WASTEWATER TREATMENT AND DISPOSAL FACILITIES CAPACITY STUDY

PROJECT REPORT

FINAL

December 2006

Prepared by:

BROWN AND CALDWELL

9665 Chesapeake Drive, Suite 201
San Diego, California 92123

CONTENTS

APPENDICES

TABLES

FIGURES

ABBREVIATIONS

AADF - average annual daily flow
ADWF - Average Dry Weather Flow
afy - acre-feet per year

BAF – Biological Aerated Filter
BOD_5 – 5-day Biochemical Oxygen Demand
BTU - British Thermal Unit

CAS – Conventional Activated Sludge
$CBOD_5$ – 5-day Carbonaceous Biochemical Oxygen Demand
CEPT – Chemically Enhanced Primary Treatment
cfm - cubic feet per minute
CFR – Code of Federal Register
City - City of Escondido
COD - chemical oxygen demand

DAFT - dissolved air flotation thickener
DWF - dry weather flow
DO - dissolved oxygen

ELO - Escondido Land Outfall
EPA – Environmental Protection Agency

fps - feet per second
ft - foot
ft^2 - square foot

g/L - grams per liter
gpcd - gallons per capita per day
gpd/EDU - gallons per day per equivalent dwelling unit
gpd/ft^2 - gallons per day per square foot
gpm - gallons per minute
GWI - groundwater infiltration

H_2S – Hydrogen sulfide
HARRF - Hale Avenue Resource Recovery Facility
HSA - Hydrologic Subarea
hp - horsepower

IBCS - Industrial Brine Collection System
IPS - Influent Pump Station
lb/d - pound per day
lb/hr - pound per hour
lb/sf-d – pound per square foot per day

ABBREVIATIONS (continued)

MBR – Membrane bioreactor
MER - mass emission rate
mg/L - milligram per liter
MG - million gallons
mgd - million gallons per day
min - minute
ml/g - milliliters per gram
MLSS – Mixed liquor suspended solids
mm - millimeters
mW - milliwatt
MWH - Montgomery Watson Harza
mW-s/cm^2 - milliwatts seconds per square centimeter

N - nitrogen
nm - nanometer
NPDES - National Pollutant Discharge Elimination System
NTU - nephlometric turbidity units

Ocean Plan - California Ocean Plan
OCS - odor control system

P - Phosphorus
PACl - polyaluminum chloride
PEP - Palomar Energy Project
psig - pounds per square inch gauge
PSRP - Process to Significantly Reduce Pathogens
PWWF - Peak Wet Weather Flow

RAS - return activated sludge
RB - Rancho Bernardo
RCP - reinforced concrete pipe
RDI/I - rainfall dependent inflow and infiltration
Rincon - Rincon Del Diablo Municipal Water District
RO - reverse osmosis
RPM - revolutions per minute

SANDAG - San Diego Association of Governments
scfm - standard cubic feet per minute
sec/cm^2 - seconds per square centimeter
sec^{-1} - second (time)
SEJPA - San Elijo Joint Powers Authority
SEOO - San Elijo Ocean Outfall
SEWRF - San Elijo Water Reclamation Facility
SOR - surface overflow rates

ABBREVIATIONS (continued)

SPA - state point analysis
SRT – solids retention time
SVI - sludge volume index

TSS - Total Suspended Solids
TWAS - thickened waste activated sludge

UV - ultraviolet

VFD - variable frequency drive
VSR - minimum percentage versus reduction

WAS - waste activated sludge
Water Board - San Diego Regional Water Quality Control Board

EXECUTIVE SUMMARY

Introduction and Purpose

The Hale Avenue Resource Recovery Facility (HARRF), a wastewater treatment facility owned and operated by the City of Escondido (City), is approaching its design capacity. To ensure that wastewater received at the HARRF continues to receive adequate treatment prior to disposal or reuse, the City conducted an evaluation of the capacity of the existing treatment facilities at the HARRF, the Escondido Land Outfall, and the San Elijo Ocean Outfall. This capacity study serves three purposes:

1. To determine the capacity of the HARRF, Escondido Land Outfall, and the San Elijo Ocean Outfall using the latest proven prediction technologies available to produce an accurate assessment;

2. To determine order-of-magnitude costs for improvements that will allow the City to adequately treat incoming wastewater up to the current rated average annual daily flow capacity of 18.0 million gallons per day (mgd) and up to the projected build-out average annual daily flow capacity of 27.5 mgd (including an average annual daily flow of 5.3 mgd from the City of San Diego); and

3. To determine order-of-magnitude costs for improvements that will provide sufficient storage and disposal capacities during storm conditions.

Previous capacity-rating studies that set the current rating for the HARRF relied on design criteria reported in various industry-accepted and USEPA publications. For this study, Brown and Caldwell used the latest plant-performance data and conducted the following activities:

- Monitored influent wastewater characteristics for two weeks;

- Evaluated activated sludge settling characteristics;

- Monitored and evaluated 30 days of wet-weather flow to calibrate process, storm flow, and hydraulic simulation models; and

- Determined treatment and hydraulic capacities based on calibrated models.

Individual process components of the HARRF, Escondido Land Outfall, and the San Elijo Ocean Outfall were analyzed in detail. Disposal options and preliminary, order-of-magnitude-level cost estimates were prepared. Technical memoranda, summarized in the report and included as appendices to this report, provide extensive details on methodologies, results, findings, conclusions, and recommendations.

Background

The HARRF was originally constructed in 1959 as a 1.0-mgd activated sludge facility. It has undergone five phases of capacity expansion and is currently rated to provide, on average, 18.0 mgd of secondary wastewater treatment and 9.0 mgd of tertiary wastewater treatment. In 2005, the HARRF treated an average annual daily flow of 15.3 mgd. During the 2005 winter storm of February, there were 10 days in which the plant influent flow averaged 21.0 mgd, and the HARRF was able to meet all current secondary discharge requirements. Average annual daily flows are projected to reach 27.5 mgd at build-out, including 5.3 mgd from the City of San Diego.

The majority of treated wastewater from the HARRF is disposed of through land and ocean outfalls. The Escondido Land Outfall runs 14.3 miles from the HARRF fence line to the San Elijo Ocean Outfall. The land outfall flows partially full for the first nine miles and completely full the last 5.3 miles. The San Elijo Ocean Outfall extends 8,000 feet offshore.

Agency permits are in place to regulate discharge to the ocean outfall, intermittent live-stream discharge to Escondido Creek, use recycled water produced at the HARRF, and discharge brine received from industries. Current permitted limits, on an average basis, are 18.0 mgd for the Escondido Land Outfall and the San Elijo Ocean Outfall discharge, 9.0 mgd for intermittent wet-weather live-stream discharge, 9.0 mgd for recycled water production, and 1.0 mgd of the total 18.0 mgd discharged through the outfall for brine discharge.

There are several terms used in this project report that relate to flow. The term average annual daily flow describes the average daily flow over one calendar year and includes wet and dry weather flows. It is typically used when discussing annual flow projections and annual discharge mass emission rates. Average dry weather flow refers to a period when wastewater flows are not expected to be impacted by wet weather periods or high groundwater levels. Average dry weather flow is used in this study to determine the capacity of a given unit process with one tank or clarifier out of service. This is to allow for preventive maintenance activities during periods of anticipated low plant influent flows. For the HARRF, the average daily wastewater flows recorded between August 1 and October 31 from 2000 to 2005 were considered as average dry weather flows. Note that because Escondido is located in an arid environment with limited rainfall, there is less than a 5 percent difference between average annual flow and average dry weather flow. Peak wet weather flow represents the maximum average one-hour anticipated flow during a design rainfall event. This value is used to design new and rate existing hydraulic capacity of treatment process and conveyance facilities.

Existing Treatment and Disposal Capacity

Table ES-1 presents the process and conveyance capacities of the HARRF and outfall facilities, and notes certain factors that limit capacity of each major process unit and/or piece of equipment. The capacity of each process unit is a function of a combination of flow rate, solids loading, organic loading and nutrient loading. As shown in the table, the secondary and solids-treatment processes were found to be the capacity-limiting processes at the HARRF.

The secondary processes of the HARRF include the aeration basins, the air blowers, the air diffusers, the secondary clarifiers, and the return activated sludge and waste activated sludge pumping stations. The process capacity of this system was determined using a calibrated activated sludge model and secondary clarifier performance analysis. Other criteria used in estimating the capacity were as follows:

- The treated effluent must meet limits for discharge to the ocean outfall
- The treated effluent must meet secondary effluent limits for direct filtration to produce recycled water that meets Title 22 standards
- An adequate dissolved oxygen concentration in the aeration basins must be maintained
- An acceptable solids loading rate to the secondary clarifiers must be maintained to limit upsets

The parameters used to determine the secondary process capacity were based on historical performance data between 2000 and 2005. The average solids residence time of 2.75 days determined from historical data was used to simulate activated sludge performance. However, it should be noted that in 2005, the average solids residence time was 3.5 days. The mixed liquor settleability (as measured by the sludge volume index) was used to evaluate secondary clarifier performance. A lower sludge volume index corresponds to a better settling sludge. Historical sludge volume index values ranged between 65 and 600 milliliters per gram (mL/g). The 90th percentile from this historical record was used as a measure of the reliable sludge settleability that can be anticipated under current activated sludge operating conditions. This value is calculated to be 203 mL/g. Use of the 90th percentile value means that current operating conditions will produce a sludge volume index of 203 mL/g or less approximately 90 percent of the time, or approximately 330 days per year.

While solids retention time can be controlled directly, a specific sludge volume index value cannot. It is affected by the solids retention time and other factors. Reconfiguring the aeration basins to incorporate a biological selector and operating at a lower solids retention time are process modifications that can reduce the sludge volume index, reduce the mixed liquor suspended solids concentration, and ultimately increase secondary process capacity. By implementing an anaerobic selector, a 90th percentile sludge volume index of 125 mL/g or less would be anticipated, based on performance of similar systems in other treatment plants. This is based on Brown and Caldwell's survey of full-scale treatment facilities with biological selectors.

In the absence of a biological selector, sludge settleability can be controlled with chemicals (e.g., return activated sludge chlorination or polymer addition). However, chemical control requires additional operator attention involving frequent microscope analyses of mixed liquor and sludge volume index measurements to prevent overdosing of chemicals. This is especially critical for return activated sludge chlorination, often used to lower the sludge volume index, where overdosing can deleteriously affect biological treatment and reduce effluent quality. The City uses these standard operating procedures for control of sludge settleability. These standard operating procedures, if implemented appropriately, can result in a 90th percentile sludge volume index of 150 mL/g, which would increase plant capacity beyond the value estimated under this study. Without these standard operating procedures, the estimated average dry weather flow capacity is 14.8 mgd (the corresponding peak wet weather flow capacity is 29.3 mgd).

In addition to controlling sludge settleability, sufficient aeration capacity is necessary for successful activated sludge operation. Sufficient aeration is needed to maintain a mixed liquor dissolved oxygen concentration of at least 2 milligram per liter (mg/L), especially at the head of the aeration tank where the mixed liquor passes from the unaerated zone to the aerated zone.

The existing aeration system (blowers and fine-bubble diffuser panels) can provide sufficient oxygen transfer to treat only 15.0 mgd average daily flow. The existing fine-bubble diffuser panels limit the amount of air that can be added to the wastewater. It is recommended that the existing aeration panels be replaced with circular flexible membrane fine-bubble diffusers that allow more air to be added to the wastewater. By replacing the fine-bubble diffusers and modifying the plant operating conditions, the capacity of the secondary system can be increased to 18.4 mgd average daily flow.

