loads, the secondary treatment system will have to be expanded or supplemented. Two alternatives for accommodating the future capacity are considered below.

11.3.1 Alternative 1 – Expand In-Kind

One potential option for expanding the two plants would be to add a third clarifier at each plant. If the RAS pumping capacities for all clarifiers were 1.0 Mgal/d, the total combined capacity of the two plants with all units in service would be about 2.49 Mgal/d AAF, which exceeds the future capacity of 2.37 Mgal/d AAF. However, in this case, it would not be possible to take either of the two oxidation ditches out of service, even under dry weather flow conditions (capacity would be 1.83 Mgal/d AAF with dry weather flows, 20°C, 8 day SRT). Therefore, it is concluded that expansion in-kind must include the addition of a new oxidation ditch.

The new oxidation ditch would be constructed at Plant 2. If it were desired to create an entirely new treatment train like the ones currently existing at Plants 1 and 2, then two new clarifiers would be added with the new oxidation ditch. However, the resultant capacity would substantially exceed the future requirement for 2.37 Mgal/d AAF. Therefore, options of adding zero, one, or two new clarifiers (and related RAS pumps) are considered below.

If no new clarifiers are added, the outflow of the new oxidation ditch would be routed to the existing clarifier splitter box such that the two existing clarifiers would serve the two ditches. If one new clarifier is added, it would be connected to the existing third outlet compartment of the existing clarifier splitter box. In this case, the three clarifiers together would serve the two ditches. If two new clarifiers are added, it would be possible to consider two scenarios: 1) dedicate the two new clarifiers to the new oxidation ditch, or 2) modify the existing clarifier splitter box to serve four clarifiers or build a new centralized four-way splitter box such that all four clarifiers together would serve the two oxidation ditches. The benefit of the second option is that taking a ditch out of service would not necessitate taking clarifiers out of service also.

If new clarifiers are added, the RAS pumping capacity associated with each new clarifier would be 1.0 Mgal/d. To maintain consistency, the RAS pumps for the two existing clarifiers at Plant 2 would be modified for the same capacity (existing capacity is 0.6 Mgal/d). However, if no new clarifiers are added, the options of either modifying or leaving the existing RAS pumps at Plant 2 can be considered. Regardless of what is done at Plant 2, the Plant 1 RAS pumps could remain at 0.8 Mgal/d per clarifier or be upgraded to 1.0 Mgal/d per clarifier.

In Table 11-4, the capacities of each plant and the total overall capacities are shown for the various combinations of alternatives discussed above. In each case, the capacity indicated is the average annual flow capacity corresponding to the indicated operating condition. Capacity results greater than the future average annual flow of 2.37 Mgal/d are highlighted. Therefore, non-highlighted results indicate that it would not be possible to operate the plant under the indicated conditions when buildout in the service area is reached. However, results close to 2.37 Mgal/d may be marginally adequate with a slight adjustment in the MCRT or other operating conditions. Key observations from Table 11-4 are listed below:

1. With all units in service, all options can provide for a future average annual flow of at least 2.37 Mgal/d under the critical design conditions (peak flows and loads, 15°C, MCRT = 10 days). Without adding any clarifiers, the available capacity would be 2.47 Mgal/d AAF without upgrading the RAS capacity at Plant 2 and 2.61 Mgal/d with Plant 2 RAS flows of 1.0 Mgal/d per clarifier. With three and four clarifiers, the available capacity is increased to 2.92 and 3.16 Mgal/d AAF, respectively, which is substantially more than needed. All of these capacities are based on Plant 1 RAS flows of 0.8 Mgal/d per clarifier, but would be increased by only 0.02 to 0.04 Mgal/d with Plant 1 RAS flows of 1.0 Mgal/d per clarifier.

2. Under the critical design conditions (peak flows and loads, 15°C, MCRT = 10 days), it would be possible to take a clarifier out of service at Plant 1, even without a clarifier addition at Plant 2, provided the RAS pumping capacity at Plant 2 is upgraded to 1.0 Mgal/d per clarifier (the indicated capacity of 2.35 Mgal/d is essentially equivalent to the future requirement of 2.37 Mgal/d). A clarifier at Plant 2 could be taken out of service under critical design conditions, only if a third or fourth clarifier is added.

3. None of the options would allow the oxidation ditch at Plant 1 to be taken out of service under the critical design conditions (peak flows and loads, 15°C, MCRT = 10 days). However, with four shared clarifiers at Plant 2 (all four clarifiers available to each ditch), one of the oxidation ditches at Plant 2 could be taken out of service, even under the critical design conditions.

4. All options would allow any clarifier or any ditch to be taken out of service under dry weather flow conditions with peak loads (20°C, MCRT = 8 days), except as follows: with only two clarifiers at Plant 2, the RAS pumping rate at Plant 2 would have to be upgraded to allow the Plant 1 oxidation ditch to be taken out of service.

In Section 7, the hydraulic implications of adding zero, one, or two clarifiers with a new oxidation ditch at Plant 2 are discussed. As indicated in that section, at least one new clarifier is needed to avoid exacerbating clarifier splitter box and oxidation ditch outlet box weir submergence issues at Plant 2 during peak flows (as compared to the scenario with two clarifiers and a 50/50 flow split between the two plants).

Table 11-4
Secondary Treatment System Capacity with Plant 2 Expansion

Units Out of Service		Plant 1 Capacity, Mgal/d [(a)(b)]		Plant 2 Capacity with Two Oxidation Ditches and Indicated Number of Clarifiers and RAS Rates, Mgal/d [(a)(c)]					Total Capacity, Mgal/d [(a)(c)(d)]				
Plant 1	Plant 2	RAS 0.8	RAS 1.0	2 Clar, RAS 0.6	2 Clar, RAS 1.0	3 Clar, RAS 1.0	4 Clar, RAS 1.0,	2+2 Clar, RAS 1.0	2 Clar, RAS 0.6	2 Clar, RAS 1.0	3 Clar, RAS 1.0	4 Clar, RAS 1.0	2+2 Clar, RAS 1.0
Peak Flows, Peak Loads, 15°C, MCRT = 10 Days													
None	None	1.03	1.07	1.44	1.58	1.89	2.13	2.13	2.47	2.61	2.92	3.16	3.16
1 Clar	None	0.77	0.79	1.44	1.58	1.89	2.13	2.13	2.21	2.35	2.66	2.90	2.90
1 Ditch	None	0	0	1.44	1.58	1.89	2.13	2.13	1.44	1.58	1.89	2.13	2.13
None	1 Clar	1.03	1.07	1.05	1.10	1.58	1.89	1.86	2.08	2.13	2.61	2.92	2.89
None	1 Ditch	1.03	1.07	0.97	1.07	1.25	1.38	1.07	2.00	2.10	2.28	2.41	2.10
Dry Weather Flows, Peak Loads, 20°C, MCRT = 8 Days													
None	None	1.55	1.59	2.20	2.43	2.86	3.18	3.18	3.75	3.98	4.41	4.73	4.73
1 Clar	None	1.18	1.22	2.20	2.43	2.86	3.18	3.18	3.38	3.61	4.04	4.36	4.36
1 Ditch	None	0	0	2.20	2.43	2.86	3.18	3.18	2.20	2.43	2.86	3.18	3.18
None	1 Clar	1.55	1.59	1.63	1.77	2.43	2.86	2.81	3.18	3.32	3.98	4.41	4.36
None	1 Ditch	1.55	1.59	1.47	1.59	1.83	2.01	1.59	3.02	3.14	3.38	3.56	3.14

(a) Capacity is average annual flow capacity corresponding to operating condition indicated. Capacity is based on basin volumes and RAS pumping capacity. Realization of capacities indicated would be contingent upon providing corresponding aeration capacities.

(b) Based on Plant 1 RAS rates of 0.8 and 1.0 Mgal/d per clarifier as indicated.

(c) Based on Plant 2 RAS rates of 0.6 and 1.0 Mgal/d per clarifier as indicated. Under all options, except "2+2" clarifiers, the flow from both ditches is combined and evenly distributed to all clarifiers. For the "2+2" clarifier option, each ditch is paired with two clarifiers, in which case, removing a ditch from service also removes both of the paired clarifiers.

(d) Total capacities indicated are based on Plant 1 RAS rates of 0.8 Mgal/d per clarifier. With RAS rates of 1.0 Mgal/d per clarifier, capacities would be increased by about 0.04 Mgal/d. Capacities greater than the future average annual flow of 2.37 Mgal/d are highlighted.

Considering all of the above, the recommended improvements are to add one oxidation ditch and one clarifier at Plant 2 and to increase the RAS pumping rate at Plant 2 to 1 Mgal/d per clarifier. The aeration capacities in the existing oxidation ditches would also have to be upgraded as discussed below. These improvements would:

1. Exceed capacity requirements under the critical design condition (peak flows and loads, 15°C, MCRT = 10 days), providing for robust and reliable operation and flexibility to operate at MCRTs higher than 10 days and/or to accommodate SVIs higher than 200 mL/g.

2. Allow any clarifier at either plant to be taken out of service, even under critical design conditions.

3. Allow any clarifier or any oxidation ditch to be taken out of service during dry weather flow conditions with peak loads.

4. Result in acceptable hydraulic conditions without excessive weir submergence during peak flows.

With one oxidation ditch and two clarifiers at Plant 1 and two oxidation ditches and three clarifiers at Plant 2, the flow and load splits to Plants 1 and 2 with all units in service should be about 35 and 65 percent, respectively. The influent pump station would have to be operated to affect this split. With any ditch or any clarifier out of service, a different flow split would be implemented as appropriate.

Future oxidation ditch aeration capacity requirements were assessed by considering various operating scenarios as shown in Table 11-5. The first row in the table shows aeration requirements under the critical design conditions with a wastewater temperature of 15 °C and an MCRT of 10 days. As shown in the second row, however, aeration requirements would be slightly higher in the summer, particularly if the plant is operated at a higher MCRT then. The final two rows of the table represent the worst-case condition for aeration requirements. When a ditch at one plant is taken out of service, the ditch at the other plant will experience the highest aeration requirement. The high temperature and MCRT values used in this analysis were chosen to represent hot summer conditions, which would result in the highest aeration requirements (lower values were used in the development of Table 11-4 to represent cooler spring and fall conditions, which govern allowable flow capacity).

Based on the data shown in the second row of Table 11-5, the design standard oxygen requirement for the oxidation ditches in Plant 1 and Plant 2 when all oxidation ditches are in service are 7,800 and 7,300 lb/d per ditch, respectively. These are well within the capacity of the existing aerators when all aerators are in service (8,800 lb/d), but exceed the capacity with one aerator out of service (6,600 lb/d). Therefore a standby aerator is needed in each ditch. When an oxidation ditch is taken out of service, the design standard oxygen requirement in each of the two remaining ditches is 10,900 lb/d. This requirement could be met with one additional 30 horsepower aerator per ditch (resulting in a capacity of 11,000 lb/d). The same standby aerator could be used to meet requirements with an aerator out of service or with an oxidation ditch out of service.

To meet the aeration requirements discussed above, two 30 horsepower floating aerators (or the equivalent) should be available for use at the same time in the existing ditches when the proposed new oxidation ditch at Plant 2 is taken out of service. If portable aeration equipment is used, the unit provided for the existing ditch at Plant 2 could also serve as the standby aerator for the proposed new oxidation ditch when one of the permanent aerators in that ditch is out of service.

Based on the criterion that Plant 2 would normally take 65 percent of the total influent flow for both plants, the design peak hour influent flow to Plant 2 would be 0.65 x 7.11 = 4.62 Mgal/d. Since the existing Plant 2 headworks and screen can handle a peak flow of up to 6.2 Mgal/d, no modifications would be needed to increase capacity. However, a new splitter box would have to be added at the screen outlet to split the flow between the existing and new oxidation ditches.

Table 11-5
Aeration Capacity Requirements with Plant Expansion
(One Ditch but no Clarifiers Added at Plant 2)

Units Out of Service	Temp, °C	MCRT, Days	% Flow to Plant 1	% Flow to Plant 2	Plant 1 SOR, lb/d (a)	Plant 2 SOR, lb/d (a)	Plant 2 SOR per Ditch, lb/d (a)
None	15	10	35	65	7,500	13,900	7,000
None	25	14	35	65	7,800	14,500	7,300
Plant 1 Ditch	25	10	0	100	0	21,800	10,900
Plant 2 Ditch	25	10	50 (b)	50 (b)	10,900	10,900	10,900

(a) Peak hour standard oxygen requirement (SOR) based on a dissolved oxygen concentration of 1 mg/L and 50 percent denitrification.

(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.