Table ES-1. Process and Conveyance Capacities of the HARRF and Outfall Facilities

Process / Outfall	Calculated Capacity (mgd)	Capacity-Limiting Factor / Remarks
Influent Pump Station	21.8 Daily Average 43.5 Peak	Calculations based on Reference 1 (below). Requires re-evaluation due to recent changes to pump impeller and operation. Comprehensive evaluation of the current physical condition and remaining useful life of the equipment (including gates, operators, valves, etc.) must be conducted to determine the need for upgrades in system components or replacement of equipment.
Preliminary Treatment		
▪ Bar Screen	22.0 to 28.0 Peak per Screen	Rating based on Reference 2. Approach velocity of 2.0-3.0 feet per second at rated capacity is within standards.
▪ Vortex Grit Chamber	14.5 to 21.0 Daily Average per Chamber	Rating based on Reference 1. Higher flows can be accommodated, but grit removal efficiencies are lower, which increases the primary sludge grit content.
Primary Treatment [(a)]	18.0 Daily Average	Existing primary clarifiers have sufficient hydraulic and process capacity for the existing plant rating of 18.0 mgd. (Reference 3)

Table ES-1. Process and Conveyance Capacities of the HARRF and Outfall Facilities

Process / Outfall	Calculated Capacity (mgd)	Capacity-Limiting Factor / Remarks
Secondary Treatment [(a)]		
▪ Aeration System (blowers, fine bubble diffusers)	15.0 Daily Average	Existing blower capacity at 2.75-day solids retention time assuming the existing fine-bubble diffusers are replaced. (Reference 4)
▪ Aeration System/Aeration Basins/Secondary Clarifiers	14.8 Daily Average 29.3 Peak	Capacity assuming 2.75-day solids retention time and 90th percentile sludge volume index of 203 mL/g.
Tertiary Treatment		
▪ Filtration	8.0 Peak (excluding reject)	Existing Dynasand filters are designed to treat a maximum of 10 mgd (9 mgd of secondary effluent + about 1 mgd of reject flow) with one unit out of service at 5 gpm/ft². The 8.0 mgd capacity is based on maximum historical operating conditions.
▪ Disinfection		
✓ UV	4.0 Peak	Based on Reference 5.
✓ Chlorination	10.0 Peak	Based on Reference 6.
Solids Processing [(b)]		
▪ Thickening (Dissolved Air Flotation)		Based on solids loading design criteria of 14 pounds per square foot per day (lb/sf-d). (Reference 7)
✓ One Unit out of Service	14.0 Daily Average	
✓ All Units in Service	28.0 Daily Average	
✓ Saturation System	14.0 Daily Average	
▪ Digestion		Although EPA 40 CFR 503 Part B regulations require a minimum of 15-day solids retention time, the criterion used for this study is a minimum 20-day solids retention time under average loading conditions with the largest unit out of service and under peak 2-week loading condition with all units in service. This criterion provides a safety factor to account for unusual events, peak conditions when a digester is out for service, and potential grit accumulation that reduces effective volume. (Reference 7).
✓ One Unit out of Service	12.0 Daily Average	
✓ All Units in Service	18.0 Daily Average	
✓ Vector Attraction Reduction	Meets Requirements	
▪ Dewatering	24.0 Daily Average	Based on hydraulic loading of 150 gallons per minute per centrifuge and two of the three centrifuges in service. It also assumes 7-day, 12 hours per day operation. (Reference 7)

Table ES-1. Process and Conveyance Capacities of the HARRF and Outfall Facilities

Process / Outfall	Calculated Capacity (mgd)	Capacity-Limiting Factor / Remarks
Escondido Land Outfall		
▪ Gravity Section	23.7 Peak	Capacity at which the water surface in this section of the pipe exceeds the manhole rim elevation, causing a spill (Reference 8)
▪ Pressurized Section	21.4 Peak	Capacity at which the water surface in this section of the pipe reaches the rim elevation of Manhole No. 74, causing a spill. (Reference 8)
San Elijo Ocean Outfall	Total Outfall Capacity: 25.8 mgd Peak City of Escondido allocated capacity: 20.4 mgd Peak	Limited by the pressure rating of the pipe under the shoreline. (Reference 8)

References:

1. *Final Letter Report for Capacity Rerating of the Hale Avenue Resource Recovery Facility (HARRF)*, May 11, 2004, Montgomery Watson Harza.
2. June 5, 2006 e-mail from Joe Nagel of Parkson.
3. *Plant Hydraulic Profile Analysis Technical Memorandum,* July 5, 2006, Brown and Caldwell – Appendix B of this Project Report.
4. *Biological Process Evaluation Technical Memorandum*, August 20, 2006, Brown and Caldwell – Appendix D of this Project Report.
5. April 21, 2004 letter from the San Diego Regional Water Quality Control Board entitled "*Authorization to Discharge Title 22 Reylced Water, Order No. 93-70*".
6. Discussion with HARRF staff and Appendix A of this Project Report.
7. *Solids System Evaluation Technical Memorandum*, October 20, 2006, Brown and Caldwell – Appendix G of this Project Report.
8. *Land and Ocean Outfall Capacity Analysis Technical Memorandum*, August 10, 2006, Brown and Caldwell – Appendix I of this Project Report.

Notes:

(a) Capacity values reported are Average Dry Weather Flow
(b) Equivalent Plant Influent Average Dry Weather Flow Capacity noted
Peak = Peak Wet Weather Flow

Recommended Near-Term Improvements at the HARRF

Implementation of permanent improvements to ensure that the HARRF maintains adequate capacity to treat incoming flows can take several years to plan, design, and construct. Meanwhile, interim, near-term improvements to the secondary and solids treatment system can be implemented to make certain that the plant can treat an average daily flow of 18.0 mgd. These improvements include reducing the solids inventory in the aeration basins, improving sludge settleability, increasing oxygen transfer capacity, increasing the efficiency and hydraulic loading to the dissolved air flotation thickeners, and increasing the hydraulic retention time in the digesters. Recommended near-term improvements are further discussed below.

Secondary Treatment Improvements

- Implement chemically-enhanced primary treatment, where coagulants and flocculants are added upstream of the primary clarifiers to increase primary solids removal. This modification will reduce the organic loading to the secondary process and lower the mixed liquor suspended solids concentration. This will allow the HARRF staff to take one primary clarifier out of service for preventive maintenance.

- Optimize the return activated sludge chlorination and polymer addition to the aeration basins to lower the sludge volume index and improve the settleability of the secondary solids.

- Increase aeration capacity by supplementing the existing diffuser capacity. This can be done by temporarily adding blowers and diffusers to the basins (since the existing diffuser panels currently limit the amount of air added to the basin), adding high purity oxygen to the incoming return activated sludge stream, or adding surface aerators. The supplemental aeration would be used to increase dissolved oxygen concentrations in the aeration basins only when needed (such as during peak flow or load periods).

Solids Processing Improvements

Dissolved Air Flotation

- Optimize polymer dosage to allow the dissolved air flotation thickeners to operate at higher solids loading rates and improve solids capture efficiency.

- Move the polymer injection point to turbulent areas to optimize mixing and contact of the polymer with the solids.

- Replace the thickener overflow weir with submerged launder pipe to provide cleaner water for recycle to the pressurized flow system.

- Provide control valve on the thickener effluent line to control the liquid level, maximize the drainage of water from the float, and increase the solids content of the thickened sludge.

- Replace pressurized flow pumps to meet necessary recycle flow for solids loading to provide sufficient flow for air saturation.

- Add second pressurization tank or increase operating level to provide sufficient residence time for air to dissolve and to reduce possibility of vortexing.

- Add continuous vent to purge excess nitrogen from the pressurization tank, increase gas absorption, and improve stability.

- Modify inlet and outlet piping to prevent vortexing and inlet pipe flooding.

- Direct a portion of the waste activated sludge to co-thicken with the primary sludge in the primary clarifiers. This is not recommended for day-to-day operation, but may be considered in an emergency if both dissolved air flotation thickeners are out of service.

Anaerobic Digesters

- Feed primary and secondary solids simultaneously to all digesters (on the same day) to ensure consistent solids feed to the digester, stabilize operation, and prevent gas production spikes.
- Verify lances and draft tubes are free of obstructions or buildup to ensure system is operating as designed.
- Verify that draft tube mixing capacity provides 16 to 24 turnovers per day which is necessary to prevent solids deposition, surface matting, dead zones, and hot spots.
- Provide dedicated compressors for Digesters Nos. 1 and 2. This is needed to provide a balanced operation to draft tube gas mixing systems.
- Perform a dye study to confirm mixing efficiency in the digesters, particularly for Digester No. 1.
- Consider recuperative thickening when taking a digester out of service in order to maintain the solids retention time required to produce Class B biosolids. Recuperative thickening is a process where a portion of the partially digested sludge is removed from the digester, thickened, and re-inserted into the digester to increase the solids retention time of the sludge. This process is typically used to meet Class A or B solids retention time requirements. Since taking a digester out of service is an infrequent activity lasting about 3 to 4 weeks at a time, centrifuges or gravity belt thickeners can be rented rather than construction of another digester. (It should be noted the solids retention time required under Class B regulations can be met at 18.0 mgd average daily plant influent flow when all existing digesters are in service).

Centrifuge Dewatering

- Provide sludge samples to centrifuge and polymer suppliers to verify that the sludge character has not changed since centrifuges were placed into service.
- Perform polymer trials to make certain that the correct polymer is being used.
- Perform periodic acid cleaning of centrate pipes and/or use polyphosphate scale inhibitors to maintain the centrate system hydraulic capacity and prevent backups from occurring.

Recommended Long-Term Improvements at the HARRF

Eight secondary treatment alternatives to increase the HARRF treatment capacity to the ultimate build-out average daily flow of 27.5 mgd were developed (see Appendix I-*System Integration and Optimization Technical Memorandum* for more details on each alternative). All secondary treatment alternatives will produce a secondary effluent that meets ocean discharge requirements. Of the eight secondary treatment alternatives, the following two were considered viable for HARRF:

- Secondary Treatment Alternative 3 - High-Rate Activated Sludge (2.0-day solids retention time) with intermittent chemically enhanced primary clarification

- Secondary Treatment Alternative 6 - Moving bed bioreactor

In addition, three tertiary treatment alternatives that enable the reliable production of 9.0 mgd of recycled water were developed. Most of the tertiary treatment alternatives are compatible with either of the two recommended secondary treatment alternatives above. These tertiary treatment alternatives include replacing or reusing the existing sand-media Dynasand filters and installing additional processes to reduce the filter influent turbidity. Tertiary treatment alternatives that use membranes instead of a sand-media filter can meet Title 22 requirements for recycled water regardless of secondary effluent quality. The tertiary treatment alternatives evaluated include the following:

i. Nitrifying biological aerated filter following secondary treatment
ii. Membrane filtration to replace the existing tertiary filters
iii. Side stream membrane bioreactor

A summary of the recommended secondary treatment process alternatives combined with the feasible tertiary treatment alternatives are presented in Table ES-2.

Table ES-2. Summary of Viable Process Alternatives for HARRF

Secondary/ Tertiary Treatment Alternative	Primary Treatment	Secondary Treatment	Additional or Tertiary Treatment
3A	Existing primaries; intermittent chemical addition	Anaerobic Selector and High Rate Activated Sludge	Biological Aerated Filter Dynasand Filter Chlorine Disinfection
3B	Existing primaries; intermittent chemical addition	Anaerobic Selector and High Rate Activated Sludge	Membrane Filtration[a] Chlorine Disinfection
3C	Existing primaries; intermittent chemical addition	Anaerobic Selector and High Rate Activated Sludge with Separate 9.0 mgd Capacity Membrane Bioreactor	Membrane Bioreactor[a] Chlorine Disinfection
6B	Existing primaries; no chemical addition	Moving Bed Bioreactor	Membrane Filtration[a] Chlorine Disinfection
6C	Existing primaries; no chemical addition	Moving Bed Bioreactor with Separate 9.0 mgd Capacity Membrane Bioreactor	Membrane Bioreactor[a] Chlorine Disinfection

a – Existing Dynasand filters abandoned

If a nitrifying biological aerated filter, membrane filter, membrane bioreactor, or moving bed bioreactor is selected, a pilot test must be performed to verify full-scale design criteria and/or process performance.

A detailed list of improvements at the HARRF included with each alternative listed in Table ES-2 is presented in Table ES-3. The improvements will enable the HARRF to treat up to 27.5 mgd of average dry weather flow. However, not all improvements must be constructed concurrently. Potential phasing of the construction to save cost, but ensure that sufficient treatment capacity is available at the HARRF; is discussed below.

Table ES-3. Summary of Recommended Improvements at the HARRF for 27.5 mgd Average Dry Weather Flow Treatment Capacity

Description	Units	Needed Improvements for Alternative 6 [(a)]	Needed Improvements for Alternative 3 [(a)]
Bar Screen			
Number to Install	---	1 (convert existing manual bar screen)	1 (convert existing manual bar screen)
Type	---	Mechanical	Mechanical
Bar Spacing	mm	6	6
Peak Capacity	mgd	22	22
Influent Pump Station			
Pumps	---	▪ Operate the 9000-gpm pumps at 10-12 percent higher than the design speed to provide additional capacity. ▪ Determine if the pump foundation, frame, motor supports and rotating system can withstand the dynamic forces resulting from operation at the higher speeds.	
Motor	---	▪ Determine if the motor design is adequate to handle the additional electrical current and voltage at the higher speed	
Variable frequency drives	---	▪ Conduct a field torsiograph test to identify torsional resonance issues and determine if the existing variable frequency drive can operate at speeds greater than 60 Hz.	
Discharge Force Main	---	▪ Upgrade of discharge force main to 36 inch pipe	
Grit Removal [(b)]			
Number to Install	---	1	1
Type	---	Vortex	Vortex
Diameter	ft	24	24
Average capacity	mgd	21	21
Primary Clarifiers	*Convert all primary clarifiers to chemically enhanced primary clarifier* [(c)]		
Primary Clarifier Basins	number	1	1
Side water depth	ft	10	10
Surface area per tank	ft^2	5,250	5,250
Primary Sludge Pump Station			
Diaphragm Pumps	number	1	1
Pump capacity, each	gpm	150	150
Design head	ft	80	80
Activated Sludge System			
Aeration Basins	number	None	1
Side water depth	ft		16.5
Surface area	ft^2		75,000
Blowers	number		1
Pump capacity, each	scfm		10,300

Table ES-3. Summary of Recommended Improvements at the HARRF for 27.5 mgd Average Dry Weather Flow Treatment Capacity

Description	Units	Needed Improvements for Alternative 6 [(a)]		Needed Improvements for Alternative 3 [(a)]		
Aeration Basin Modifications		*Convert 25 percent of the existing aeration basin to moving bed bioreactor. One aeration basin will be dedicated to membrane bioreactor in Alt 6C.*		*Convert 20 percent of the existing aeration basins into biological selector zone. One aeration basin will be dedicated to membrane bioreactor in Alt 3C.*		
Submersible Mixers	number	None		7 (6 duty + 1 standby)		
Advanced Treatment System (to produce recycled water)						
Type of system		Alt 6A Biological aerated filter	Alt 6B Micro filtration	Alt 3A Biological aerated filter	Alt 3B Micro filtration	Alt 3C Moving bed bioreactor
Average capacity	mgd	9.0	9.0	9.0	9.0	9.0
Return Activated Sludge Pump Station				Alt 3A and B		Alt 3C
Return Activated Sludge Pumps	number	3		3		4
Pump capacity, each	gpm	4,107		4,107		10,000
Mixed Liquor Pump Station						
Waste Activated Sludge Pumps	number	2		2		
Pump capacity, each	gpm	2430		2430		
Sludge Thickening						
Co-Thickening				Alt 3A and 3B		Alt 3C
Dissolved Air Flotation Thickener Basin	number	2		2		2
Diameter	ft	37		37		37
Thickened Sludge pumps	number	2		2		2
Pump capacity, each	gpm	260		260		260
Pressurization System[(d)]	number	4		4		4
Pump capacity [(e)], each	gpm	1,000		1,000		1,000
Pump pressure	ft	175		175		175
Compressor [(f)]	number	1		1		1
Compressor capacity, each	scfm	15		15		15
Separate Thickening				Alt 3A and 3B		Alt 3C
Dissolve Air Flotation Thickener Basin	number	1		1		2
Diameter Dissolve Air Flotation	ft	36		36		36
Thickened Sludge pumps	number	2		2		2
Pump capacity, each	gpm	260		260		260
Pressurization System	number	1		1		1
Pump capacity[(f)], each	gpm	500		500		500
Pump pressure	ft	175		175		175
Compressor	number	1		1		1
Compressor capacity, each	scfm	15		15		15
Sludge Degritting and Dewatering System[(b)]						
Slurrycup Grit Washing Units	number	2		2		
Diameter	ft	56		56		
Capacity, each	gpm	650 to 950		650 to 950		

Table ES-3. Summary of Recommended Improvements at the HARRF for 27.5 mgd Average Dry Weather Flow Treatment Capacity

Description	Units	Needed Improvements for Alternative 6 (a)	Needed Improvements for Alternative 3 (a)	
Grit Snail	number	1	1	
Capacity	cu yd /hr	4	4	
Sludge Digestion				
Co-Thickening				
Anaerobic Digesters	number	1	1	
Tank Diameter	ft	109	109	
Side water depth	ft	25	25	
Separate Sludge Thickening			Alt 3A and 3B	Alt 3C
Anaerobic Digesters	number	1	1	1
Tank Diameter	ft	141	141	142
Side water depth	ft	25	25	25
Dewatering System				
Co-Thickening				
Centrifuge number	number	1	1	
Average capacity each,	gpm	150	150	
Operating Centrifuge number	hrs/day	12	12	
Separate Sludge Thickening				
Centrifuge number	number	2	2	
Average capacity each,	gpm	150	150	
Operating schedule	hrs per day	12	12	

Notes:

(a) Needed improvements to the existing HARRF to enable treatment of 27.5 mgd average daily flow expected at build-out. Assumes that near-term improvements have not been implemented. Alternative 6A, 6B, 3A, 3B and 3C relate to improvements needed to produce secondary effluent with water quality characteristics appropriate for ocean discharge, plus the addition of an advanced treatment system that could produce up to 9.0 mgd of recycled water. Advanced treatment options for each alternative are as follows: Alternative A = nitrifying biological aerated filters; Alternative B = membrane filtration; Alternative C = membrane bioreactors.

(b)Grit removal system at the headworks is required for separate sludge thickening option only. Sludge degritter and dewatering system is required for co-thickening option only.

(c) Chemically enhanced primary treatment will be used for near-term solution for the HARRF regardless which alternative is selected. For build-out condition, chemically enhanced primary treatment will only be used for Alternative 3 on a routine basis. For Alternative 6, chemically enhanced primary treatment is necessary during construction only, and will not be necessary upon completion.

(d) Each pressurized injection system consists of one tank and one pump. Compressors are operated on a common manifold that services all dissolved air flotation thickeners. Assumes that two (one duty and one standby) pressurized injection systems are added to existing dissolved air flotation thickeners. 100 percent redundancy is provided.

(e) Assumes that the compressed air system operates on a common discharge manifold to allow service to all dissolved air flotation thickeners. Additional compressors noted are required due to the new dissolved air flotation thickeners proposed.

(f) Assumes existing system is upgraded to match the capacity of the new proposed systems to provide uniformed sizing for redundancy.

Construction Phasing of the HARRF Improvements

The projected average annual daily flow up to the year 2050 is shown on Figure ES-1. As shown, the projected HARRF flow is expected to exceed the permitted average annual daily flow capacity of 18.0 mgd in approximately 2014 and will reach the projected build-out average annual daily flow of 27.5 mgd in approximately 2041. (Source: *Flow Projection Analysis TM, Brown and Caldwell, December 1, 2006).* The projected flows were derived by plotting the annual average daily flow recorded from 2000 to 2005, developing a trendline from the anticipated growth in 2006 to 2008 in the HARRF sphere of influence, and extending it to future years. The HARRF treatment capacity was determined to be less than the permitted capacity of 18.0 mgd average annual daily flow, so treatment capacity will need to be expanded to match increasing flows.

Also depicted in Figure ES-1 is a stepped line that describes the proposed phased improvements that must occur at the HARRF to keep pace with the population and development growth within the sphere of influence.

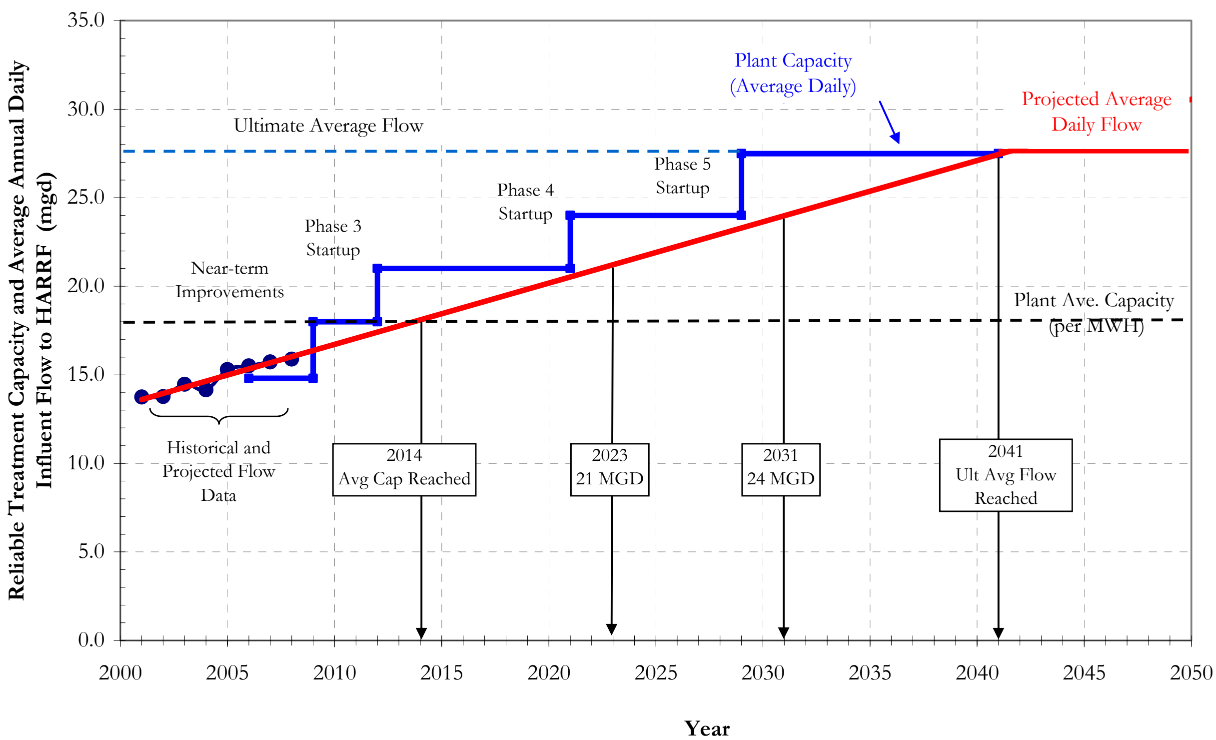


Figure ES-1. Proposed Phased Improvements to Achieve Reliable Treatment Capacity and Projected Average Annual Daily Influent Flow at the HARRF

As shown in Figure ES-1, plant improvements are proposed to occur in four phases (including the near-term improvements) to minimize overbuilding in any phase, thus optimizing capital expenditures and ensuring that the reliable treatment capacity is always greater than the projected flow.

An estimated schedule for planning, design, and construction for the four phases is shown in Table ES-4. Phase numbering designation begins with "Phase 3" since the plant has recently undergone Phase 1 and 2 improvements. Note that the actual timing may differ, depending on the actual rate of development and increase in flows. The timing shown in Table ES-4 is based on a single construction contract for each phase. Plant startup for a particular phase improvement is assumed to occur two years prior to the time when the anticipated flow reaches the plant capacity before the improvements. This time gap will allow operators to get familiar with the new process units and optimize the new systems. Figure ES-2 illustrates the capacity increases in individual unit processes with each phase, using Alternative 3B for illustration. Specific improvements associated with each phase are described below.

Under the current NPDES permit, the City is required to submit a written report to the Water Board "within 90 days after the monthly average influent flow rate for any 30-day period equals or exceeds 75 percent of the design capacity (13.5 mgd) of the waste treatment and/or disposal facilities." The report must include the City's "intended schedule for studies, design, and other steps needed to provide additional capacity for the waste treatment and/or disposal facilities, and/or control the flow rate before the waste flow exceeds the capacity of present units." Considering that the HARRF received on average 15 mgd in 2005, this 75 percent of capacity trigger/criterion is in effect. However, this study, which considers a phased construction plan to make certain the HARRF continues to have adequate capacity to treat incoming wastewater flows until buildout, is believed to meet this criterion.

Table ES-4. Planning Schedule for Design and Construction Improvements at the HARRF

Phase	Reliable Treatment Capacity After Improvement, mgd[a]	Initiate Planning/Design	Initiate Construction	Plant Startup	Year When Capacity is Reached
Near-Term	18.0	2007	2008	2009	2014
Long-Term-Phase 3	21.0	2007	2009	2012	2023
Long-Term-Phase 4	24.0	2017	2019	2021	2031
Long-Term-Phase 5	27.5	2027	2028	2029	2041

(a) Average dry weather flow capacity with one unit out of service for each process.