In summary, expansion of the secondary treatment system would include the following improvements at Plant 2:

- New Splitter Box
- New Oxidation Ditch
- New Clarifier and Associated RAS Pump System
- Existing RAS Pumps Replacement
- Two Standby Aerators (one transferable to Plant 1)

No significant benefit can be gained by increasing the RAS pumping capacity at Plant 1, therefore such improvements are not recommended.

A capital cost estimate for the required secondary treatment improvements is shown in Table 11-6. As indicated, the total cost for all improvements is $6.05 million.

Based on the capacity assessments presented in Table 11-3 and discussed previously in this section, the new splitter box, oxidation ditch, and standby aerators are needed now to allow an existing oxidation ditch to be taken out of service. The new clarifier and RAS pump system is needed before the average annual flow within Discovery Bay exceeds approximately 2.0 Mgal/d, the capacity of the existing system. Since the existing average annual flow is about 1.8 Mgal/d and since it will take a couple of years to plan, design and construct the oxidation ditch and related improvements, the new clarifier and RAS pump system will undoubtedly be needed at the same time or immediately after the ditch is completed. Therefore, all of these improvements should be constructed as one project.

Table 11-6
Secondary Treatment System Expansion In-Kind Cost Estimate

Item	Cost, $ Millions (a)
New Splitter Box at Plant 2 Headworks	0.05
New Oxidation Ditch at Plant 2	1.10
New Clarifier Splitter Box at Plant 2	0.05
New Clarifier at Plant 2	0.65
New RAS Pump Station at Plant 2	0.25
Replace Existing Plant 2 RAS Pumps	0.12
Standby Floating Brush Aerators in Existing Ditches	0.18
Subtotal 1	2.40
Electrical @ 25% of Subtotal 1	0.60
Site Piping @ 10% of Subtotal 1	0.24
Sitework @ 5% of Subtotal 1	0.12
Subtotal 2	3.36
Contingencies @ 20% of Subtotal 2	0.67
Subtotal 3	4.03
General Conditions, Overhead and Profit @ 20% of Subtotal 3	0.81
Total Construction Cost	4.84
Engineering, Admin. and Environmental @ 25%	1.21
Total Capital Cost	6.05

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9,000.

11.3.2 Alternative 2 – Expand Using Salsnes Filter

Under this alternative, one Salsnes filter unit would be installed at each plant. A Salsnes filter is a device that is used to filter raw sewage to remove a portion of the BOD and suspended solids, thereby greatly reducing the load on downstream secondary treatment facilities. A Salsnes filter can provide BOD and suspended solids reductions similar to a primary clarifier. A Salsnes filter was pilot tested at the Discovery Bay Wastewater Treatment Plant in March, 2009. Results from the pilot testing showed TSS removals from 68 to 93 percent and BOD removals from 10 to 49 percent. To be conservative, for this analysis, it is presumed that the Salsnes filter would remove 65 of the TSS and 10 percent of the BOD. The solids removed in the Salsnes filter would be compacted to approximately 40 percent dry solids and hauled to a sanitary landfill for disposal.

All of the capacity assessments prepared for Table 11-3 were repeated with the inclusion of the Salsnes units. The results are shown in Table 11-7. By comparing Table 11-7 to Table 11-3, it can be noted that the effects on the existing secondary treatment systems of adding the Salsnes units are approximately as follows:

- The capacity is increased between 35 and 40 percent.
- The sludge production from the secondary treatment system (not including the solids removed at the Salsnes units) per Mgal/d treated is reduced by about 40 percent.
- The oxygen requirements per Mgal/d treated are reduced by about 9 percent.

To offset the savings in secondary process sludge production and aeration requirements, the Salsnes units produce a very substantial solid waste stream that must be disposed of. For example, with 65 percent removal of the future average annual TSS load of 3,953 lb/d, the dry solids from the Salsnes units would be about 2,600 lb/d. With compaction to 40 percent solids, the wet weight of the solids waste stream would be 3.25 tons per day. Assuming 10 tons per load in a rolloff container, that would require one load of solids to be hauled and disposed of about every three days when the plant reaches full future capacity.

Based on the results shown in Table 11-7, with the Salsnes units added, the capacity of the existing secondary treatment systems would be increased to 2.71 Mgal/d (1.40 Mgal/d at Plant 1 and 1.31 Mgal/d at Plant 2). If the existing RAS pumps were also upgraded, the capacity would be 2.88 Mgal/d (1.44 Mgal/d at each plant).

Based on Scenario 3 in Table 11-7, the peak hour standard oxygen requirement under peak loading conditions would be about 12,200 lb/d per oxidation ditch (DO=1 mg/L, 50% denitrification) at the capacity of 2.88 Mgal/d. The corresponding requirement at 2.37 Mgal/d would be about 10,000 lb/d.

Based on Scenarios 6 and 8, taking an oxidation ditch out of service during dry weather conditions with peak loads would be difficult. Even with the RAS pumps upgraded at both plants, the theoretical capacity with one oxidation ditch out of service would be 2.23 Mgal/d, which is less than the future average annual flow of 2.37 Mgal/d. However, if the MCRT was lowered to 7 days, which should be feasible, the capacity of 2.37 Mgal/d can be satisfied. Also, it is likely that the actual BOD removal by the Salsnes filters will be greater than the conservative value of 10 percent assumed in this analysis.

The peak hour standard oxygen requirement indicated in Table 11-7 for Scenario 8 is 18,900 lb/d (DO=1 mg/L, 50% denitrification), based on the capacity of 2.23 Mgal/d. At 2.37 Mgal/d, the required aeration capacity would be about 20,100 lb/d, which is 11,300 lb/d more than the capacity of the existing rotors. It would be impractical to satisfy this difference with floating brush aerators – it would take five 30 horsepower units, which could not be accommodated in each of the existing ditches.

Because of the above considerations, the Salsnes alternative would not eliminate the need to build a third oxidation ditch. Therefore, use of Salsnes filters would not be cost effective, which eliminates this alternative from further consideration.

Table 11-7
Secondary Treatment System Capacity Assessment Results with Salsnes Filter Added

Scenario	Description	Mixed Liquor Temp, °C	MCRT, days	AAF [a] Capac., Mgal/d	Max Month MLSS, mg/L	Max Month WAS, lb/d	Max Day SOR [b] Per Oxidation Ditch, lb/d		Max Hour SOR [b] Per Oxidation Ditch, lb/d	
							DO = 2 No Denit.	DO = 1 50% Denit.	DO = 2 No Denit.	DO = 1 50% Denit.
1	Existing Plant 1 (RAS = 0.8 Mgal/d per Clarifier)	15	10	1.40	2,400	2,000	12,500	9,400	16,000	11,900
2	Existing Plant 2 (RAS = 0.6 Mgal/d per Clarifier)	15	10	1.31	2,300	1,900	11,600	8,700	14,900	11,100
3	Both Plants Together with RAS Upgrade to 1 Mgal/d per Clarifier	15	10	2.88	2,500	4,100	12,800	9,600	16,400	12,200
4	Both Plants Together with RAS Upgrade to 1 Mgal/d per Clarifier	15	8	3.15	2,300	4,800	13,900	10,500	17,900	13,300
5	Existing Plant 1 (RAS = 0.8 Mgal/d per Clarifier) with One Clarifier Out of Service During Dry Weather Flows	20	8	1.61	2,200	2,300	14,300	10,800	18,400	13,700
6	Existing Plant 2 (RAS = 0.6 Mgal/d per Clarifier) During Dry Weather Flows	20	8	2.02	2,700	2,900	18,000	13,500	23,000	17,100
7	Either Plant with RAS Upgrade to 1 Mgal/d per Clarifier with One Clarifier Out of Service During Dry Weather Flows	20	8	1.64	2,200	2,300	14,500	10,900	18,700	13,900
8	Either Plant with RAS Upgrade to 1 Mgal/d per Clarifier During Dry Weather Flows	20	8	2.23	3,000	3,200	19,800	14,800	25,400	18,900

(a) AAF = Average Annual Flow
(b) SOR = Standard Oxygen Requirement

11.3.3 Consideration of Mixed Liquor and RAS Transfers between Plants

As discussed in Section 11.1, there are existing facilities at Plants 1 and 2 for transferring mixed liquor from one plant to the other, which could be used to allow the clarifiers in one plant to supplement the clarifiers in the other plant in the event that a clarifier is out of service. However, to use this system, there must also be a way to route the corresponding amount of RAS settled in the remote clarifiers to the oxidation ditch from which it originated. Also, there must be provisions for transferring the correct amount of mixed liquor and for returning the correct amount of RAS to keep all oxidation ditches and clarifiers in balance. For example, in the existing situation with two ditches and four clarifiers, if one clarifier is out of service, it would be desired for each of the three clarifiers remaining in service to handle 2/3 of the mixed liquor from one ditch. Therefore, the clarifier remaining in service should handle 2/3 of the mixed liquor from the ditch at that Plant, so only 1/3 of the mixed liquor flow to the clarifier splitter box should be transferred to the other plant. Therefore, the weir length in the spare compartment of the splitter box should only be half as long as the weirs in the compartments normally used. The clarifiers at the plant with both clarifiers in service would handle the equivalent of 4/3 of the mixed liquor from one oxidation ditch. Therefore, 1/4 of the total RAS flow developed in the plant with two clarifiers would have to be returned to the plant with one clarifier.

To implement the system described above, the existing waste activated sludge transfer pipeline from Plant 1 to Plant 2 could be used for returning the required amount of RAS, after adjusting for any desired WAS flows. A new RAS transfer pump system would be required at each plant. It is believed that all of the mechanical equipment and controls required to implement such a system would be too expensive and complex to make them worthwhile.

As an alternative to transferring mixed liquor and RAS as described, the influent flow split to the two plants could be adjusted to transfer a portion of the total flow from the plant with a clarifier down to the other plant, thereby reducing the load on the remaining clarifier. Of course, this would result in reducing the load on the corresponding oxidation ditch, which is undesirable. While this alternative would not fully maximize the treatment capacity of the ditches and clarifiers remaining in service, it is believed that this would be an adequate operation during the time that a clarifier is down.

With the addition of another oxidation ditch and clarifier at Plant 2, the plant will have the ability to operate with any one clarifier out of service, even without mixed liquor and RAS transfers between plants. Therefore, provisions for mixed liquor and RAS transfers between plants are not believed to be necessary and are not recommended.

Section 12
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. At the present time, the Secondary Effluent Lift Station is used to pump the secondary effluent to the downstream Parshall flume and UV disinfection system. However, the Secondary Effluent Lift Station and other facilities in the area were designed to accommodate the future addition of effluent filters ahead of the Parshall flume. In this section, the existing Secondary Effluent Lift Station Facilities are described and improvements required to accommodate future flows and the possible addition of filters are discussed.

12.1 Description of Existing Facilities

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. The large pumps have a design capacity of 2.2 Mgal/d each and the small pumps have a design capacity of 1.25 Mgal/d each. However, those are nominal capacities based on certain operating conditions. Based on hydraulic analyses completed for this investigation, the reliable capacity of the pump station is estimated to be about 6.9 Mgal/d, with one large pump out of service. However, the flow would be about 2.55 Mgal/d per large pump and 0.9 Mgal/d per small pump.

12.2 Future Flow and Head Requirements

As indicated in Section 5, the future peak hour influent flow to the combined wastewater treatment plants is 7.11 Mgal/d. Any flow equalization to be considered in conjunction with possible filters would be located downstream from the Secondary Effluent Lift Station, so this pump station should be capable of handling the entire peak hour flow. However, the peak hour flow at the location of the Secondary Effluent Lift Station could be slightly different than the plant influent flow for two reasons: 1) some peak flow attenuation could occur within the secondary treatment systems, and 2) the flow would be increased by net plant recycle flows, such as potential filter backwash flows and sludge dewatering return flows (to the extent they exceed sludge wasting rates). These flow impacts would be relatively minor and, considering the large uncertainty in the peak flow projection, it is adequate for this analysis to use the influent flow. The analysis could be refined at the time of any future design.

If filters are not added to the wastewater treatment plant, the Secondary Effluent Lift Station will continue to pump to the Parshall flume ahead of the UV disinfection system. If filters are added, pumping to the filter complex (includes coagulation and flocculation facilities) will be required. The water surface elevation at the entry to the filter complex is projected to be around 102 feet, which is about 5 feet higher than the water surface elevation at the entry to the Parshall flume.

12.3 Future Improvements

If filters are not added, it is likely that the existing pumps can remain unchanged. Although the existing capacity of 6.9 Mgal/d is slightly lower than the projected plant influent flow of 7.11 Mgal/d, these flows are essentially the same, considering the uncertainties involved in projecting future peak flows. The adequacy of this capacity could be reviewed in future years as growth approaches buildout and based on historical peak flows occurring at that time. If needed, the capacity of the pump station could be increased by slightly over-speeding the existing pumps using the existing variable frequency drives.