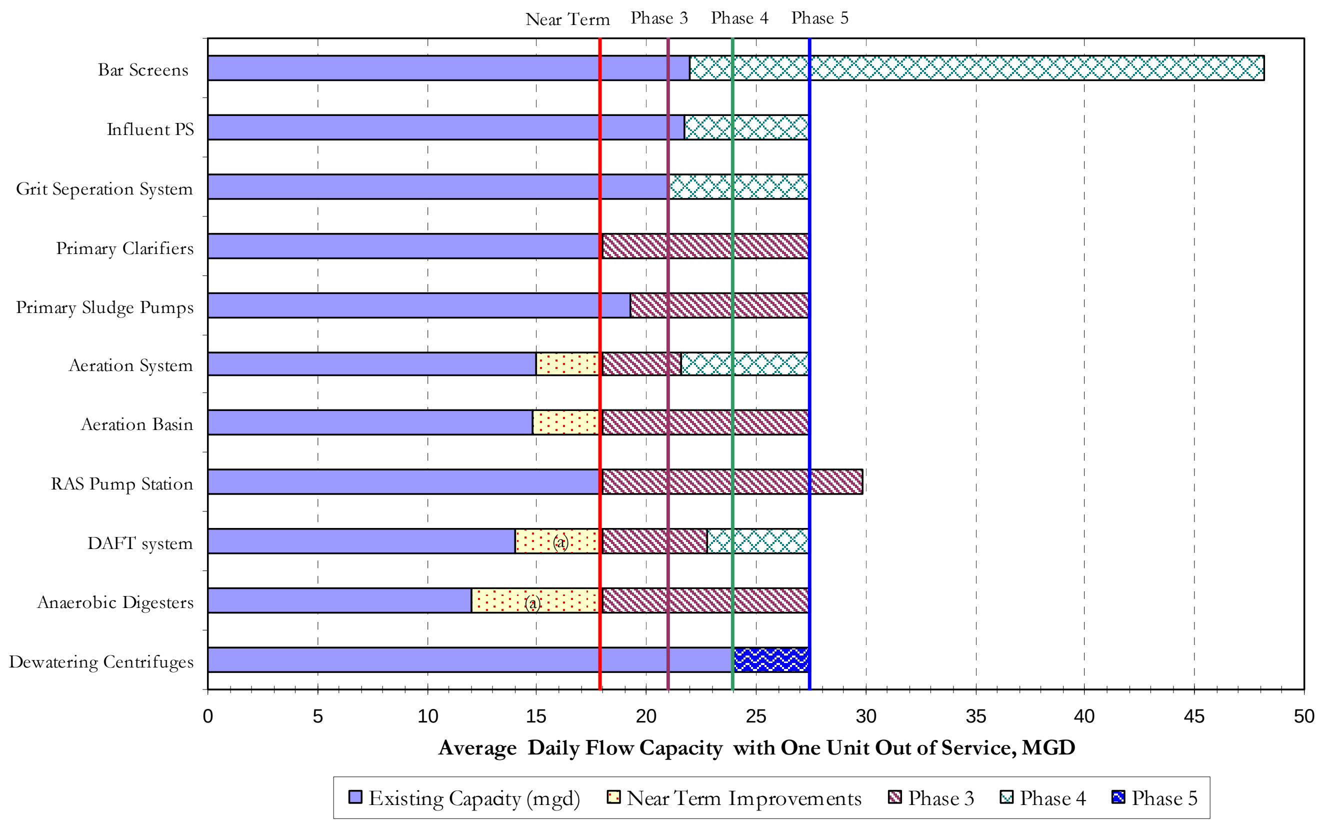


Figure ES-2. Capacity Increases for Each Phase at the HARRF

Note: (a) All the units are in operation during near-term improvements.

Phase 3 (21.0 mgd). Phase 3 includes the following improvements:

- Construction of one primary clarifier and associated primary sludge pumping
- Construction of one aeration basin with anaerobic selector
- Replacement of the fine-bubble aeration system in the five existing aeration tanks
- Retrofit of anaerobic selectors in five existing aeration basins
- Implementation of hydraulic improvements to aeration basin inlet and outlet gates, and secondary clarifier influent orifices
- Construction of a new waste activated sludge pump station
- Upsizing of the existing waste activated sludge pipe
- Construction of one new dissolved air flotation thickener
- Construction of one new anaerobic digester sized to handle 27.5 mgd
- Improvement of the headworks and primary clarifier odor control system
- Chemical storage and feed facilities for chemically enhanced primary treatment
- Replacement of the existing return activated sludge pumps with two larger capacity pumps

Phase 4 (24.0 mgd). Phase 4 includes the following improvements:

- Conversion of the existing manual bar screen to a mechanical bar screen
- Modifications to the influent pump station and force main
- Addition of sludge de-gritting system
- Addition of one aeration blower
- Construction of one new dissolved air flotation thickener

Phase 5 (27.5 mgd). Phase 5 includes the following improvements:

- Addition of one dewatering centrifuge
- Modification of the existing centrifuge feed and drain piping

Detailed discussion on each phase of construction is provided in the main body of this project report.

Long-Term Improvements to the Tertiary System

It is assumed that the replacement of the existing Dynasand filters with membrane filters, and improvements to the existing UV and chlorine contact systems will be included in one of the construction phases discussed above. The chlorine contact improvements include partitioning the chlorine contact tank into two or three parallel tanks to optimize contact time and minimize chlorine use. The UV improvements include UV bulb replacement and hydraulic improvements to optimize the flow split between the two channels and the flow pattern within each channel. These improvements coupled with demonstrating a higher secondary effluent UV transmittance should increase the UV capacity allowed by the Department of Health Services.

Construction Phasing of Outfall Improvements

Minor capacity improvements can be obtained by sealing Manhole 74, and sealing the inlet and outlet manholes for all siphon structures. Prior to conducting this work, further investigation of the pipe condition and hydraulic impact associated to these improvements is recommended.

The schedule for implementing the outfall improvement projects, as shown in Table ES-5, depicts a series of intermediate studies and projects which ultimately result in new disposal facilities (ie; new outfall or storage facilities). As the peak flow for the 10-year wet-weather event exceeds the current Escondido Land Outfall capacity, the schedule is based on the earliest reasonable time for conducting the studies and construction projects. Prior to the estimated completion date (2010) of constructing a new outfall/storage facility, the outfall will continue to have insufficient capacity resulting in potential non-compliant spills. In order to alleviate these disposal constraints, the following operational improvements are recommended:

- Prepare for known capacity demands based on forecasted rainfall events
- Investigate creek flows during wet-weather events and evaluate live-stream dilution factors

Table ES-5. Recommended Escondido Land Outfall Improvement Projects and Phasing

Phase	Description	Estimated Project Completion	Projected Peak Flow	Capacity Difference (mgd)	Escondido Land Outfall Capacity (mgd)
Immediate	Optimize capacity management procedures	2006	27.6	-6.2	21.4
Near-Term	Conduct condition assessment survey of Escondido Land Outfall	2007	28.2	-6.8	21.4
Near-Term	Seal Manhole 74	2007	28.2	-4.5	23.7
Near-Term	Seal inlet and outlet manholes local to siphons	2007	28.2	-3.2	25.0
Near-Term	Conduct outfall / storage alignment study	2007	28.2	-3.2	25.0
Near-Term	Design new outfall / storage facilities	2008	28.8	-3.8	25.0
Long-Term	Construct new outfall / storage facilities	2010	30.1	18.9	49.0

The San Elijo Ocean Outfall has maximum hydraulic capacity of 25.8 mgd. The attenuated 10-year design peak wet-weather flow effluent flow from HARRF (27.6 mgd) combined with the maximum allowable peak flow discharged from San Elijo Joint Powers Authority (5.3 mgd) exceeds the current capacity of the San Elijo Ocean Outfall. Therefore, similar to the Escondido Land Outfall, it is recommended immediate steps are taken to expand or replace the existing San Elijo Ocean Outfall.

The peak flow currently received at the San Elijo Ocean Outfall is hydraulically limited by the upstream Escondido Land Outfall capacity (21.4 mgd) and dependent on the operation of the San Elijo Joint Powers Authority regulator structure. As the Escondido Land Outfall capacity is limited by the San Elijo Ocean Outfall (via the regulator structure) construction of a new land outfall must be conducted in parallel with the expansion of the San Elijo Ocean Outfall. Depending on the San Elijo Ocean Outfall expansion options described in this report, the following key activities will need to be conducted in order to expand or replace the San Elijo Ocean Outfall:

- Apply and obtain permits to construct both off and on-shore segments of the new San Elijo Ocean Outfall (3-4 years)
- Conduct basis-of-design studies including (1- 2 years)
  - Geotechnical investigations
  - Alignment study
  - Capacity re-evaluation study
  - Environmental impact study
  - Pipeline pre-design
- Design on/off-shore outfall pipeline (1 year)
- Construct on/off-shore outfall (2 years)

Effluent Disposal Improvement Options

Currently, treated wastewater from the HARRF is disposed of by discharge through the Escondido Land Outfall/San Elijo Ocean Outfall system and intermittent live-stream discharge to Escondido Creek during wet weather when the creek flow rate exceeds a threshold value. In addition, a portion of the treated wastewater is further treated to produce recycled water, which reduces the flow to the outfall system through landscape irrigation, turfgrass irrigation, and cooling tower evaporation. Three disposal and equalization options considered during this study include upgrading or building a new outfall (land and ocean), expanding equalization facilities at the HARRF, and using live-stream discharge. The goal is to find the solution that satisfies these City criteria:

- Disposal and storage of peak flow and volume for a 10-year design event;
- Maximize use of live-stream discharge;
- Maximize use of existing disposal and equalization facilities; and
- Minimize the cost of building new facilities.

Table ES-6 summarizes the eight scenarios considered for improving the disposal of treated wastewater. These scenarios include combined hydraulic use of the land outfalls, equalization, and live-stream discharge. Of the eight, six were considered viable and were further evaluated and cost estimates were developed for each. Note that the capacity and size of the land outfall indicated in Table ES-6 are for a pipeline that will convey the entire effluent requiring disposal under the scenario and replace the existing Escondido Land Outfall. The City may opt to rehabilitate the existing Escondido Land Outfall to provide redundancy or abandon it altogether. The costs for rehabilitation and abandonment are discussed later.

Table ES-6 Land Outfall/Equalization/Live-Stream Discharge Scenarios

Scenario ID	Total Land Outfall Capacity (mgd)	New Land Outfall Size (inch)	Equalization Capacity (million gallon)	Live-Stream Discharge (mgd)	Facility Improvements	Notes
A	49.0	72	0.0	0.0	New Land Outfall	1,6,7
B	25.0	N/A	28.5	0.0	New Land Outfall (minor) + Equalization	2,3
C	33.0	42	25.0	0.0	New Land Outfall + Equalization	4,7
D	33.0	42	12.0	9.0	New Land Outfall + Equalization + Live-Stream Discharge	4,6,7
E	33.0	42	2.0	15.0	New Land Outfall + Equalization + Live-Stream Discharge	4,6,7
F	33.0	42	0.0	20.0	New Land Outfall + Live-Stream Discharge	4,6,7
G	38.0	54	0.0	15.0	New Land Outfall + Live-Stream Discharge	5,6,7
H	45.0	72	0.0	9.0	New Land Outfall + Live-Stream Discharge	5,6,7

Notes

Shaded boxes indicate most viable scenarios. The size and capacity indicated for the new land outfall are for a pipeline that will completely replace the existing Escondido Land Outfall.

1. Land outfall upgraded by constructing a new outfall providing a total capacity of 49 mgd. Actual pipe size may be as small as 54 inches in diameter, depending on the alignment and slope. Size shown is the largest required.
2. Capacity of existing land outfall increased to 25.0 mgd by sealing siphon manholes.
3. Equalization capacity includes 3.5 million gallon required to attenuate dry weather flows.
4. Outfall capacity designed to dispose of peak build-out dry weather flow (33 mgd).
5. Outfall capacities derived from peak flows reduced due to live-stream discharges.
6. Viable options accounting for equalization construction / site requirements.
7. New land outfall size will vary based on profile determined from alignment study.