If filters are added, the reliable capacity of the existing pumps would be reduced to about 5.7 Mgal/d, due to the higher head. This is clearly inadequate, so improvements would be needed. Based on preliminary evaluations and discussions with the manufacturer of the pumps, the pump station reliable capacity could be increased to 7.11 Mgal/d by replacing the existing impellers with full-diameter impellers and over-speeding the pumps by about 30 to 100 rpm (depends on the flow split between large and small pumps). This will also require replacing the 5 horsepower motors on the small pumps with 7.5 horsepower motors and the 15 horsepower motors on the large pumps with 20 horsepower motors. The estimated cost for these modifications, including pump removal and installation by a contractor and shipment to and from the pump manufacturer, is $100,000. Although uncertain without a more detailed design evaluation, another $100,000 should be allowed for electrical modifications, possibly including replacement of all variable frequency drives and conductors to the larger motors. Therefore, a budget estimate for the total construction cost is $200,000. With engineering and administration, the total capital cost budget should be about $250,000 (first quarter 2011 cost level).

Section 13
Tertiary Filtration

The Discovery Bay WWTP does not currently include tertiary filtration facilities, but filtration may be needed for more reliable UV disinfection, for possible reclamation reuse or as a result of future more stringent permit requirements. In this Section, an alternative analysis of filtration technologies is presented. Flow equalization ahead of the filters is considered as a possible means of reducing the design capacity and cost of the filters and the downstream disinfection system. Possible layouts and costs for coagulation and flocculation facilities ahead of the filters are also developed.

13.1 Current and Potential Future Requirements

The current discharge permit for the plant includes a monthly average effluent limitation of 30 mg/L for total suspended solids (TSS). Total coliform organisms are limited to 23 most probable number (MPN) per 100 ml as a 7-day median and 240 MPN/100 ml as a value that cannot be exceeded more than once in any 30-day period.

As discussed in Section 8, the plant has not been completely reliable in meeting the effluent coliform limits. To mitigate this issue, the UV disinfection system was recently upgraded and provisions were made to temporarily divert low quality secondary effluent to the sludge lagoons when UV performance would otherwise be compromised. At the time of writing this document, it is unknown whether the improvements will assure adequate disinfection reliability. If not, filtration could be added to greatly improve UV disinfection performance and assure reliable compliance with the existing discharge permit limits for total coliform.

In addition to the possibility of providing filters to assure more reliable compliance with the existing permit, it is possible that filters may be required in the future because of more stringent requirements for discharge into Old River or to allow unrestricted reclamation reuse of the effluent.

Effluent quality requirements for water recycling have been established by the California Department of Public Health (CDPH) and are contained in Title 22, Chapter 4 of the California Code of Regulations (Title 22). In accordance with Section 60304 of Title 22, wastewater effluent used for landscape irrigation in areas of public exposure and effluent used for irrigation of food crops where the water contacts the edible portions of the crop must be "disinfected tertiary recycled water", which requires filtration in accordance with the following requirements (Section 60301.320):

> "Filtered wastewater" means an oxidized wastewater that meets the criteria in subsection (a) or (b):
>
> > (a) Has been coagulated and passed through natural undisturbed soils or a bed of filter media pursuant to the following:

(1) At a rate that does not exceed 5 gallons per minute per square foot of surface area in mono, dual or mixed media gravity, upflow or pressure filtration systems, or does not exceed 2 gallons per minute per square foot of surface area in traveling bridge automatic backwash filters; and

(2) So that the turbidity of the filtered wastewater does not exceed any of the following:

(A) An average of 2 NTU within a 24-hour period;

(B) 5 NTU more than 5 percent of the time within a 24-hour period;

and

(C) 10 NTU at any time.

(b) Has been passed through a microfiltration, ultrafiltration, nanofiltration, or reverse osmosis membrane so that the turbidity of the filtered wastewater does not exceed any of the following:

(1) 0.2 NTU more than 5 percent of the time within a 24-hour period; and

(2) 0.5 NTU at any time.

In accordance with Section 60301.230, total coliform organisms in disinfected tertiary recycled water must not exceed 2.2 MPN/100 ml as a 7-day median, 23 MPN/100 ml more than once in 30 days or 240 MPN/100 ml at any time.

Even if water recycling is not practiced, there is a potential that future permit requirements for discharge to Old River could specify treatment equivalent to that required for recycling as indicated above.

13.2 Design Flows

Plant influent design flows and loads are developed in Section 5. The key influent flow criteria that impact the design of the tertiary filtration system are as follows:

Average Day Maximum Month Flow (ADMMF)	2.37 Mgal/d
Peak Day Wet Weather Flow (PDWWF)	4.74 Mgal/d
Peak Hour Wet Weather Flow (PHWWF)	7.11 Mgal/d

The final design flows to the tertiary treatment system will include the flows indicated above, plus in-plant recycle flows, such as filter backwash water and sludge dewatering return flows (to the extent they exceed sludge wasting rates). The return flows would be relatively minor and are neglected for this analysis.

The cost of the tertiary treatment system and the downstream UV disinfection system will depend heavily on the maximum flows for which these facilities are to be designed. One option would be to design these systems to handle the full PHWWF. However, since this flow is much greater than the PDWWF, there is a potential to realize substantial savings in facilities requirements and costs by flow equalization. The option of providing flow equalization to limit the maximum flow to the filters (and downstream facilities) to the PDWWF of 4.74 Mgal/d is considered in this section.

13.3 Flow Equalization Facilities

The recommended method for implementing flow equalization upstream from the filters would be to divert excess peak flows (flows greater than 4.74 Mgal/d) upstream of the coagulation and flocculation facilities to a lined earthen basin using a downward opening weir gate. Then, after peak flows subside, the stored water would be drained back to the Secondary Effluent Lift Station at a controlled rate using a modulating valve.

As a general guideline, the equalization basin volume should be about 25 percent of the total peak day flow, or about 1.2 Mgal. Possible basin configuration information is presented in Table 13-1. The basin would be built partly above grade and partly below grade to suit hydraulic grade requirements.

Table 13-1
Possible Equalization Basin Configuration

Parameter	Value
Basin Volume, Mgal	1.2
Basin Water Depth, ft	8
Freeboard, ft	2
Total Depth, ft	10
Side Slope (H:V)	3:1
Length and Width at Bottom, ft	120
Length and Width at Max. Water Surface, ft	168
Length and Width Inside Berm Top, ft	180
Length and Width Outside Berm Top, ft	204
Liner Type	60 ml HDPE

13.4 Teritary Filtration Alternatives

A number of filtration technologies could be utilized to produce tertiary effluent consistent with Title 22 regulations for unrestricted reuse of wastewater. Alternatively, these same filtration technologies could be used if filtration is to be provided without reclamation. The technologies generally can be categorized as granular media filtration, cloth-media surface filtration, other media filtration, and membrane filtration. Membrane filtration is excluded from this analysis because it is much more expensive than the other filtration systems.

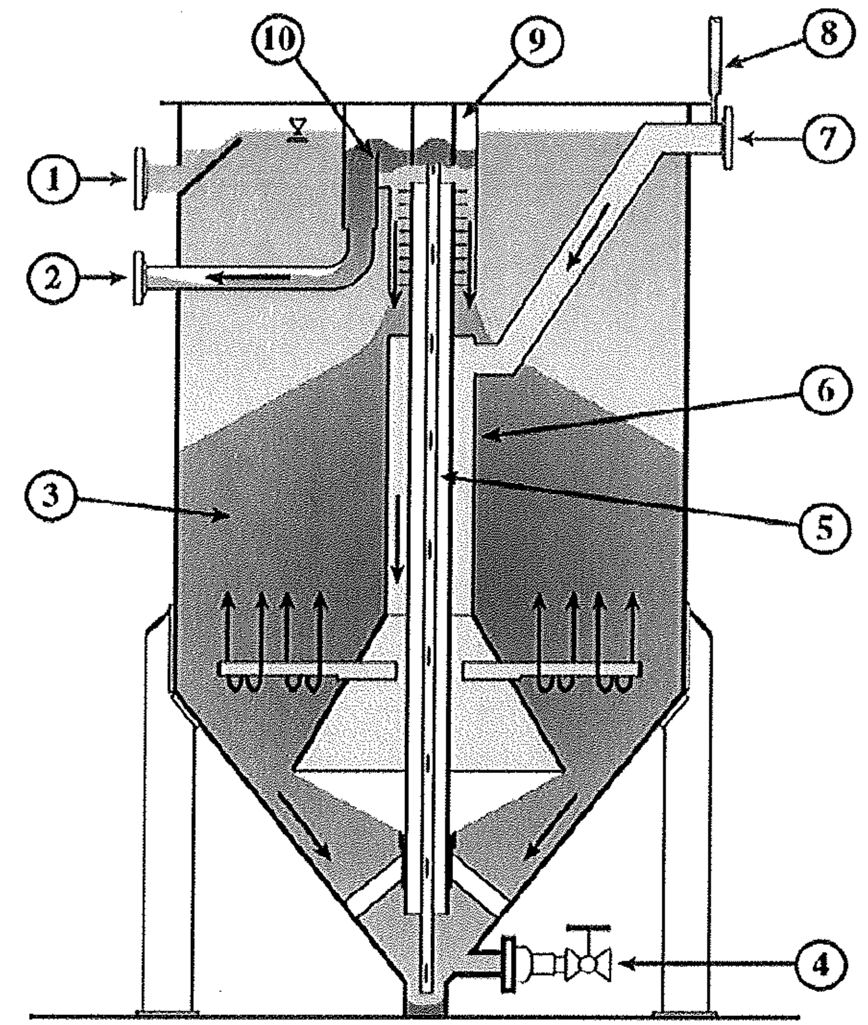
Based on studies and applications in other areas, it is known that continuous backwash sand filters (a granular media filter) and cloth disk filters are among the most cost-effective options. Because of this and because Plant 2 was planned for the future implementation of continuous backwash filters, these two alternatives are considered below. A third alternative, somewhat similar to the cloth disk filter, but using stainless steel micromesh as the filter medium, is considered also.

13.4.1 Continuous Backwash Sand Filters

Continuous backwash sand filters are arranged for upward flow through a deep media bed. Influent enters the center of the filter through a central feed chamber. The central feed chamber has a series of radial arms to evenly distribute the influent flow to the media bed near the bottom of the filter. As the water flows upward through the filter, solids are removed. Filtrate exits the filter near the top and flows over a fixed weir plate that maintains a constant level. The filter media and captured solids within the filter are constantly in motion downward to the intake of an airlift pump in a recessed chamber below the filter inlet radial arms. From there, the media is lifted back to the top of the filter. The high energy, turbulent upward flow inside the airlift provides a scrubbing action that effectively separates the sand and the captured solids before discharging them in the washbox at the top of the filter. The washbox is a baffled chamber that allows for countercurrent washing and gravity separation of the filter media and the captured solids. Media cleaning is accomplished utilizing filtered water from the upper chamber of the filter. Regenerated filter media is returned to the top of the filter bed as it falls by gravity through the washbox. An adjustable V-notch weir directs the reject flow out of the filter, carrying concentrated captured solids to a suitable disposal point. Figure 13-1 shows a schematic diagram of the continuous backwash filter.

13.4.2 Cloth Disk Filters

AquaDisk by Aqua Aerobics is a cloth disk filter system that has been used extensively in California and is the basis of this investigation. AquaDisk filters consist of a nylon fiber, random weave pile fabric supported by open frame structures that are arranged in disks (see Figure 13-2). During normal operation the disks are submerged completely in the water. Water flows by gravity from the outside of the disks through the filter cloth into the center of the disks to a central collection header. As solids accumulate on the media, a mat forms on the surface, headloss increases, and the liquid level in the tank increases. Typical headloss through the filter is between eight and ten inches, with a maximum of 12 inches. When the water reaches a certain level (or at a set time), the backwash cycle is initiated. Backwash is accomplished by the use of suction lines connected to backwash pumps on one end and to backwash 'shoes' on the other end. As the disk rotates, the backwash shoes exert a partial vacuum against a small portion of the disk. The vacuum draws filtered water through the disk in the opposite direction to normal filtration, the fibers of the cloth are raised, and trapped solids are released. During backwash, filtration is not interrupted on disks not undergoing backwash. Typical average backwash water use rates are less than 2-3% of the influent flow. Because of quiescent conditions in the tanks, heavy solids tend to settle to the bottom and periodically have to be pumped from the tank. The AquaDisk pile cloth filters were designed for the tertiary treatment of effluent from conventional activated sludge secondary treatment and were granted Title 22 approval by CDPH in 2002.



1. Filtrate
2. Reject
3. High Quality Silica Media
4. Drain
5. Airlift
6. Central Feed Chamber
7. Influent
8. Headloss Sight Gauge
9. Splash Guard
10. Adjustable V-Notch Weir

Figure 13-1
General Schematic of Parsons DynaSand Continuous Backwash Sand Filter

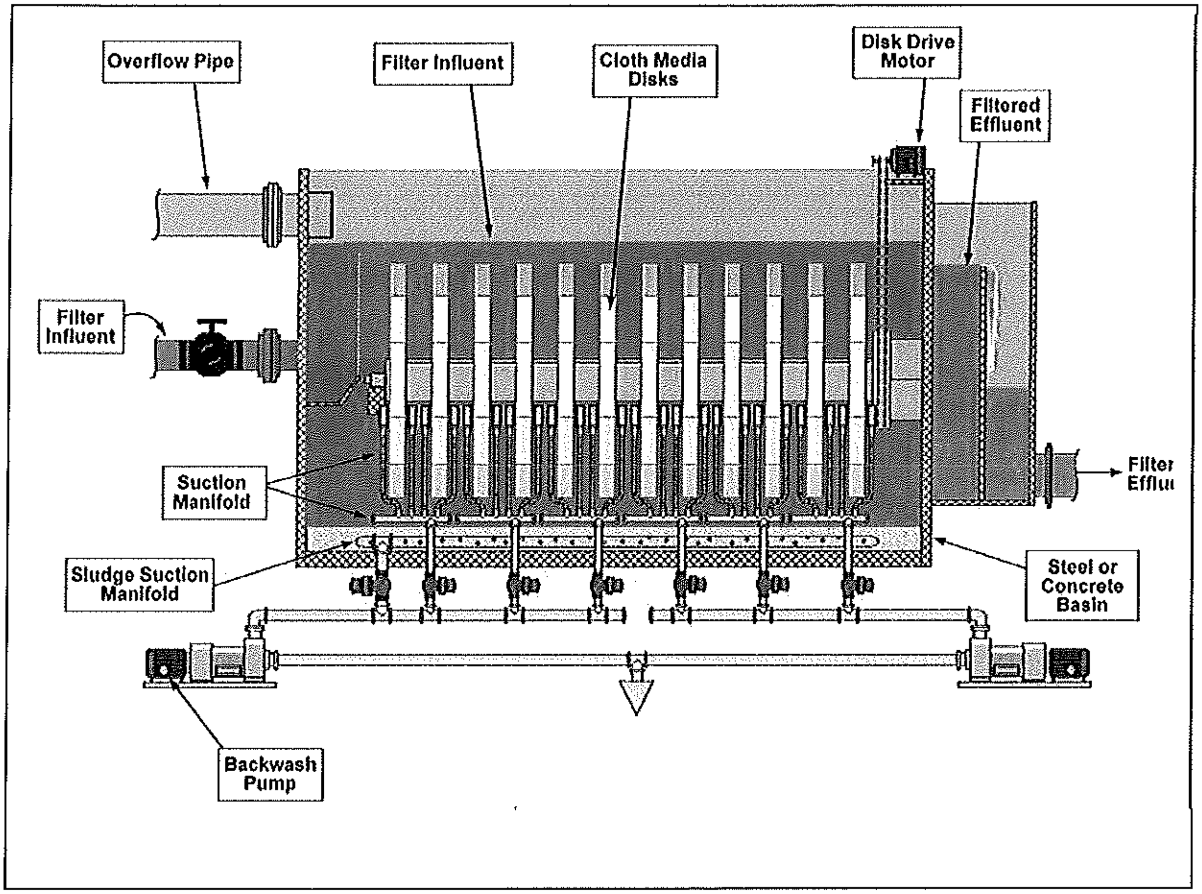
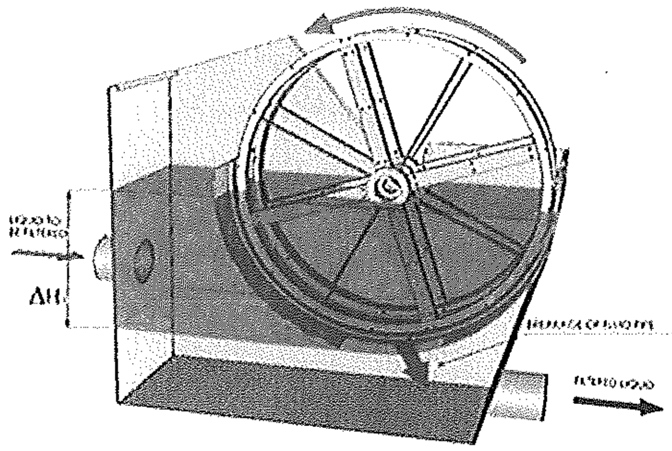


Figure 13-2
General Schematic of AquaDisk Cloth-Medium Surface Filter

13.4.3 SST Micromesh Disk Filters

The Title 22 approved Ultrascreen® disk filter is manuafautred by Nova Water Technologies. The Ultrascreen® is an inside-out surface filtration system that consists of continuously rotating disk filters made of woven stainless steel mesh. The influent flow is directed into the center "inside" of the disk and flows out through the filter mesh to the effluent outlet (see Figure 13-3). The disks are continuously rotating throughout the filtration cycle as the filtration mesh is fed at angles less than 90 degrees, to achieve "dynamic tangential filtration". As shown in Figures 13-3, the effluent side of the filter is not partially submerged like other disk filtration technologies. Free filtrate discharge occurs with the Ultrascreen®.

The Ultrascreen Microfilter®
Disk Submergence



The Ultrascreen Microfilter®
Flow Patterns

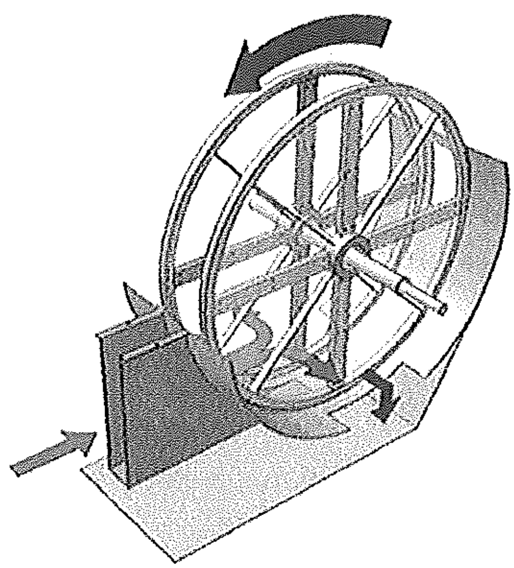


Figure 13-3
General Schematic of NOVA Ultrascreen SST Micromesh Filter

The disk of the Ultrascreen® is made of AISI 316 stainless steel micronic screen mesh. Due to the rotation of the disk and the "dynamic tangential filtration", it is claimed by the manufacturer that particles smaller than 10 micrometers (μm) can be removed with the 20 μm nominal size mesh screen. It is also claimed by the manufacturer that "dynamic tangential filtration" will lead to less solids accumulation on the media which allows the filter to operate at higher hydraulic loading rates while still meeting effluent turbidity limits. A proprietary silicone rubber blend seal sits against the disk sides and prevents short-circuiting. The silicone rubber blend seal allows the disks to rotate while preventing untreated effluent from bypassing the system.

13.4.4 Design Criteria and Comparison of Alternatives

Design criteria for the three filtration alternatives are shown in Tables 13-2 and 13-3 for scenarios with and without equalization, respectively. Advantages and disadvantages of the filtration alternatives are presented in Table 13-4.

13.5 Coagulation and Flocculation Requirements

Based on Title 22 regulations, coagulation (chemical addition to promote particle agglomeration) facilities are required (but may not need to be used all of the time). For effective coagulation to occur, it is essential that the coagulant chemicals be mixed rapidly and completely with the entire wastewater flow stream. After coagulation, sufficient contact time and gentle mixing should be provided to allow a visible floc to form prior to filtration. Use of a chemical flocculant at this point may be beneficial. Although not specifically required in Title 22, flocculation basins are recommended to promote adequate floc development. A rapid mix chamber followed by a two-stage flocculation basin is recommended. Design criteria for the rapid mix chamber and the flocculation basins are provided in Table 13-5 (with upstream flow equalization) and Table 13-6 (without upstream flow equalization).

Table 13-2
Filter Design Criteria – Scenario 1 (With Upstream Flow EQ)

System Components	Continuous Backwash Sand Filter	Cloth-Disk Filter [(a)]	SST Micromesh Disk Filter
Average Hydraulic Loading Rate, gpm/ft^{2a}	1.83	1.29	3.74
Peak Hydraulic Loading Rate, gpm/t^2	3.65	2.55	7.48
Max Hydraulic Loading Rate, gpm/ft^2 [a]	4.39	3.8	14.96
Number of Units/Cells	6 (5 duty and 1 standby)	3 (2 duty and 1 standby)	2 (1 duty and 1 standby)
Number of Modules per Cell	3	NA	NA
Number of Disk per Unit	NA	8 [(a)]	20
Total Filter Area ft^2	900	1291.2 [(a)]	440
Maximum Headloss, in	36	12	25.6
Backwash Requirements / Reject Water, %	3-5	1.85	0.5 - 1

(a) Cloth-Disk Filter sizes are same for Scenario 1 (with EQ) and Scenario 2 (without EQ).

Table 13-3
Filter Design Criteria – Scenario 2 (Without Upstream Flow EQ)

System Components	Continuous Backwash Sand Filter	Cloth-Disk Filter [(a)]	SST Micromesh Disk Filter
Average Hydraulic Loading Rate, gpm/ft^{2a}	1.37	1.27	3.12
Peak Hydraulic Loading Rate, gpm/t^2	4.1	3.8	9.35
Max Hydraulic Loading Rate, gpm/ft^2 [a]	4.7	5.7	14.03
Number of Units/Cells	8 (7 duty and 1 standby)	3 (2 duty and 1 standby)	3 (2 duty and 1 standby)
Number of Modules per Cell	3	NA	NA
Number of Disk per Unit	NA	8 [(a)]	8
Total Filter Area ft^2	1200	1291.2 [(a)]	528
Maximum Headloss, in	36	12	25.6
Backwash Requirements / Reject Water, %	3-5	1.85	0.5 - 1

(a) Cloth-Disk Filter sizes are same for Scenario 1 (with EQ) and Scenario 2 (without EQ).

Table 13-4
Advantages and Disadvantages of Filtration Alternatives

	Continuous Backwash Sand Filter	Cloth-Disk Filter	SST Micromesh Disk Filter
Advantages	▪ Extensive track record; longer operating history than other options ▪ Minimal mechanical equipment. ▪ Highly reliable ▪ Excellent downstream UV disinfection performance	▪ Low headloss. ▪ Low backwash flow ▪ Compact footprint compared to granular medium filtration	▪ Approved under higher loading rate. ▪ Smaller space requirements than other alternatives ▪ Low backwash flow
Disadvantages	▪ Process air required ▪ Relatively high backwash flow ▪ Requires concrete cells	▪ Good chemical conditioning may be required to ensure reliable downstream UV system performance	▪ No full-scale installations in California ▪ Performance of downstream UV system unknown

Table 13-5
Preliminary Rapid Mix and Flocculation System Design Criteria (With Upstream Flow Equalization)

Parameter	Value
Peak Flow, Mgal/d	4.74
Average Flow, Mgal/d	2.37
Rapid Mix	
Type	Mechanical
Orientation	Vertical
Impeller Type	Turbine
Detention Time @ Peak Flow, sec	15
Detention Time @ Average Flow, sec	30.0
Volume, gal	823
Velocity Gradient "G", sec-1	700
Power Required, HP	2.7
Depth (incl. 2 ft freeboard), ft	8
Length, ft	4.3
Width, ft	4.3
Flocculation Basins	
Type	Mechanical
Orientation	Vertical
Impeller Type	Paddle
Total Detention Time @ Peak Flow, min	17
Total Detention Time @ Average Flow, min	34
Total Volume, gal	55960
No. of Basins	2.0
Depth (incl. 2 ft freeboard), ft	16
Length, ft	16.3
Width, ft	16.3
Basin 1 "G", sec-1	80
Basin 1 Power Requirement, HP	1.2
Basin 2 "G", sec-1	60
Basin 2 Power Requirement, HP	0.7

Table 13-6
Preliminary Rapid Mix and Flocculation System Design Criteria (Without Upstream Flow Equalization)

Parameter	Value
Peak Flow, Mgal/d	7.11
Average Flow, Mgal/d	2.37
Rapid Mix	
Type	Mechanical
Orientation	Vertical
Impeller Type	Turbine
Detention Time @ Peak Flow, sec	15
Detention Time @ Average Flow, sec	45.0
Volume, gal	1235
Velocity Gradient "G", sec-1	700
Power Required, HP	4.0
Depth (incl. 2 ft freeboard), ft	8
Length, ft	5.2
Width, ft	5.2
Flocculation Basins	
Type	Mechanical
Orientation	Vertical
Impeller Type	Paddle
Total Detention Time @ Peak Flow, min	17
Total Detention Time @ Average Flow, min	51
Total Volume, gal	83940
No. of Basins	2.0
Volume per Basin, cu. ft.	5611
Depth (incl. 2 ft freeboard), ft	16
Length, ft	20.0
Width, ft	20.0
Basin 1 "G", sec-1	80
Basin 1 Power Requirement, HP	1.8
Basin 2 "G", sec-1	60
Basin 2 Power Requirement, HP	1.0

13.6 Filtration Alternative Costs and Selection of Preferred Alternative

Estimated capital, annual and present worth costs for the three filtration alternatives, combined with equalization, coagulation, and flocculation facilities are presented in Table 13-7. The estimates are based on the following assumptions:

- First quarter 2010 cost level, ENR 20-Cities CCI = 9000.
- Poly aluminum chloride (PAC) is the assumed coagulant at a dose of 10 mg/L. PAC usage is assumed to be 30 days per year for the continuous backwash alternative and 45 days per year for the other two alternatives. Unit cost of PAC is $1/gal.
- Continuous backwash filter will include Ecowash system, an enhancement that reduces backwash and energy requirements.
- The present worth costs are based on 20 years at inflation-adjusted discount rate of 3% and present worth factor of 14.88.
- Basis of labor cost is $60/hr.
- Unit power cost is $0.12/kWh.