Cost Estimate of Improvements

Order-of-magnitude construction cost estimates were developed for the improvements recommended in this study. Costs are reported in 2006 end-of-calendar year dollars. Developing operation and maintenance costs was not within this study's scope, nor were land costs included. Costs were developed for the near-term and long-term improvements discussed previously. The near-term improvements to ensure the HARRF has 18.0 mgd average dry weather flow treatment capacity are estimated to cost about $ 3.9 million.

The 27.5 mgd average dry weather flow treatment capacity at the HARRF can be achieved by implementing either Alternative 3 or 6, which are the viable long-term improvements considered for the HARRF (refer to Table ES-2 for further explanation of the alternatives). Tables ES-7 presents the costs for Alternatives 3 and 6 to upgrade the treatment capacity to 27.5 mgd.

Capital cost of Alternative 3A is the lowest ($119 million). Cost of implementing Alternatives 3B and 3C are $137 and $139 million, respectively, for the sludge co-thickening option. Alternatives 6A and 6B costs are estimated at $126 and $144 million, respectively, for the sludge co-thickening option.

Operation and maintenance cost for the 9.0 mgd biological aerated filter, membrane bioreactor, and membrane filtration units are estimated to show the relative difference in cost of operating these systems to reliably produce 9.0 mgd tertiary effluent. The annual operational and maintenance cost of the 9.0 mgd biological aerated filter system (sub-alternative A) is expected to be $620,000, which is about 10 and 50 percent lower than the 9.0 mgd membrane filtration (sub-alternative B) and membrane bioreactor (sub-alternative C), respectively.

Phasing of the improvement to the ultimate capacity of 27.5 mgd will involve additional costs mainly associated with mobilization and demobilization, and the difficulties encountered when having to construct while keeping existing process units in service. For illustration of this cost impact, Table ES-8 presents the costs associated with long-term secondary and tertiary treatment improvements at the HARRF for Alternative 3B in phases. Phasing is anticipated to add 3-5 percent to the costs reported in Table ES-7.

The near-term improvements to increase the capacity of the existing Escondido Land Outfall are estimated to cost about $5.8 million, which includes conducting a closed circuit television inspection of the entire length of the existing land outfall, and sealing manhole 74 and the inlet and outlet manholes for all the siphon structures (total of 12 manholes).

Table ES-9 shows costs for different effluent disposal scenarios at the HARRF. Comparatively, Scenario A had the lowest cost ($450 million) and Scenario H was the most expensive ($548 million).

Table ES-7. Planning-Level Project Cost for Long Term Improvement Alternatives for HARRF Average Flow of 27.5 mgd (in 2006 End-of-Calendar Year Dollars)

Alternative	Description	Sludge Thickening Option	Total Project Cost ($Million)
3A	High Rate Conventional Activated Sludge with Chemically Enhanced Primary Treatment and 9 mgd Biological Aerated Filter	Co-thickening	**119**
		Separate	138
3B	High Rate Conventional Activated Sludge with Chemically Enhanced Primary Treatment and 9 mgd Microfiltration	Co-thickening	**137**
		Separate	156
3C	High Rate Conventional Activated Sludge with Chemically Enhanced Primary Treatment and 9 mgd Membrane Bioreactor	Co-thickening	**139**
		Separate	166
6A	Moving Bed Bioreactor with 9 mgd Biological Aerated Filter	Co-thickening	**126**
		Separate	145
6B	Moving Bed Bioreactor with 9 mgd Membrane Bioreactor	Separate	**144**
		Separate	163

Table ES-8. Planning-Level Project Cost for Alternative 3B – Phased Construction (in 2006 End-of-Calendar Year Dollars)

Construction Phase	Total Project Cost ($Million)
Improvements for Expansion of Secondary Treatment Capacity	
Phase 3	75
Phase 4	11
Phase 5	3
Improvements for Expansion of Tertiary Treatment Capacity	
Phase 3	52

Table ES-9. Planning-Level Cost Estimate for Viable Effluent Disposal Scenarios (in 2006 End-of-Calendar Year Dollars)

Scenario	Summary of Improvements	Cost ($Million)	Total Cost ($Million)
A	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity using membrane filtration	137	450
	Escondido Land Outfall Improvements: ▪ New 72-inch-diameter pipeline with 49.0 mgd capacity	233	
	San Elijo Ocean Outfall Improvements: ▪ New 54-inch-diameter ocean outfall with 62.0 mgd capacity	80	
D	HARRF Improvements: ▪ 27.5 mgd secondary diameter ocean outfall with 62.0 mgd capacity	137	481
	Live-Stream Discharge: ▪ Membrane filtration with 9.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis with 9.0 mgd effluent capacity	98	
	Flow Equalization: ▪ Demolition of the existing 2.0 MG equalization basins ▪ Two 7 MG equalization basins construction	33	
	Escondido Land Outfall Improvements: ▪ New 42-inch-diameter pipeline with 33.0 mgd capacity	142	
	San Elijo Ocean Outfall Improvements: ▪ New 48-inch-diameter ocean outfall with 42.0 mgd capacity	71	
E	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	506
	Live-Stream Discharge: ▪ Membrane filtration with 15.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis with 15.0 mgd effluent production capacity	152	
	Flow Equalization: ▪ One 2 MG equalization basin construction	4	
	Escondido Land Outfall Improvements: ▪ New 42-inch-diameter pipeline with 33.0 mgd capacity	142	
	San Elijo Ocean Outfall Improvements: ▪ New 48-inch-diameter ocean outfall with 42.0 mgd capacity	71	
F	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	533
	Live-Stream Discharge: ▪ Membrane filtration with 20.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis for 20.0 mgd effluent production capacity	183	
	Escondido Land Outfall Improvements: ▪ New 42-inch-diameter pipeline with 33.0 mgd capacity	142	
	San Elijo Ocean Outfall Improvements: ▪ New 48-inch-diameter ocean outfall with 42.0 mgd capacity	71	

Table ES-9. Planning-Level Cost Estimate for Viable Effluent Disposal Scenarios (in 2006 End-of-Calendar Year Dollars)

Scenario	Summary of Improvements	Cost ($Million)	Total Cost ($Million)
G	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	530
	Live-Stream Discharge: ▪ Membrane filtration with 15.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis for 15.0 mgd effluent production capacity	152	
	Escondido Land Outfall Improvements: ▪ New 54-inch-diameter pipeline with 38.0 mgd capacity	170	
	San Elijo Ocean Outfall Improvements: ▪ New 48-inch-diameter ocean outfall with 48.0 mgd capacity	71	
H	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	548
	Live-Stream Discharge: ▪ Membrane filtration with 9.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis with 9.0 mgd effluent production capacity	98	
	Escondido Land Outfall Improvements: ▪ New 72-inch-diameter pipeline with 45.0 mgd capacity	233	
	San Elijo Ocean Outfall Improvements: ▪ New 54-inch-diameter ocean outfall with 58.0 mgd capacity	80	

Note: Description of the discharge scenarios included here is given in Table ES-6

Key assumptions for the estimates presented in Tables ES-7, ES-8 and ES-9 include the following:

General

- The reported costs are planning-level estimates with -35 percent to +50 percent accuracy.
- Costs include all necessary earthwork, grading and sheeting/shoring necessary.
- Costs for alternatives requiring placement of the facilities outside of the current plant boundaries do not include land and/or easement acquisition costs. Costs for Escondido Land Outfall improvements requiring a secondary outfall do not include land and/or easement acquisition cost as well.

HARRF Improvements

- The HARRF improvement costs shown in Table ES-9 are for Alternative 3B-High-rate Conventional Activated Sludge with Microfiltration because it represented a mid-point cost of the three alternatives. Selection of Alternative 3A would result in about $18 million reduction in the overall cost. Selection of Alternative 3C would result in a $2 to $10 million increase.

- The HARRF improvement costs reported assume co-thickening will be practiced. Implementing improvements to allow separate thickening will increase the costs reported in Table ES-7 by $19 to $27 million, depending on the improvement alternative selected.

Escondido Land Outfall Improvements

- Costs reported for Escondido Land Outfall improvements assume that a new land outfall will be constructed to convey all the flows from the HARRF requiring ocean disposal. They do not include the cost to abandon or refurbish the existing Escondido Land Outfall. Depending on the selected option, the following costs must be added to those reported in Table ES-9.
  - ✓ Rehabilitation is estimated to cost about $27 million for the following:
    - Closed circuit television inspection of the entire length of the existing Escondido Land Outfall,
    - Relining of 5 miles of the existing pipe,
    - 70 days of by-pass pumping,
    - Follow-up closed circuit television inspection of the relined pipe, and
    - Rehabilitation of 53 manholes.
  - ✓ Abandonment of the Escondido Land Outfall is estimated to cost about $3 million for the following:
    - Filling of the existing Escondido Land Outfall with sand
    - Removing the tops of manholes and filling them with slurry
- Costs for environmental mitigation and monitoring is included and assumed to be $2 million for construction of the new land outfall and $500,000 for abandonment and rehabilitation of the existing Escondido Land Outfall.

- The length of the new land outfall is assumed to be equal to 1.15 times the total length of the existing outfall (from the HARRF to the Regulator Structure). Two alignments were investigated. The longest route was found to be about 11 percent longer than the existing alignment. The 1.15 multiplier, which results in a longer length than those examined, should cover any deviations from the alignments examined.

San Elijo Ocean Outfall Improvements

- The San Elijo Ocean Outfall improvements assume that a new parallel ocean outfall will be constructed and that the 79:21 capacity ratio between the City and the San Elijo Joint Powers Authority is maintained in the future. The savings in downsizing to the next lower size (a reduction of 6 inches in diameter) to convey only the HARRF effluent flows in new ocean outfall result in a decrease in installed cost of between $2 to $9 million.

Live-stream Discharge

- The cost for reverse osmosis is included in some of the disposal options that require intermittent live-stream discharge. Based on the current permit allowing intermittent discharge, the City is required to redirect Escondido Creek flows back to the HARRF during the dry weather periods to recover the amount of nutrients (i.e., nitrogen and phosphorus) discharged to the creek during the wet weather season. There is a concern that the varying quality of Escondido Creek will not yield sufficient amount of nutrients to recover the quantities discharged to the creek. Therefore, provisions are included to remove nitrogen and phosphorus in the tertiary effluent. Reverse osmosis is selected because it can easily be "switched on" when needed without the need for acclimation. Pilot testing should be conducted to determine the effectiveness of reverse osmosis in removing nitrogen under local conditions.

Flow Equalization

- Improvements noted in Table ES-7 are in addition to the 7.0-million-gallon equalization storage basins already in place

Soft Costs – HARRF and Outfall Improvements

- Assumed soft costs for the HARRF and the outfall improvements are presented in Table ES-10. The difference between the assumed soft costs between the HARRF and outfall improvement is explained under the remarks section of the table.

Table ES-10. Soft Costs for the HARRF and Outfall Improvements

Item	HARRF Improvements	Outfall Improvements	Remarks
Electrical and Instrumentation	22 percent of mechanical, piping, and building cost	None	Outfall improvements does not require any electrical and instrumentation component
Construction Contingency	40 percent of raw construction cost	50 percent of raw construction cost	Currently there are more unknowns for the outfall improvements such as the geotechnical information
Contractor Overhead and Profit and General Conditions	22 percent of raw construction cost	17 percent of raw construction cost	Additional cost for construction equipment, trailers, and temporary utilities related to the HARRF improvements.
Escalation to End of 2006 Calendar Year	8 percent of raw construction cost	8 percent of raw construction cost	–

Table ES-10. Soft Costs for the HARRF and Outfall Improvements

Item	HARRF Improvements	Outfall Improvements	Remarks
Miscellaneous Markups	16 percent of raw construction cost	16 percent of raw construction cost	–
Engineering	20 percent of total capital cost	15 percent of total capital cost	Less engineering time and disciplines are needed for the outfall improvement project
SCADA	10 percent of total capital cost	None	No SCADA system is needed for the outfall improvement
Construction Management	10 percent of total capital cost	6 percent of total capital cost	More construction management, i.e. additional inspection services, is needed for the HARRF improvements due to complexity of the construction.
Legal and Administration	10 percent of total capital cost	10 percent of total capital cost	–

Assumptions that are not covered above, but are important in developing the cost for each scenario, are presented below.