Based on the costs shown in Table 13-7 and the extensive and favorable track record of continuous backwash sand filters ahead of UV disinfection, the continuous backwash sand filter alternative with flow equalization is recommended. It is noted that flow equalization will result in substantial cost savings for UV filtration and final effluent pumping, which are not reflected in Table 13-7.

A preliminary layout of the coagulation, flocculation, and filtration facilities is shown in Figure 13-4.

Table 13-7
Filtration Alternative Cost Analysis

Item	Cost for Indicated Alternative, $ [a]					
	Scenario 1 (With Flow Equalization)			Scenario 2 (Without Flow Equalization)		
	Continuous Backwash	Cloth Disk [b]	SST Mesh Disk	Continuous Backwash	Cloth Disk [b]	SST Mesh Disk
Capital Cost						
Equalization Basin, Piping, Valves and Controls	270,000	270,000	270,000	0	0	0
Concrete structures and canopy (if applicable)	250,000	210,000	180,000	310,000	210,000	225,000
Piping, metals, and ancillaries	330,000	375,000	340,000	440,000	375,000	452,000
Filter Equipment, Installed	1,251,000	1,796,000 [b]	975,000	1,552,000	1,796,000 [b]	1,065,000
Subtotal 1	2,101,000	2,651,000	1,765,000	2,302,000	2,381,000	1,742,000
Elect/Instrum, 25% of Subtotal 1, Unless Noted Otherwise	525,000	663,000	441,000	575,000	595,000	436,000
Sitework, 5% of Subtotal 1 Unless Noted Otherwise	105,000	133,000	88,000	115,000	119,000	87,000
Site Piping, 10% of Subtotal 1, Unless Noted Otherwise	210,000	265,000	177,000	230,000	238,000	174,000
Subtotal 2	2,941,000	3,712,000	2,471,000	3,222,000	3,333,000	2,439,000
General Conditions, Overhead and Profit, 20%	588,000	742,000	494,000	645,000	667,000	488,000
Subtotal 3	3,529,000	4,454,000	2,965,000	3,867,000	4,000,000	2,927,000
Contingencies, 20%	706,000	891,000	593,000	773,000	800,000	585,000
Total Construction Cost	4,235,000	5,345,000	3,558,000	4,640,000	4,800,000	3,512,000
Engineering and Administration, 25%	1,059,000	1,336,000	890,000	1,160,000	1,200,000	878,000
Total Capital Cost	5,294,000	6,681,000	4,448,000	5,800,000	6,000,000	4,390,000
Annual Costs						
Labor	9,360	9,360	9,360	10,920	10,920	10,920
Power	11,040	600	4,440	14,683	840	5,905
Chemicals	11,859	17,789	17,789	17,789	26,684	26,684
Maintenance Materials	3,500	5,200	6,500	5,000	6,500	8,645
Total Annual Cost	35,759	32,949	38,089	48,392	44,944	52,154
Present Worth Costs						
Present Worth of Annual Costs	532,000	490,000	567,000	720,000	669,000	776,000
Total Present Worth Cost	5,826,000	7,171,000	5,015,000	6,520,000	6,669,000	5,166,000

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9000.
(b) Cloth-Disk Filter sizes are same for Scenario 1 (with EQ) and Scenario 2 (without EQ).

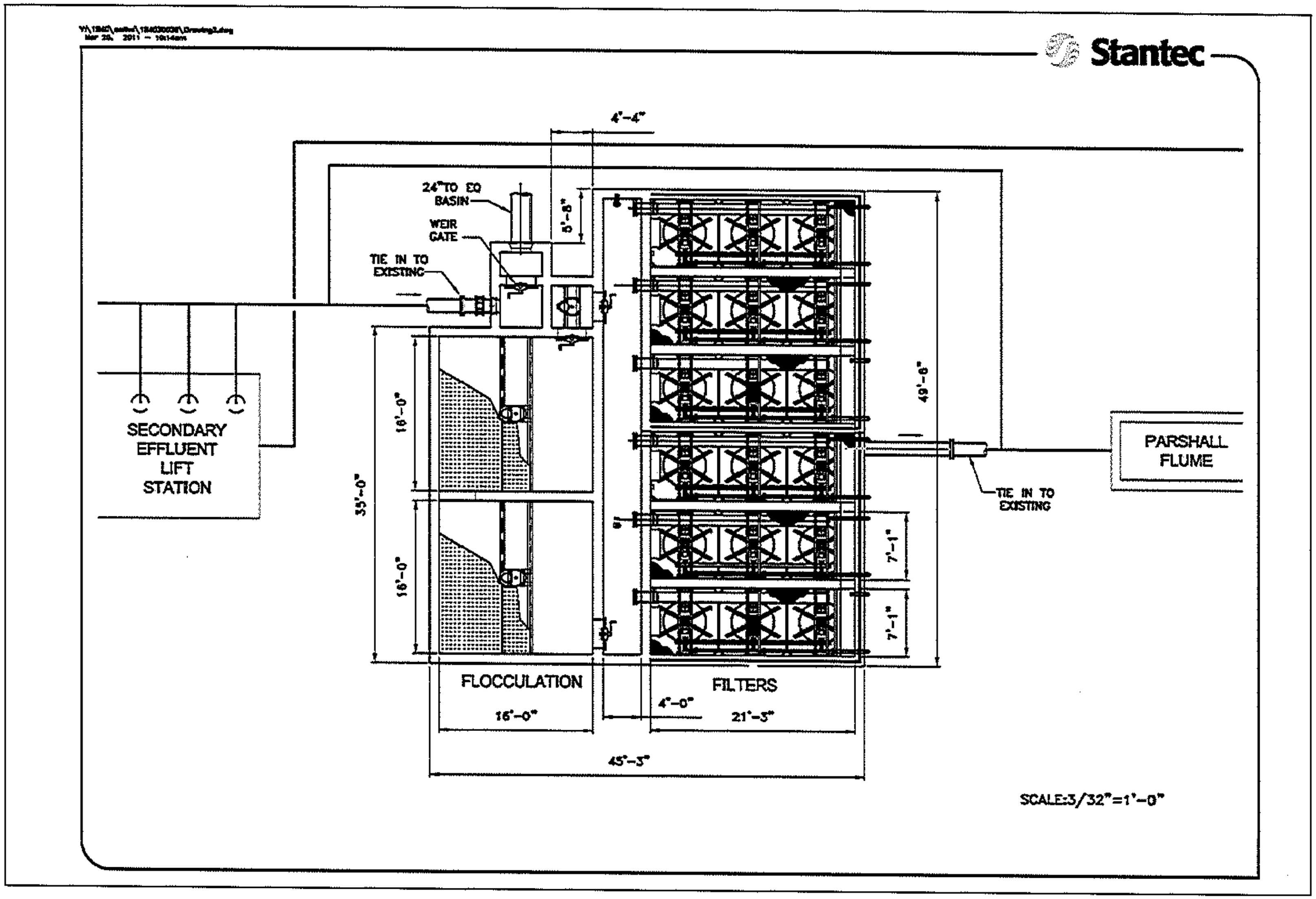


Figure 13-4
Possible Continuous Backwash Filter Layout

Section 14
UV Disinfection

Ultraviolet (UV) disinfection is currently employed at the Discovery Bay Wastewater Treatment Plant as the means for meeting effluent coliform limits specified in the plant's National Pollution Discharge Elimination System (NPDES) permit for discharge into Old River. The permit requirements for total coliform and recent violations of these requirements are discussed in Section 8. As mentioned in Section 8, recent (2010) improvements to the UV disinfection system and related facilities have been made to improve compliance with the permit, but it is not yet known whether an adequate level of disinfection system reliability can be obtained without further improvements, possibly including effluent filtration.

In this section, the existing UV facilities and the recent improvements to them are discussed in more detail. Then, water quality and UV dose requirements, as well as other UV system design criteria and costs are developed for three potential scenarios for UV system expansion.

14.1 Existing UV Facilities

Currently, the UV system at the Discovery Bay Wastewater Treatment Plant includes two UV channels. The first channel contains TrojanUV3000 equipment that was installed in 2000. The second channel contains TrojanUV3000Plus equipment that was installed in 2010 to replace the previous Bailey/Fisher and Porter UV system. While both systems currently in operation are manufactured by Trojan and operate on similar principals, the capacities of the two UV systems are quite different, as indicated in Table 14-1.

Table 14-1
Existing UV System Capacity

Condition	Peak Flow Capacity, Mgal/d [a]		
	TrojanUV3000	TrojanUV3000Plus	Total
All Banks in Service [b] [c]	1.3	4.8	6.1
One Bank in Each Channel Off-Line [c]	0.9	3.2	4.1
One Bank in Each Channel Off-Line, with Dose Safety Factor [d]	0.6	2.8	3.4

(a) Capacities calculated based on UV Dose = 80 mJ/cm^2 (before safety factor), UV Transmittance = 55%, and total coliform = 23 MPN/100 mL. In order to realize these capacities, the turbidity of the secondary effluent should generally be less than 10 NTU (see discussion in Section 14.2).

(b) Total number of banks is 3 for UV3000 and 4 for UV3000Plus.

(c) No safety factor.

(d) Dose safety factor for UV system performance variability = 1.25 for UV3000 and 1.1 for UV3000Plus

As indicated in the footnotes to Table 14-1, the capacities indicated in the table are based on an applied UV dose of 80 mJ/cm^2 and are conditioned on having a secondary effluent turbidity generally less than 10 NTU. The bases of these criteria are discussed later in this section. The capacities are also based on an assumed UV transmittance of 55%, which is the default value required to be used for the design in the absence of long-term site-specific data.

The reliable capacity of the UV disinfection system should be based on the capacity with one bank in each channel off-line. Furthermore, unless on-site viral bioassay testing is completed to validate the capacities given without dose safety factors, it would be prudent to apply dose safety factors as was done for the last row in Table 14-1.

Currently, only the UV3000Plus system is generally being used. To allow operation of both UV channels at the same time, provisions would have to be made for splitting the total UV system flow to the two channels in proportion to capacity. This could be done by blocking a portion of the influent weir to the UV3000 system.

As indicated in Table 14-1, the peak flow capacity using only the UV3000Plus system with one bank off-line is 3.2 Mgal/d without a dose safety factor and 2.8 Mgal/d with a dose safety factor of 1.1. Assuming the typical peak hourly flow on any given day could be about 1.5 times the average flow for the day, the typical peak hourly flow associated with the current average annual flow would be 1.5 x 1.8 Mgal/d = 2.7 Mgal/d. Similarly, the typical peak hourly flow associated with the current peak month flow would be 1.5 x 1.98 Mgal/d = 2.97 Mgal/d. These are both within the capacity of the existing UV3000Plus system with all banks on-line and with one bank off-line. However, with a safety factor applied, the typical peak hourly flow associated with the peak month flow would slightly exceed the capacity of the UV3000Plus system with one bank off-line. Nevertheless, it is apparent that the UV3000Plus system alone should be adequate almost all of the time for existing flows.