Scenario D

- It is assumed that the existing 2-million-gallon equalization basin will be demolished and two new 7-million-gallon reservoirs will be constructed: one placed in the same location as the existing equalization basin and the other constructed outside of the current plant boundaries. Land acquisition cost is not included.

Scenario E

- An additional 2-million-gallon reservoir will be constructed outside of the current plant boundaries. Land acquisition cost is not included.

Scenarios F and G

- The additional 9.0 mgd membrane filters are assumed to be located in the parking lot south of the existing tertiary filters. The remaining additional 6.0 mgd membrane filters and 15.0 mgd chlorine contact basins and reverse osmosis system are assumed to be built outside of the current plant boundaries.

Scenario H

- The additional 9.0 mgd membrane filters are assumed to be located in the parking lot south of the existing tertiary filters. The remaining additional 11.0 mgd membrane filters and 20.0 mgd chlorine contact basins and the reverse osmosis system are assumed to be built outside of the current plant boundaries.

Other Recommendations

Further discussion on other recommendations is presented below.

Odor Control

- Replace primary effluent with secondary effluent or reclaimed water as the wetting agent in the bioscrubber.
- Establish continuous recycling of the wetting agent instead of using a pass-through effluent system.
- Install covers on the primary clarifiers and withdraw foul air from beneath the covers only to reduce the amount of foul air needing treatment from 66,000 cfm to 30,000 cfm.

Effluent Disposal

- Initiate an alignment study to determine possible routes for a new land outfall and determine the constraints involved.
- Conduct condition assessments of the existing land and ocean outfalls.
- Conduct near-term capital improvements to maximize the existing capacity of the Escondido Land Outfall.
- Evaluate the costs of acquiring land to construct a new land outfall.
- Periodically update the projected build-out flows using accurate available land use data.
- If considering expanding equalization at the HARRF, evaluate land and construction costs.
- Maintain the ability to dispose of 9.0 mgd to Escondido Creek during extreme conditions to provide disposal flexibility by renewing the current permit for intermittent live-stream discharge.
- Evaluate the advantages and disadvantages of constructing a separate ocean outfall that conveys only effluent from the HARRF.
- Consider a regional land and ocean outfall that can benefit other communities.

1.0 INTRODUCTION

The Hale Avenue Resource Recovery Facility (HARRF), a wastewater treatment facility owned and operated by the City of Escondido (City), is fast approaching its design capacity. To ensure that wastewater received at the HARRF continues to receive adequate treatment prior to disposal or reuse, the City has retained Brown and Caldwell to evaluate the capacity of the existing treatment facilities at the HARRF, the Escondido Land Outfall (ELO), and the San Elijo Ocean Outfall (SEOO).

The capacity study performed and discussed in this project report actually serves three purposes. The first purpose is to determine the capacity of the HARRF, ELO and SEOO using the latest and proven prediction technologies available to produce an accurate assessment. Previous capacity rating studies that set the current rating of the HARRF relied on design criteria reported in various industry-accepted texts and EPA publications. For this study, Brown and Caldwell used the latest plant performance data and conducted a two-week wastewater characteristics monitoring, settling characteristics evaluation, and a 30-day wet weather flow monitoring to calibrate process, storm flow, and hydraulic simulation models. These calibrated models were then used to determine the treatment and hydraulic capacities reported.

The second purpose of the study is to identify and determine order-of-magnitude costs for improvements that will allow the City to adequately treat incoming wastewater up to the current rated average annual daily flow capacity of 18.0 million gallons per day (mgd) and the projected build-out flow capacity of 27.5 mgd. Various treatment processes were evaluated based on reliability, space requirements, operation and maintenance requirements, and the ability for the process to reliably produce recycled water. Project costs for alternatives considered to be viable were developed and are reported in later sections.

The final purpose of the study is to identify and determine order-of-magnitude costs for improvements that will provide sufficient storage and disposal capacities during storm conditions to avoid spillage of treated or untreated wastewater to waterways/areas not approved by local regulatory agencies. From 38 years of rainfall data and about three years of the HARRF influent flow data (including flows recorded during the latest storm events); a 10-year peak flow event was simulated. Various effluent disposal options, including storage and equalization, live-stream discharge, and land and ocean outfall disposal were then developed and subsequently evaluated. Costs were determined for the most viable alternatives.

Individual process components of the HARRF as well as the land and ocean outfalls were analyzed in detail. Included in the appendices of this project report are the individual technical memoranda that provide extensive details on methodologies, results, findings, conclusions, and recommendations. A summary of the contents of the technical memoranda is presented in this project report. In addition, discussions on disposal options and preliminary, order-of-magnitude level costs estimates are presented.

2.0 BACKGROUND

Presented in this section are 1) information on the existing HARRF, ELO, and SEOO; 2) permits that regulate the treatment, storage, and disposal of wastewater at the HARRF; and 3) wastewater flow projections.

There are several terms used in this project report that relate to flow. The term average annual daily flow (AADF) describes the average daily flow over one calendar year and includes wet and dry weather flows. It is typically used when discussing annual flow projections and annual discharge mass emission rates. Average dry weather flow (ADWF) refers to a period when wastewater flows are not expected to be impacted by wet weather periods or high groundwater levels. For this study, the average daily wastewater flows recorded between August 1 and October 31 from 2000 to 2005 were considered as ADWF. Note that because Escondido is located in an arid environment with limited rainfall, there is less than a 5 percent difference between average annual flow and average dry weather flow. Peak wet weather flow (PWWF) describes a peak flow averaged over a one-hour period, which typically occurs during inclement weather. This term is typically used for design and to determine the hydraulic capacity of units and conveyance facilities.

2.1 Existing Facilities

In this section, the HARRF facilities, the ELO, and the SEOO currently in place are described. This information forms the basis of the recommended improvements presented in later sections.

2.1.1 HARRF

The HARRF was originally constructed in 1959 as a 1.0-mgd activated sludge facility and underwent modifications and upgrades in 1965, 1973, 1981, 1998, and 2000. The HARRF is currently rated to provide 18.0 mgd of secondary treatment and 9.0 mgd of tertiary treatment. In 2005, the HARRF treated an average of 15.3 mgd daily. The HARRF consists of the following conveyance and treatment units: mechanical and manual barscreens, grit removal systems, primary clarifiers, aeration basins, secondary clarifiers, flocculation basins, filters, UV and chlorine disinfection, chemical feed facility, dissolved air flotation thickeners (DAFT), anaerobic digesters, solids dewatering centrifuges, flow equalization basin, energy recovery system, and major pump stations including an influent pump station, a return activated sludge/waste activated sludge (RAS/WAS) pump station, a secondary effluent pump station, and a reclaimed water pump station. Physical descriptions and currently reported capacities of the primary, secondary, tertiary, solids process units, and other mechanical equipment are presented in Table 2-1. Results of the capacity assessment of each process are discussed in a subsequent section.

2.1.2 Escondido Land Outfall (ELO)

A majority of the treated wastewater from the HARRF is disposed through the land and ocean outfall. The ELO stretches 14.3 miles from the HARRF fence line to the SEOO and is composed of a series of 30-, 33-, and 36-inch-diameter pipelines. Treated effluent within the upper nine miles of the ELO flows by gravity; it flows under pressure for the remainder of the way.

Table 2-1. Characteristics of Existing HARRF Facilities
(Capacity Ratings Prior to this Capacity Study)

Item	Units	Values [(a)]	
Barscreens			
Type	-	Mechanical	
Manufacturer – Model	-	Parkson Aqua Guard	
Channel Width	ft	4.5	
Bar Spacing	mm	6	
Capacity, each[(b)]	mgd	22	
Type	-	Manual	
Manufacturer	-	-	
Channel Width	ft	3.5	
Bar Spacing	inch	2	
Influent Pump Station			
Constant Speed Pumps			
Number Installed	-	2	
Manufacturer	-	Allis-Chalmers	
Capacity, each	gpm	4,600	
Design head, each	ft	30	
Horsepower, each	hp	50	
Variable Speed Pumps			
Number Installed	-	2	2
Manufacturer	-	Fairbanks Morse	Fairbanks Morse
Capacity, each	gpm	5060	9,000
Design head	ft	30	40
Horsepower, each	hp	50	125
Headworks			
Grit Removal Chamber			
Number Installed	-	2	
Type	-	Vortex	
Manufacturer	-	Schloss Eng. Equip., Inc.	
Diameter	ft	24	
Total Process Capacity (@90 sec HRT) (one unit out of service)	mgd	21	
Grit Pumps			
Number Installed	-	4	
Type	-	Recessed Impeller	
Manufacturer	-	Wemco	
Capacity, each	gpm	220	
Classifier			
Number Installed	number	2	
Type	-	Helical screw	
Manufacturer	-	Schloss	
Capacity, each	gpm	220	
Motor Size, each	hp	½	

Table 2-1. Characteristics of Existing HARRF Facilities (Capacity Ratings Prior to this Capacity Study)

Item	Units	Values [a]	
Parshall Flume			
Number Installed	-	1	
Manufacturer	-	Plasti-Fab	
Throat width	ft	5	
Peak Capacity	mgd	53	
Primary Clarifiers			
Number of Primary Clarifier Basins Installed	-	3	1
Side water depth	ft	8	10
Surface area per tank	ft^2	5,250	5,250
Total Process Capacity @ 1,000 gpd/sf SOR			
All Clarifiers in Service	mgd	21	
Largest Clarifier Out of Service	mgd	None reported	
Primary Sludge Pump Station			
Diaphragm Pumps			
Number Installed	-	3	4
Manufacturer	-	Gorman Rupp	Dorr Oliver
Discharge Volume per Stroke, each	Gallons	4.5	3.8
Equivalent Average Total Capacity [c]	mgd	19.3	
Design head	ft	80	
Compressor			
Number Installed	-	2 (1 duty +1 standby)	
Manufacturer	-	Hydro Vane	
Capacity	scfm	96	
Pressure	psig	100	
Horsepower	hp	25	
Aeration System			
Aeration Basins			
Number Installed	-	5	
Side water depth	ft	16.5	
Surface area	ft2	10,000	
Total Process Capacity [d]	mgd	18.0	
Blowers			
Number Installed	-	3 (2 duty + 1 standby)	
Capacity, each	scfm	10,300	
RAS/WAS Pump Station			
RAS Pumps			
Number Installed	-	3 (2 duty + 1 standby)	3 (2 duty + 1 standby)
Type	-	Horizontal Non-Clog; Variable Speed	Horizontal Non-Clog; Variable Speed
Manufacturer	-	Fairbanks Morse	Fairbanks Morse
Capacity, each	gpm	4,107	2,173
Design head	ft	37	35
Motor Size	hp	60	25
Total Process Capacity [e]	mgd	37.8	37.8

Table 2-1. Characteristics of Existing HARRF Facilities (Capacity Ratings Prior to this Capacity Study)

Item	Units	Values [a]	
WAS Pumps			
Number Installed	-	2 (1 duty + 1 standby)	
Type	-	Variable Speed, Progressive cavity	
Manufacturer	-	Moyno	
Capacity, each	gpm	515	
Design head	ft	35	
Motor size	hp	5	
Secondary Clarifiers			
Secondary Clarifier Basins			
Number Installed	-	2	2
Diameter	ft	80	110
Side water depth	ft	15	15
Total Process Capacity			
All Clarifiers in Service @ 620 gpd/sf	mgd	18.0	
One Large Clarifier Out of Service @ 690 gpd/sf	mgd	18.0	
Secondary Effluent Pump Station			
Equalization Pumps			
Number Installed	-	4	
Type	-	Vertical turbine	
Manufacturer	-	Fairbanks Morse	
Capacity, each	gpm	5,675	
Design head	ft	27	
Motor size	hp	50	
Filter Influent Pumps			
Number Installed	-	3	
Type	-	Vertical turbine	
Manufacturer	-	Fairbanks Morse	
Capacity, each	gpm	3,470	
Design head	ft	34	
Motor size	hp	40	
Flocculation			
Flocculation Basins			
Number Installed	-	2	
Stages per basin	-	2	
Length of each stage	ft	16.75	
Width of each stage	ft	16.75	
Depth of each stage	ft	19.9	
Flocculation Mixers			
Number installed	-	4	
Manufacturer	-	Philadelphia	
Stage Number	--	1	2
Power requirement	hp	3	3