The current extreme peak hour flow of 5.4 Mgal/d cannot be accommodated using only the UV3000Plus system, even with all channels on-line and without a safety factor (capacity = 4.8 Mgal/d). However, that does not necessarily mean that passing that flow through the UV3000Plus system would result in an effluent total coliform limit violation. The permit allows one excursion per month above an effluent total coliform level of 240 MPN/100 mL. Also, to meet the 7-day median limit of 23 MPN/100 mL, up to half of the coliform tests in a given week could be above 23 MPN/100 mL. While limited statistical excursions above 23 MPN/100 mL can be tolerated, it is prudent to assess the UV system capacity based on continuously meeting the 7-day median total coliform limit. Accordingly, the applicable peak flow capacities indicated in Table 14-1 should not be exceeded. To the extent that secondary effluent flows exceed these capacities, excess peak flows should be trimmed by diverting to the sludge lagoons or to an equalization basin, such as discussed later in this section.

14.2 Possible Scenarios for UV System Expansion

Three scenarios for UV system expansion have been identified as follows:

> Scenario 1: Continuation of existing conditions, including UV disinfection to meet a weekly median total coliform limit of 23 MPN/100 mL after secondary treatment.

Scenario 2: UV disinfection to meet a weekly median total coliform limit of 23 MPN/100 mL, but with effluent filtration provided to improve UV system performance.

Scenario 3: UV disinfection to meet a weekly median total coliform limit of 2.2 MPN/100 mL after effluent filtration. This scenario is based on the possible adoption of more stringent effluent limitations for discharge to Old River or for unrestricted reuse of the wastewater effluent for irrigation.

Key permit effluent limitations, pre-disinfection water quality requirements and UV dose requirements for the three scenarios are shown in Table 14-2 and are discussed further below.

Scenario 1 represents a continuation of existing conditions, whereby the wastewater continues to receive secondary treatment for discharge into Old River under current permit requirements. Alternatively, the effluent could be used for irrigation of fodder crops. As indicated in Table 14-2, the weekly average turbidity of the influent to the UV disinfection system should be about 10 NTU or lower to assure reliable compliance with a 7-Day median total coliform limit of 23 MPN/100 mL at a UV dose of 80 mJ/cm^2. A precise relationship between the turbidity level, the UV dose and the disinfected effluent total coliform level is not known. In site-specific testing conducted in mid-2010, a UV dose of 80 mJ/cm^2 resulted in total coliform levels less than 23 MPN/100 ML when the turbidity was 10 NTU or lower, but not when turbidities were about 20 NTU or higher. Turbidities between 10 and 20 NTU were not tested. Another key result of the study is that a UV dose of 100 mJ/cm^2 did not generally provide better disinfection performance than a dose of 80 mJ/cm^2, regardless of the turbidity. Accordingly, under this scenario, it is planned to use a target UV dose of 80 mJ/cm^2 and to divert secondary effluent to the sludge storage lagoons if the turbidity exceeds an adjustable setpoint value. The appropriate setpoint value will have to be determined, but will likely be between 10 and 20 NTU.

The operations as described above for Scenario 1 are consistent with newly established existing conditions. As indicated in Section 8, it is not currently known whether these operations will be successful in providing reliable compliance with the effluent total coliform limit. If not, effluent filtration could be required, which is the basis of Scenario 2.

Under Scenario 2, effluent filtration is provided, not to meet more stringent effluent permit limits on BOD, TSS, and/or turbidity, but to assure reliable compliance with effluent total coliform limits with UV disinfection. However, once filters are added, the plant will be able to meet more stringent requirements for BOD, TSS, and turbidity and, for that reason, more stringent requirements may be imposed. With effluent filters added, the UV dose needed for disinfection to a total coliform limit of 23 MPN/100 mL would be only 40 mJ/cm^2.

Scenario 3 is based on producing “disinfected tertiary recycled water” in accordance with State of California Department of Public Health Water Recycling Criteria (Title 22) for reuse where there is public exposure, or the equivalent effluent quality for river discharge. In this case, there would be very stringent permit effluent limitations on BOD, TSS, and turbidity, as indicated in Table 14-2. The 7-day median total coliform limit would be reduced to 2.2 MPN/100 mL. The UV dose requirement for Scenario 3 is 100 mJ/cm^2.

Table 14-2
Permit Effluent Limitations, Water Quality Requirements and UV Dose for Three Scenarios

Parameter	Scenario 1: 23 MPN/100 mL, No Filters	Scenario 2: 23 MPN/100 mL With Filters	Scenario 3: 2.2 MPN/100 mL With Filters
Permit Effluent Limitations:			
BOD, 30-Day Avg., mg/L	20	20 [a]	10
TSS, 30-Day Avg., mg/L	30	30 [a]	10
Turbidity, Weekly Avg., NTU	NA	NA [a]	2
Turbidity, Daily Maximum, NTU	NA	NA [a]	5
Total Coliform, 7-Day Median, MPN/100 mL	23	23	2.2
Total Coliform, Exceed Once in 30 Days, MPN/100 mL	240	240	23
Pre-Disinfection Water Quality:			
BOD, 30-Day Avg., mg/L	20	10	10
TSS, 30-Day Avg., mg/L	30	10	10
Turbidity, Weekly Avg., NTU	10+/-	2	2
Turbidity, Daily Maximum, NTU	NA	5	5
UV Dose, mJ/cm^2	80	40	100

(a) Permit limits for BOD, TSS, and turbidity may be made more stringent because the plant's ability to meet more stringent requirements with filters.

(b) The UV dose is controlled by the NPDES permit requirements for surface water discharge and Waste Discharge Permit/Title 22 requirements for reclamation reuse of the wastewater.

14.3 Future UV System Design Criteria

Design criteria for UV system expansion are considered in the following paragraphs.

14.3.1 Flow

Future flow projections are presented in Section 5. As indicated in that section, the future average annual, peak day and peak hour flows are 2.37, 4.74 and 7.11 Mgal/d, respectively. However, it is recommended that flow equalization be implemented upstream from the possible future filters and the UV system. With flow equalization, the peak flow to the filters (if used) and the UV system would be limited to the peak day average flow of 4.74 Mgal/d. Under Scenario 1, the cost of the equalization facilities would be more than offset by the cost savings for UV disinfection and the Export Pump Station (the impact of equalization on the Export Pump Station is discussed in Section 7). Under Scenarios 2 and 3, with filtration included, equalization is even more cost-effective. The equalization facilities are discussed in Section 13.

14.3.2 UV Transmittance and Turbidity

The effectiveness of UV light in inactivating bacteria and viruses is impacted by both the transmittance and turbidity of the water. Transmittance is the ability of the effluent to transmit ultraviolet light. Factors known to affect UV transmittance include dissolved organics, dissolved iron, color, and turbidity. Turbidity is a measure of the ability of a solution to scatter light. Light scattering is usually caused by the presence of small particles. A transmittance of 55 percent is specified as a default in the National Water Research Institute (NWRI) Guidelines if limited or no data on the existing wastewater effluent is available and is assumed for Scenario 1. Higher transmittance of the wastewater can drastically reduce the size of the UV system needed, saving both capital and operating costs. It is believed that a UV transmittance of 65% can be demonstrated with effluent filtration and is assumed for Scenarios 2 and 3.

14.3.3 UV Dose Requirements

As noted in Table 14-2, the UV dose requirements are 80 mJ/cm^2 , 40 mJ/cm^2 and 100 mJ/cm^2 for Scenarios 1, 2 and 3, respectively.

14.4 UV System Improvements and Costs

The existing UV disinfection system can meet the low dose requirements indicated for Scenario 2 at the future equalized peak day flow of 4.74 Mgal/d. Therefore, no improvements are required under Scenario 2.

For both Scenarios 1 and 3, the recommended improvements are the same. In both cases, the existing UV3000 system in one channel would be replaced with a UV3000Plus system, including four banks and matching the recently upgraded channel. Under Scenario 3, the higher dose can be provided with the same facilities as Scenario 1 because of the higher transmittance. In both cases one of the banks in each channel is a redundant bank. A redundant UV channel is not needed.

The total construction cost of the improvements for Scenarios 1 and 3, including a 20 percent contingency allowance, is estimated at $940,000. Allowing 25 percent for engineering, administration, and environmental, the total capital cost is estimated at $1.2 million.

Annual UV disinfection system O&M costs when the plant reaches design capacity are estimated at about $115,000 each for Scenarios 1 and 3. The corresponding cost is $79,000 per year for Scenario 2.

14.5 UV System Phasing Plan

Based on the discussions presented above, the following actions should be taken as soon as possible:

1. Provide features to block a portion of the influent weirs to the UV3000 system as needed to split flows to the UV channels in proportion to capacity. This will allow both channels to be used at the same time, which will maximize overall system capacity and performance. These features should be removable to allow an equal flow split to the two channels in the event that the UV3000 channel is upgraded to a UV3000Plus system in the future. It is presumed that the weir blocking modifications can be completed by District staff with engineering oversight. A budget allowance of $10,000 is suggested.

2. Confirm the extent to which the sludge storage lagoons can be used for flow diversions ahead of the UV disinfection system. This will depend on sludge storage volumes and plans for sludge removal. If capacity is available to allow peak flow trimming ahead of the UV disinfection system, revise the existing automatic diversion features that currently allow poor quality secondary effluent to be temporarily diverted to the sludge storage lagoons to also allow peak flow trimming to the sludge storage lagoons (i.e., diversion of a portion of the flow as opposed to all or none).

3. Conduct viral bioassay testing for the two existing UV disinfection channels to confirm performance and capacities. A budget allowance of $50,000 should be made for this testing, assuming both channels are tested at the same time.

4. Once peak flow capacities are verified consider the addition of a new flow equalization basin ahead of the UV disinfection system. However, inasmuch as the optimal design of this facility will be impacted by the decision on whether or not to add effluent filters, it may be beneficial to defer these improvements as long as adequate peak flow diversions can be made to the sludge lagoons. The cost of flow equalization facilities is considered in Section 13.

As developed in Section 8 and previously in this section, it is not currently known whether the recent improvements to the UV disinfection system, including provisions for diverting poor quality secondary effluent to the sludge storage lagoons, will prove to be practical and reliable for attaining compliance with the existing effluent limitations for total coliform organisms. If the turbidity setpoint for diverting secondary effluent to the sludge storage lagoons needed to assure reliable disinfection performance is found to be triggered too often or for durations that are too long, the available capacity of the sludge lagoons to accept such diversions could be exceeded. Also, since the diverted water eventually must be returned and retreated through the secondary treatment system at Plant 2, the volume of return flows could compromise the capacity and performance of the secondary treatment system. Accordingly, it is important to carefully monitor these operations to evaluate the overall acceptability of the current system.

If it is found that the existing UV system is able to provide reliable performance without effluent filtration and the effluent total coliform limit remains at 23 MPN/100 mL, the existing UV3000 channel should be upgraded to a UV3000Plus system before the peak hour flow through the UV system exceeds the UV disinfection system capacity that is determined after viral bioassay testing. The peak flow through the UV system can be controlled by peak flow trimming to the sludge storage lagoons or to the equalization basin, when constructed. However, peak flow trimming to less than the average flow on the peak day is probably not practical. Therefore, the average flow on the peak day should be taken as the minimum required design flow for the UV disinfection system. Since the current peak day average flow is 3.6 Mgal/d and the reliable UV disinfection system capacity may be only about 3.4 Mgal/d (from Table 14-1, with safety factor), the UV system upgrade may be required now. If a substantially higher capacity is determined from the viral bioassay testing and adequate peak flow trimming provisions exist, it may be possible to defer the UV system upgrade for a few years.

If it is found that effluent filtration is needed to assure reliable disinfection performance, design and construction of the effluent filters (and upstream flow equalization facilities, if not already constructed) should be initiated at that time. Once the effluent filters are constructed, no modifications to the UV system would be needed as long as the effluent coliform limit remains at 23 MPN/100 mL as a 7-day median.

If the permit requirements for total coliform become more stringent for river discharge or to allow reclamation, equalization facilities, filters and the UV system upgrade to UV3000Plus will all be required. Any of these features not already existing when the more stringent permit requirements are proposed will have to be constructed at that time. These facilities must be in operation before the more stringent permit requirements take effect.

Section 15

Salinity Reduction

15.1 Purpose

The California Regional Water Quality Control Board, Central Valley Region (Regional Board) has issued orders to the Town of Discovery Bay Community Services District (District) to reduce specific conductance of wastewater effluent disposed to Old River from the Discovery Bay Wastewater Treatment Plant (WWTP). The District has initiated separate salinity source control studies to identify mitigation strategies. Previous salinity management studies conducted by the District have identified reverse osmosis (RO) treatment of wastewater effluent as one of the potential options for reducing specific conductance or electrical conductivity of the wastewater effluent. The purpose of this Section is to analyze RO design and cost parameters and to assess the viability of a side-stream RO treatment system and associated RO concentrate management. Included in the remainder of this section are a general description of RO treatment and considerations of key design criteria, pretreatment requirements, facilities requirements, concentrate disposal, and estimated capital, operation, and maintenance costs.