Table 2-1. Characteristics of Existing HARRF Facilities (Capacity Ratings Prior to this Capacity Study)

Item	Units	Values [(a)]	
Filters			
Filters			
Type	-	Standard bed, sand, continuous upflow backwash	
Manufacturer	-	Dynasand	
Number of Cells Installed	-	8	
Cell size, each	ft^2	200	
Filters per Cell	-	4	
Bed depth	inch	40	
Air Compressor			
Number Installed	-	2 (1 duty + 1 Standby)	
Type	-	Rotary vane	
Manufacturer	-	Hydro Vane	
Capacity, each	scfm	95	
Pressure, each	psig	125	
Power requirement	hp	125	
UV Reactors			
Type	-	Low pressure, high intensity	
Manufacturer	-	Trojan (3000)	
Number of channels	-	2	
Channel length	ft	56	
Channel width	ft	5	
Channel depth	in	78	
No. of bank lamps per channel	-	5 (4+1)	
No. of lamps per bank	-	320	
Chlorine Contact Chamber			
Width	ft	11.25 (long channels) and 9.75 (short channels)	
Total Length	ft	961	
Min. Water Depth (weir crest)	ft	9	
Rated Capacity	mgd	10.35	
Modal Contact Time at Rated Capacity	min	135	
Target Effluent Chlorine Residual	mg/L	2.5	
Transfer Pump Station			
Number Installed	-	3	
Type	-	Vertical turbine	
Manufacturer	-	Fairbanks Morse	
Capacity, each	gpm	3,470	
Design head	ft	34	
Reclaimed Water Pump Station			
Reclaimed Water Pumps			
Number Installed	-	2	3
Type	-	Vertical turbine	
Manufacturer	-	Fairbanks Morse	Fairbanks Morse
Capacity, each	gpm	750	1,550
Design head	ft	580	580
Motor size	hp	150	300

Table 2-1. Characteristics of Existing HARRF Facilities
(Capacity Ratings Prior to this Capacity Study)

Item	Units	Values [a]		
Utility Water Pumps	number		4	
Capacity, each	gpm		600	
Design head	ft		208	
Motor size	hp		50	
Dissolved Air Flotation Thickeners (DAFT)				
DAFT Units	ID Number	1		2
Number Installed	-	1 (East)		1 (West)
Diameter	ft	35		35
Side water depth	ft	8.5		10.5
Thickened Sludge Pumps				
Number Installed	-	2 (1+1)		2 (1+1)
Type	-		Progressive cavity	
Manufacturer	-	Seepex		Seepex
Capacity, each	gpm	260		260
Design head	ft	50		36
Pressurization System				
Number Installed	-	1		1
Pump type	-		Centrifugal	
Manufacturer	-	Peerless		Peerless
Pump capacity, each	gpm	450		500
Pump pressure	ft	162		175
Compressor type	-		Piston	
Manufacturer			Comp Air	
Capacity, each	scfm	15		17.2
Capacity, each	lb/hr	67.4		17.3
Pressure	psig	100		100
Pressurization tank size	-	4'-6" x 5'-4"		4'-6" x 5'-4"
Liquid depth	inch	30		30
Liquid volume	gal	328		328
Anaerobic Digesters				
Digesters ID Number		1	2	3
Digester type	-	Primary	Primary	Primary
Tank Diameter	ft	80	85	85
Side water depth	ft	25	25	25
Unit volume	1000 gal	940	1,061	1,061
Mixing				
Number of Draft Tubes Installed	-	1	3	3
Number of Lances Installed	-	4	6	6
Lance Diameter	inch	3	3	3
Gas Compressor type			Rotary Lobe	
Number	number		3 (2+1)	4 (3+1)
Capacity	scfm		1,000	
Pressure	psig	6.5		7-8

Table 2-1. Characteristics of Existing HARRF Facilities (Capacity Ratings Prior to this Capacity Study)

Item	Units	Values (a)		
Recirculation Pump type	-	Centrifugal		
Manufacturer	-	Vaughn		
Number Installed	-	1		3
Capacity	gpm	220-250		220-250
Operating head	ft	24		24
Motor size	hp	5		5
Heat Exchangers				
Number Installed	-	1	1	1
Type	-	Spiral	Spiral	Spiral
Manufacturer	-	Alfa Laval	Alfa Laval	Alfa Laval
Size	Million BTU	1	1	1
Dewatering System				
Storage Tank (Secondary Digester)				
Diameter	ft	55		
Sidewater depth	ft	23		
Volume	1000 gal	409		
Dewatering Equipment				
Number Installed	-	2 Duty, 1 Standby		
Type	-	Centrifuge		
Manufacturer, Model	-	Andritz, D5L		
Capacity, each	gpm	150		
Operating schedule	hrs per day	12		

Notes:

(a) Sources (unless indicated otherwise): *Final Letter Report for Capacity Rerating of the Hale Avenue Resource Recovery Facility (HARRF*), MWH, May 11, 2004 (process capacity reported in the letter report is provided in the table); Operation and Maintenance Manual for Phase 2 Improvements, MWH, 2005; *Volume 2 – Drawing for the Construction of Hale Avenue Resource Recovery Facility Phase 2 – Treatment Upgrades and Water Reclamation Facilities As-built Drawings*, MWH, June 1999; *Construction of Hale Avenue Wastewater Treatment Facilities Expansion Volume 2-Drawings,* JMM, October 1981; HARRF Plant personnel

(b) Source: Joe Nagel of Parkson, June 5, 2006 e-mail.

(c) Six pumps in service – equivalent process capacity noted

(d) Assumes SRT = 3.8 days and MLSS 2,500 mg/L

(e) Assumes all duty pumps in service and RAS Flow = 0.625 * Influent Flow

2.1.3 San Elijo Ocean Outfall (SEOO)

The SEOO consists of two main segments: the land segment, which extends from the property line of the San Elijo Water Reclamation Facility to the Cardiff State Beach; and the ocean segment, which extends approximately 8,000 feet off shore. The SEOO was constructed in two phases. Phase I was completed in 1965, consisting of a 4,000-foot-long, 30-inch-diameter reinforced concrete pipe (RCP) that terminated at a depth of approximately 55 feet. A 192-foot-long, 30-inch-diameter pipe was added at a southward right-angle bend from the 4,000-foot main line. This pipe and the final 120-foot section of the main line included diffusers that provided a minimum initial dilution of 120 to 1. The Phase I outfall system was rated at 15.0 mgd and, most notably, the main line was designed for an internal pressure of 50 feet (*Hale Avenue Resource Recovery Facility Phase II Treatment Process Upgrades and Enhancements Facility Plan,* Water 3 Engineer, Inc., March 1999).

Phase II of the outfall construction was completed in 1974 mainly because discharge from the HARRF was diverted from the Escondido Creek to the ocean outfall. The outfall modification extended the terminus another 4,000 feet towards the ocean, consisted of 48-inch double-rubber-gasketed RCP, and terminated at a depth of 148 feet below mean sea level. New diffusers were installed within the final 1,200-foot segment of the new extension, and the old diffusers along the 1965 outfall were capped. A total of 200 diffusers exist, providing 237:1 initial dilution of the discharged effluent. The diffuser has two hundred 2-inch-diameter ports spaced six feet apart through the diffuser pipe side wall. The latest underwater video inspection by Thales GeoSolutions (Pacific), Inc., performed in 2003 shows some of the ports are impaired by debris and possibly by encroaching marine growth. The ballast rock protection for the pipeline has somewhat deteriorated because of the wave action and sand movement. Recent repairs have addressed these deficiencies. History has shown that the ballasts must be inspected and repaired (if necessary) at least once every ten years.

2.2 Permits

Existing permits regulating the discharge to the ocean outfall, the intermittent discharge to the Escondido Creek, the use of recycled water produced at the HARRF, and the discharge of brine received from industries are described in this section.

2.2.1 Ocean Outfall Discharge

The current National Pollutant Discharge Elimination System (NPDES) permit (CA0107981, Order No. R9-2005-0101) issued by the San Diego Regional Water Quality Control Board (Water Board) limits the monthly average HARRF effluent discharge flow rate to 18.0 mgd. The City can, however, discharge as much as 20.1 mgd (or 79 percent of the current rated capacity of the ocean outfall of 25.5 mgd) at peak conditions. The outfall rating is mainly based on the most sensitive segment of the outfall alignment – the nearshore pipes which are rated to withstand up to 50 feet of internal pressure. The percentage allocated to the City is a contractual agreement with the owner and manager of the outfall, the San Elijo Joint Powers Authority (SEJPA), who share the use of the ocean outfall. Twenty-one percent of the remaining portion, or 5.3 mgd, is reserved for SEJPA to allow discharge of treated wastewater from the San Elijo Water Reclamation Facility (SEWRF).

2.2.2 Intermittent Live-Stream Discharge

On December 10, 2003, the Water Board adopted Order No. R9-2003-0394, NPDES Permit No. CA0108944, allowing the City to discharge up to 9.0 mgd of tertiary treated effluent, provided that all of the following conditions are met (Section A.1.3 of the Order):

1. *The discharge to the San Elijo Ocean Outfall from the HARRF and the San Elijo Water Pollution Control Facility (a.k.a. San Elijo Water Reclamation Facility) exceeds the maximum capacity of the outfall.*

2. *All emergency in-plant storage has been used.*

3. *Stream flows recorded at the County of San Diego's stream gauging station, located approximately 100 yards upstream of the HARRF, exceed an average flow of 300 cubic feet per second during the discharge and are not below 100 cubic feet per second at any time during the discharge.*

4. *The mouth of the San Elijo Lagoon is open or the Regional Board Executive Officer approves otherwise.*

5. *The discharge occurs between November 1 and April 30.*

2.2.3 Recycled Water

Recycled water production was first evaluated at the HARRF in 1991. Demands identified early in the recycled water program include industrial and irrigation uses within the City and in the Rincon del Diablo Municipal Water District (Rincon). The program was divided into three phases. Phase I was estimated to involve a total of 3,400 acre-feet per year (afy) or 2.6 mgd average annual use, while Phase II consisted of 900 afy of demand, increasing the average annual demand to 3.3 mgd. The ultimate reuse system will reportedly provide more than 4,500 afy or 4.0 mgd average and 7.9 mgd peak day demand (*Hale Avenue Resource Recovery Facility Phase II Treatment Process Upgrades and Enhancements Facility Plan*, Water 3 Engineering, Inc., March 1999).

In 1993, the Water Board adopted Order No. 93-70 which allowed the City to discharge 3.0 mgd average annual and 5.0 mgd peak day flow of wastewater treated to Title 22 standards. Uses identified included irrigation of golf courses, parks, street landscape, schools, agriculture, and other landscape areas which previously used potable water for irrigation. In 1999, Order No. 93-70 was amended, increasing the allowable peak reuse rate to 9.0 mgd. In the same amended Order, it was identified that ultraviolet (UV) light would replace chlorination for disinfection of the tertiary-treated wastewater. The specified minimum UV dose required (under worst operating conditions) was noted as 140 milliwatts seconds per square centimeter ($mW\text{-}s/cm^2$). The amended Order also indicated that coagulation was not required as long as the filter effluent turbidity did not exceed 2 nephelometric turbidity units (NTU) and a coagulation system can be automatically activated if the influent turbidity exceeded 5 NTU.

Recent demands for the HARRF recycled water has a significant impact on the effluent disposal. The Palomar Energy Center (PEC), a 550-megawatt power plant, intends to use 4.3 mgd (average) to 7.2 mgd (peak) of recycled water for cooling purposes. The reuse water quantity depends on the number of power-generating engines being operated, which is governed by the power demands of the area served. During the cooling process, water is lost to the environment through evaporation, which is estimated to be up to 2.7 mgd (*Attachment F- Fact Sheet, Order No R9-2005-0139*).

2.2.4 Industrial Brine Discharge

The consequence to the cooling process is the concentration of dissolved solids in the process stream. This brine solution will be returned to the HARRF through a pipeline called the Industrial Brine Collection System (IBCS) for dechlorination and mixing with the HARRF effluent prior to final disposal through the outfall. It is expected that an average of 1.0 mgd and a maximum of 1.4 mgd of brine from the PEC may be returned to the HARRF. Minor amounts of brine discharges from Boncor, Culligan, and Goal Line L.P. also will be returned along the IBCS for a total brine discharge of 1.5 mgd to the outfall. This brine discharge is a portion of the total 18.0 mgd allowed to be discharged to the ocean. The Water Board adopted Order No. R9-2005-0139, NPDES Permit No. CA0109215, on September 14, 2005 that allowed the City to discharge the brine waste to the ocean via the ELO and the SEOO.