15.2 Reverse Osmosis – General Description

Reverse osmosis, as illustrated in Figure 15-1, is the reversal of the natural osmotic process, accomplished by applying pressure in excess of the osmotic pressure to the more concentrated solution. This pressure forces the water through the membrane against the natural osmotic gradient, thereby increasingly concentrating the water on one side (i.e., the feed) of the membrane and increasing the volume of water with a lower concentration of dissolved solids on the opposite side (i.e., the filtrate or permeate). The required operating pressure varies depending on the total dissolved solids (TDS) in the feed water (i.e., osmotic potential), as well as on membrane properties and temperature.

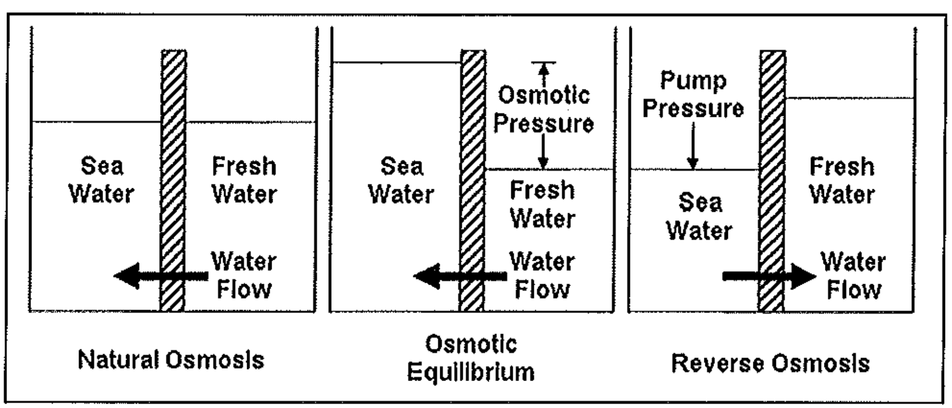


Figure 15-1
Illustration of Reverse Osmosis

15.3 Implementation of Reverse Osmosis as a Side-Stream Treatment Process

If RO treatment is implemented to reduce the electrical conductivity of the plant effluent, it is likely that a side-stream treatment system would be used to eliminate almost all salinity in the RO-treated portion of the flow, such that when this side-stream flow is re-combined with the remainder of the plant flow, the overall electrical conductivity objective would be met.

The existing WWTP consists of preliminary and secondary treatment units including screening, oxidation ditches and a UV disinfection system. The addition of tertiary filters is being considered (Section 13) to address possible future permit requirements. The influent flow for a side-stream RO treatment system would be obtained from a location downstream of tertiary filtration and upstream of the UV disinfection system. The side-stream flow would be held relatively constant so the RO treatment units would not have to be sized for peak flow conditions.

A membrane filtration (MF) process is proposed as an additional pretreatment step for RO treatment. There are several advantages to a MF pretreatment process, which are highlighted in the following sections. The sizing and design of the MF-RO system is dependent on the targeted reduction in specific conductance for the plant effluent, the plant influent flowrate, and average influent specific conductance, which may change before final design decisions are made. Table 15-1 is a summary of the design criteria assumptions for this analysis. The effluent electrical conductivity (EC) prior to RO treatment of 2200 µmho/cm is consistent with the existing effluent quality.

Table 15-1
Preliminary Design Criteria

Treatment	MF-RO
Main Flow, Mgal/d	2.37
Effluent Electrical Conductivity (EC) prior to RO, µmho/cm	2200
Estimated Effluent TDS prior to RO, mg/L	1375
Est. RO Recovery, %	80
Est. TDS removal, %	90
Targeted Final Blended Effluent EC, µmho/cm	1000
Side-Stream Flow to MF, Mgal/d	1.62
Side-Stream Flow to RO, Mgal/d	1.5
RO Reject Flow, Mgal/d	0.225
RO Permeate Flow, Mgal/d	1.275
Flow to VSEP (i.e., RO Reject Flow), gpm	156
VSEP Permeate Flow, gpm	125
VSEP Reject Flow, gpm	31
Volume of Brine (VSEP Reject) Requiring Disposal, gpd	45,120
Estimated Blended Effluent TDS, mg/L	616
Estimated Blended Effluent EC, µmho/cm	1000

A preliminary analysis of RO treatment requirements was conducted based on a single expanded set of water quality data provided by the District. Parameters that were not provided by the District and MF-RO treatment performance criteria were assumed for this analysis. The assumptions would need to be validated if further consideration is to be given to an RO treatment system after completion of this Master Plan. Based on preliminary analysis, an RO treatment system sized for a capacity of 1.5 Mgal/d should be sufficient to achieve the targeted effluent electrical conductivity of 1000 μmho/cm.

15.4 RO Pretreatment

Pretreatment is a vital step for a successful RO treatment application. RO membranes are not designed to remove suspended (particulate) solids; therefore, the main objective of RO pretreatment is to minimize the amount of suspended solids loading reaching the RO system. In addition to particulate matters, the ionic and organic constituents play a major role in determining the overall water recovery and the necessity for chemical pretreatment requirements, such as pH adjustment and/or scale prevention.

Fouling of RO Membranes usually occurs due to one or more of the following factors:

- Suspended solids (particulate matter) in the feedwater
- Scale formation of metals
- Precipitation of low solubility salts
- Adsorption of organic materials on the membrane surface and biofouling (organic growth)

15.4.1 Suspended Solids

The efficiency of pretreatment in removing particulate matter can be determined by measuring the silt density index (SDI). The RO membrane manufacturers normally specify a maximum allowable SDI for warranty requirements. In general, an SDI of less than 5 is required as a minimum warranty requirement. Membrane filtration (MF) is becoming the industry standard for removing suspended solids and improving SDI. The SDI of MF filtered water is generally much lower than 3.

15.4.2 Scale Formation

Due to the hardness of District water anti-scalant chemicals must be added continuously to the RO influent in order to control scale formation.

15.4.3 Precipitation of Low Solubility Salts

Typically, acid addition is required when the Langlier Saturation Index (LSI) is above 2.5. Acid is used to reduce the LSI to 2.5 at which point anti-scalant is very effective. The LSI of Discovery Bay WWTP influent is currently unknown.

15.4.4 Organic Fouling

Although RO membranes reject dissolved organics very effectively, organic-laden waters, such as wastewater have a tendency to foul the membranes. Often, the water recovery in wastewater applications is limited by the organic content in the feedwater rather than inorganic constituents. Therefore, secondary treatment followed by chloramination is recommended to reduce the organic loading and organic fouling potential.

15.5 Membrane Filtration

MF design criteria and key elements of the system are discussed briefly below. A schematic of an MF-RO system is shown in Figure 15-2.

Pressure membrane manufacturers identified in the preliminary analysis were contacted to determine design criteria for the membrane filtration system. A summary of the proposed design criteria is shown in Table 15-2.

Table 15-2
MF Design Criteria Summary

Design Criteria	Value
System Type	**Pressure**
Net Production Capacity	1.5 Mgal/d
System Redundancy	Minimum two trains with one standby train
Influent Total Suspended Solids (TSS)	10 mg/l
Influent Turbidity	2 NTU
Effluent Turbidity	<0.2 NTU
Effluent Total Suspend Solids	<1.0 mg/l
Design Temperature	15 °C

The MF system would include membrane trains and valve racks, chemical cleaning and neutralization systems, a chemical transfer system, compressed air and air-scour system, and an overall control system.

Vertical membrane modules with feed, filtrate and air manifolds at the top and bottom of the module is the most common configuration in pressure membrane systems. Valves, flow controllers and instrumentation would be located at the end of each train.

The membrane modules are backwashed to remove accumulated materials on the membrane surface. A backwash pump is used to pump filtered membrane effluent in the reverse direction of flow through the membranes. Air-scour, provided in the membrane modules, assists in re-suspending solids from the fiber surface to the bulk flow. Air compressors, a dedicated dryer and an air receiver tank located in the membrane building would provide a continuous supply of air to the air-scour system and pneumatic valves.

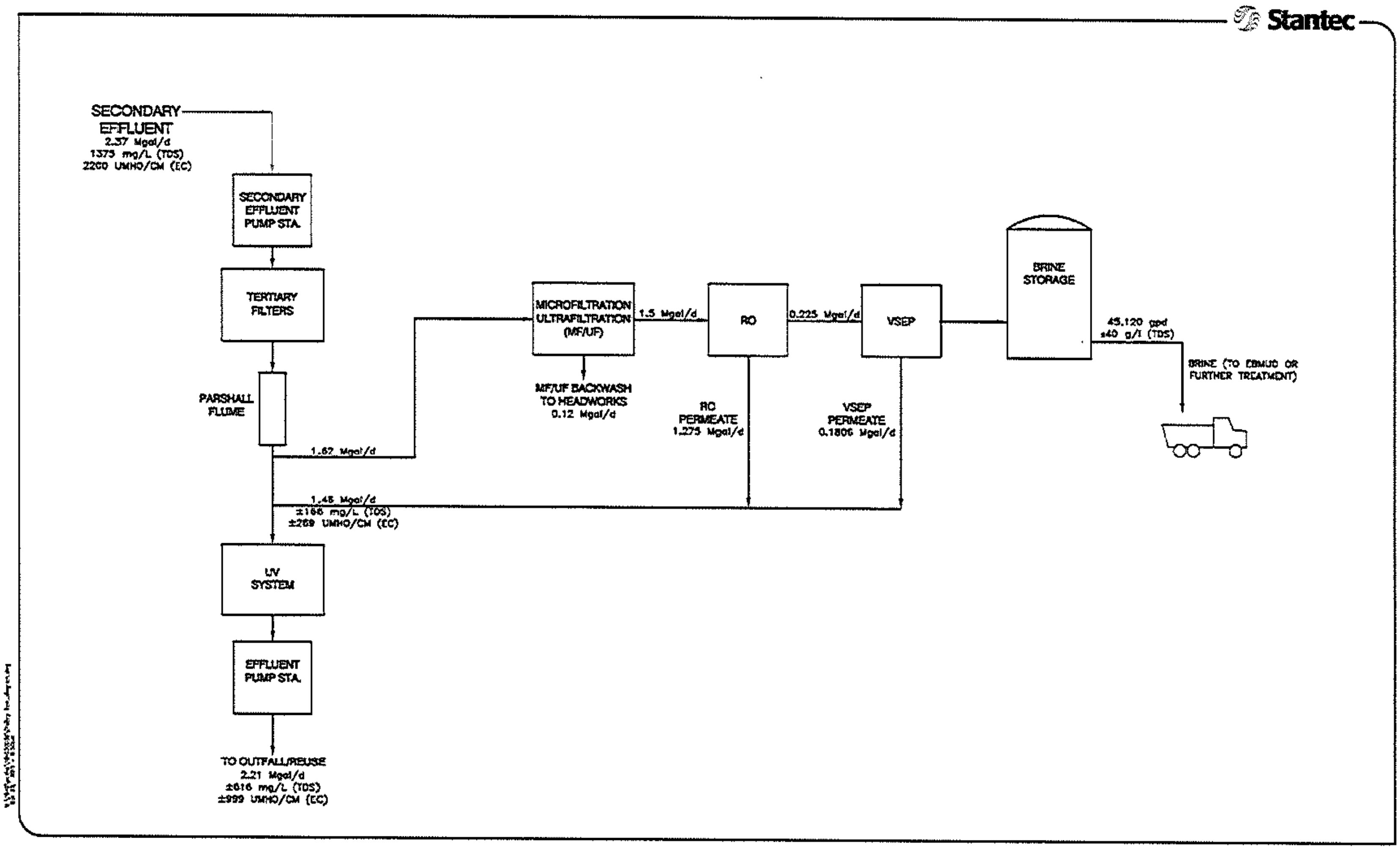


Figure 15-2
MF-RO Schematic

The primary cause for loss of membrane production capacity was found to be irreversible fouling caused by organic and inorganic substances. An intense and well-suited cleaning regime typically results in successful prevention of irreversible fouling. Sodium hypochlorite, caustic and citric acid are the frequently used membrane-cleaning agents. Citric acid is used to dissolve inorganic compounds and caustic is recommended for removing organic compounds. Sodium hypochlorite is a highly recommended cleaning agent to control biological fouling. The process of recirculating cleaning chemicals through the membrane system to restore the flux is referred to as a clean-in-place (CIP) procedure. When a membrane module requires chemical cleaning, chemicals are transferred from bulk storage to the heated CIP tank and mixed with potable water using a CIP pump. Heating the chemical solutions enhances the effectiveness of the cleaning procedure and also increases the rate of solubility of the chemical. Spent cleaning solution is routed to a neutralization tank capable of handling two volumes of CIP waste.

The capital cost and annual operation and maintenance (O&M) costs of a MF system are presented in Section 15.9.

15.6 Reverse Osmosis

A single pass RO system with a two bank configuration for higher water recovery (overall 80 percent) is proposed. The reject stream for Bank 1 becomes the feedwater for Bank 2 as shown in Figure 15-3. In contrast to micro- or ultrafiltration systems, there are no backwash mechanisms for RO systems, but RO systems do require chemical cleaning.

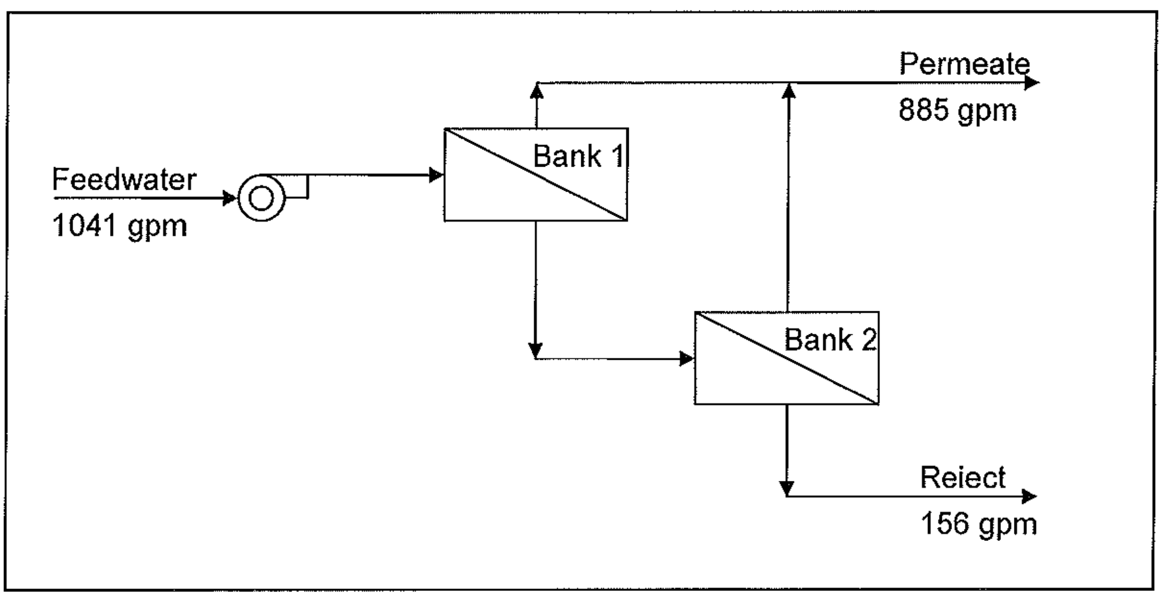


Figure 15-3
RO System Configuration

Spiral-wound modules were developed as an efficient configuration for the use of semipermeable membranes to remove dissolved solids and thus are most often associated with RO treatment. The basic unit of a spiral-wound module is a sandwich arrangement of flat membrane sheets called a "leaf" wound around a central perforated tube. One leaf consists of two membrane sheets placed back to back and separated by a fabric spacer called a permeate carrier. The layers of the leaf are glued along three edges, while the unglued edge is sealed around the perforated central tube. A layer of plastic mesh called a spacer that serves as the feed water channel separates each leaf. Feed water enters the spacer channels at the end of the spiral-wound element in a path parallel to the central tube. As the feed water flows across the membrane surface through the spacers, a portion permeates through either of the two surrounding membrane layers and into the permeate carrier, leaving behind any dissolved and particulate contaminants that are rejected by the semi-permeable membrane. The filtered water in the permeate carrier travels spirally inward around the element toward the central collector tube, while the water in the feed spacer that does not permeate through the membrane layer continues to flow across the membrane surface, becoming increasingly concentrated in rejected contaminants. This concentrate stream exits the membrane element parallel to the central tube through the opposite end from which the feed water entered. A diagram of a spiral-wound element is shown in Figure 15-4.

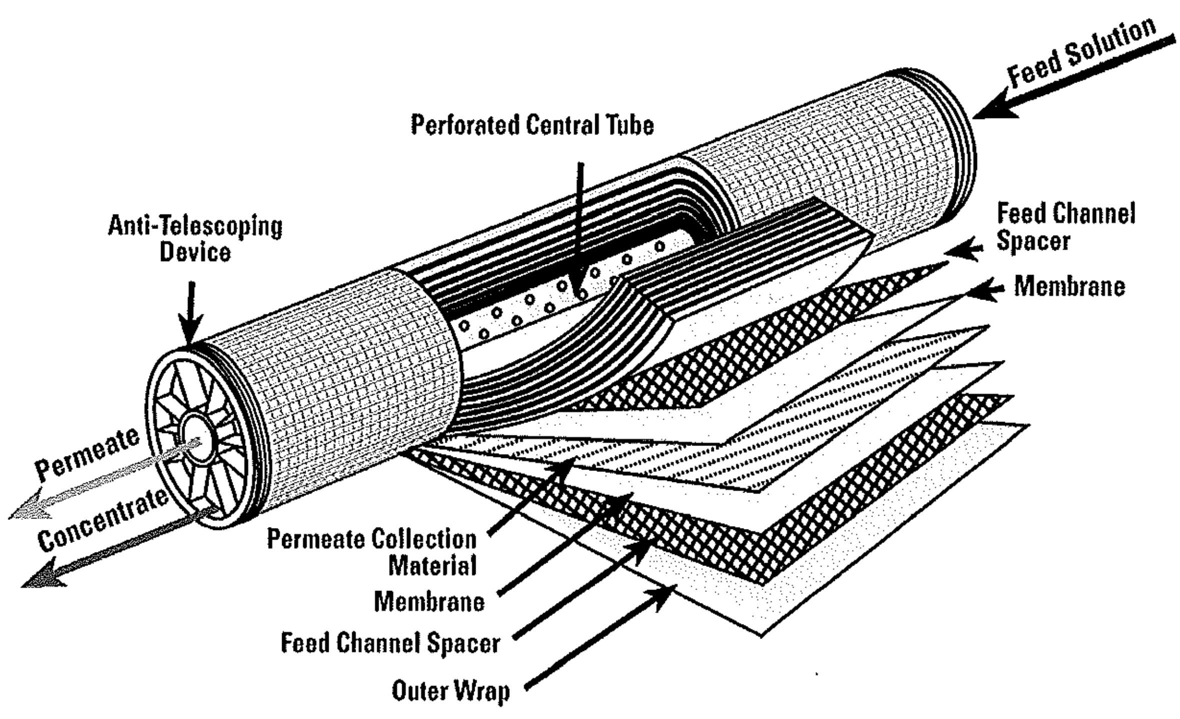


Figure 15-4
Spiral-Wound Membrane Element Diagram

The MF-RO facilities would be located within an enclosed building. The capital cost and annual O&M costs of the RO system are presented in Section 15.9.

15.7 RO Concentrate Management

Concentrate generated from RO treatment contains high amounts of TDS and organic compounds that are rejected by the RO membranes. Management of RO concentrate, which is typically 15-20% of the feed flow, poses the greatest challenge and costs for inland communities such as Discovery Bay.

15.7.1 Brine Concentration

A brine concentration step, which significantly reduces the RO concentrate volume, is typically utilized when ocean discharge or deep well injection disposal options are not available. Discovery Bay's location makes direct ocean discharge cost-prohibitive. Availability of an aquifer near to the WWTP that is suitable to take RO concentrate was uncertain at the time of this analysis. Therefore, the use of a Vibratory Shear Enhanced Processing (VSEP) brine concentrator is assumed for this analysis.

VSEP employs torsional vibration of the membrane surface, which creates high shear energy at the surface of the membrane. The result is that colloidal fouling and polarization of the membrane due to concentration of rejected materials are greatly reduced. Figure 15-5 illustrates the minimization of cake formation using VSEP.

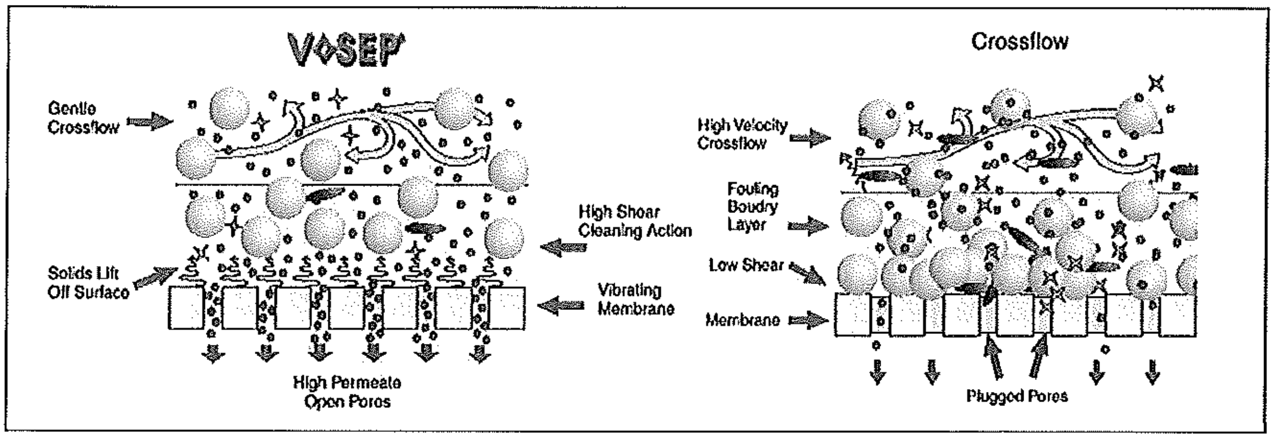


Figure 15-5
Vibratory Shear Enhanced Processing (VSEP)

A VSEP brine concentrator system would reduce the volume of brine by 90%. The VSEP membrane filter pack consists of leaf elements arrayed as parallel discs and separated by gaskets. The membrane disk stack is oscillated above a torsion spring that moves the stack back and forth approximately an inch at 50-60 Hz. The oscillation produces a shear at the membrane surface of about ten times the shear rate of the best conventional systems. The capital cost, and annual O&M costs of VSEP system are presented in Section 15.9.

15.7.2 Concentrate Management

Reject from the brine concentrator will have significant amounts of inorganic salts. Following are the commonly employed concentrate management options:

1. Open-topped lined evaporation ponds
2. Hauling or conveyance to facilities that have an ocean discharge

The option (#1) of storing and managing the reject in open-topped lined ponds has several potential issues, such as a) large land requirements; b) disturbance to the movement of migratory birds and potential bird deaths; c) generation of dust and air pollution during dry periods; d) habitat control; and e) fate of the evaporation pond after its useful life. East Bay Municipal Utility District (EBMUD), located 55 miles east of Discovery Bay is one the nearby facilities that have an ocean discharge. Hauling to EBMUD (Option #2) is one of the potential concentrate disposal options. However, hauling 45,000 gpd of concentrate would entail significant O&M costs. Although this is the basis of the annual costs indicated in Section 15.9, below, further volume reduction methods and other alternatives would have to be considered.

15.8 Overall Costs

Costs associated with an MF-RO system followed by a VSEP brine concentrator and hauling of brine to EBMUD, are summarized in Table 15-3.

Table 15-3
MF-RO-VSEP Cost Summary

Item	Cost, \$M [a]
Capital Costs [b]	
MF	4.0
RO	6.8
VSEP	4.9
Total	**15.7**
Annual Costs	
MF	0.1
RO	0.43
VSEP	0.25
Brine Hauling and Disposal	1.34
Total	**2.12**

(a) First quarter 2011 cost level. ENR 20-Cities CCI = 9000.
(b) Including construction of all required facilities, contingency allowance, engineering and administration.

15.9 Conclusions

The following conclusions are made based on the preliminary analysis presented above:

- MF-RO-VSEP treatment and hauling the brine to EBMUD is technically feasible, but cost-prohibitive.
- The overall energy consumption of the Discovery Bay WWTP would increase several fold from present values if an MF-RO-VSEP system were implemented.
- The consumption of chemicals, energy, replacement membranes, cleaning agents and hauling fuel would cause this system to have an enormous carbon footprint. The net impact on the environment would probably be considered detrimental, even though a higher quality plant effluent would be produced.

Before consideration of implementing an MF-RO-VSEP system, all reasonable efforts to control the salinity of the wastewater influent through source control and/or use of alternative water supplies should be investigated.