2.3 Projected Flows

Sewer flows collected and transported by a sanitary collection system are comprised of both dry weather and wet weather flows. Dry weather flows are generated primarily of residential, commercial, and industrial land uses. The land use impacts the magnitude of flow, and the daily and seasonal patterns. In addition to land-use based flows, dry weather flows usually are comprised of ground-water infiltration flows generated from a variety of man-made and natural sources. These flows enter the collection system via pipe cracks, fissures, illegal connections and private laterals.

Wet weather flows are generated by rain-related inflow and infiltration flows entering the collection system. The magnitude and timing of these flows typically creates a "worst-case" peak flow scenario impacting both the collection system and the treatment facilities. The magnitude of these flows is dependent on the structural condition of the collection system. For example, an old system with significant cracks and fissures will create high wet weather peak flows, whereas a new system will generate significantly lower wet weather flows. Therefore, the system age, current condition and future rehabilitation projects will all impact future wet weather flows.

The method deployed to estimate the projected average annual flows at the HARRF involved the following steps:

1. Identify and classify land development projects (planned, under-construction, or complete) from 2006 through to 2010.
2. For each development project, identify the land-use type, building size, dwelling units, and appropriate unit flow factors.
3. Calculate the average daily flow generated from each development project and totalize for each future year.
4. Develop cumulative annual average flows from 2006 through to 2010 by adding the future flows to the existing 2005 flow at the HARRF.
5. Using historical and future flows through 2008, plot a linear relationship between flow and year. Note, future flows from 2009 and 2010 were not used as limited knowledge of planned developments skewed the projection.
6. Compare projected flows with historical sewer connection trends to verify analysis.

7. Estimate time period (year) when the projected average daily flow will reach the current rated plant capacity of 18.0 mgd and the estimated build-out average daily flow of 27.5 mgd.

The following assumptions were used during the flow projection analysis:

- No increase in ground water and wet weather inflows resulting from a "trade-off" between increased development and collection system rehabilitation improvements.
- Future annual average wet weather and ground water flows remain constant and equal to the 2005 wet weather flows.
- Sewer discharge per capita flow rates remain constant through to build-out.
- Sewer discharge unit flow rates:
  - Residential: 250 gallons per day (gpd)/dwelling unit
  - Commercial: 1500 gpd/acre
  - Industrial (Light) 2000 gpd/acre
  - Industrial (Heavy) 5000 gpd/acre
  - Hospital: 210 gpd/bed
  - Restaurant: 30 gpd/seat
- Future land development is not limited by available developable land. This assumption is countered by future developments occurring through densification.

Land Developments

The land development projects (proposed, in-design and under construction) were summarized by the City and presented to Brown and Caldwell for analysis. Additional information describing the land use, acreage, dwelling units and estimated year of completion were obtained from further discussions with the City, specific plan documents and the City's Planning Commission meeting minutes obtained via the City's public web site.

The development projects, as summarized in Table 2-2, were allocated into appropriate years of completion ranging from 2007 through to 2010. The projects collated and presented in this analysis are expected to change due to economic, environmental and political issues. In addition, projects listed in 2009 and 2010 are considered under-estimates and will most likely increase as the City continues to grow.

The following assumptions were used during the analysis of the development projects:

- North County Transit District cleaning facility assumed to use "significant" water usage, hence 5,000 gpd/acre unit flow rate.
- Escondido Research Technology Center (ERTC) hospital (Phase 3) completed in 2008
- ERTC hospital (Phase 4) completed in 2010
- ERTC Stone Brewery expansion on-line in 2008 (additional 40,000 gpd)

- ERTC vacant lots sold, built and occupied by 2007

Table 2-2. Land Development Flows

ID	Year	Residential	Commercial	Industrial	Total Flow (mgd)
1	2007	26	8	3	0.431
2	2008	18	5	3	0.158
3	2009	27	6	1	0.283
4	2010	2	0	2	0.042

Flow Projections

The basis of this analysis is to estimate the times when the average annual daily flows at the HARRF reach the current rated capacity of 18.0 mgd and the build-out flow of 27.5 mgd. The projected flows were derived by linearly extrapolating both historical flows (from 2000 to 2005) and estimated "development" flows (from 2006 to 2008). Note, the flows calculated for 2009 and 2010 were eliminated from the analysis as these under-estimated flows skewed the projection, delaying the years at which 18.0 and 27.5-mgd capacities are reached.

Figure 2-1 depicts the projected flow relationship along with the estimated years when 18.0 and 27.5 mgd flows are reached. The chart also displays a projected sewer connection trend-line extrapolated from new sewer connections added from 2000 to 2005. Although the projected sewer connection trend is "flatter", the overall trend is comparable with the flow projection trend-line.

The average annual flow at the HARRF is projected to increase on average by 0.352 mgd per year resulting in the following events:

- Current rated capacity of 18.0 mgd will be reached in 2014.
- Master plan build-out (ultimate) flow of 27.5 mgd will be reached in 2041.

The design flows at build-out for areas served by the HARRF are presented in Table 2-3.

Table 2-3. Average Annual Daily Flow Projected at Build-out at the HARRF[(a)]

Item	Value
Average Annual Daily Flow (mgd)	
City of Escondido[(a)]	22.2
Rancho Bernardo[(b)]	5.3
TOTAL	**27.5**
Peak Hourly Wet Weather Flow (mgd)	
City of Escondido[(c)]	44.4
Rancho Bernardo[(d)]	9.0
TOTAL	**53.4**

Notes:
(a) Source: November 2005 Wastewater Collection System Master Plan Update for the City of Escondido
(b) Dry weather flow capacity contractually purchased by the City of San Diego
(c) Derived using a peaking factor of 2.0 deemed appropriate based on available data.
(d) Peak capacity of Pump Station No. 77 after latest improvements

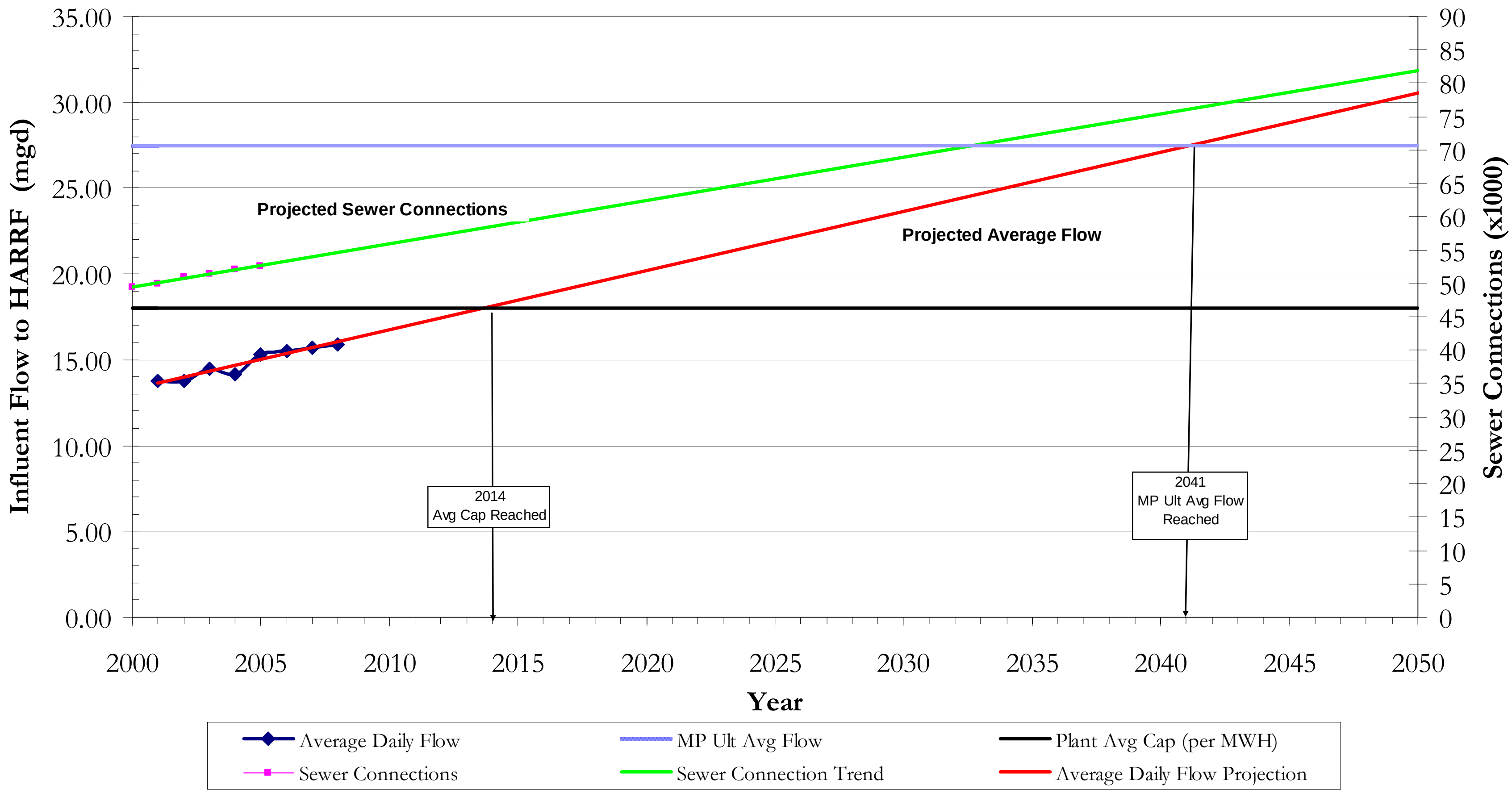


Figure 2-1. Influent Flow Projection Graph

3.0 EXISTING TREATMENT AND DISPOSAL FACILITIES CAPACITY EVALUATION

Presented in this section is a summary of the results of the capacity assessment performed for the HARRF, the Escondido Land Outfall, and the San Elijo Ocean Outfall. Detailed discussions are provided in Appendices A to J.

3.1 Existing Treatment Capacity at the HARRF

The capacity of each unit process was determined based on the lesser of the hydraulic capacity and the process capacity. The hydraulic capacity of each unit process represents the maximum flow rate that can be handled without overflowing tanks or channels and without flooding outlet weirs where applicable (e.g., primary clarifiers, mixed liquor splitter box, secondary clarifiers). Typically, hydraulic capacity is applicable for liquid treatment processes only. The capacity of each unit process represents the flow rate that can be treated to produce acceptable effluent quality. The process capacity of solids-handling facilities is based on maximum sludge production at a given AADF.

This approach determined the ability of the existing HARRF treatment facilities to treat the range of flows and loadings corresponding to the permitted average daily flow of 18.0 mgd and to comply with effluent discharge limits and recycled water regulations. Additional process units and/or changes in operating strategy needed to provide the permitted capacity were identified.

Capacity assessment of bar screens, grit chambers, and solids-processing units was performed based on AADF. For primary clarifiers and activated sludge, average dry weather flow (ADWF) was used, and all flow and loading peaking factors were based on ADWF. ADWF was determined by taking the average influent flow from August 1 to October 31 for six years (from 2000 to 2005). Note that because Escondido is located in an arid environment with limited rainfall, there is less than a 5 percent difference between average annual flow and average dry weather flow.

3.1.1 Bar Screens

Debris removal is provided by two 4.5-foot-wide Parkson Aqua Guard® mechanical bar screens with 6 mm nominal openings. A 3.5-foot-wide manual bar screen with 2-inch openings is provided for redundancy during peak flows if one of the mechanical bar screens is out of service. The bar screens remove large objects and other debris that may otherwise clog the raw sewage influent pumps and/or impact performance of downstream processes. Each mechanical bar screen is equipped with a screenings washer and compactor. The washed and compacted screenings are collected in a common bin located in a ventilated enclosure to contain odors.

According to the manufacturer's representative, each bar screen system is rated for 22.0 mgd capacity (*June 5, 2006 e-mail from Joe Nagel of Parkson*), based on a downstream water elevation of 4.1 feet at peak flow. In the same communication string, it was mentioned that the capacity may be as high as 28.0 mgd if the downstream water elevation is increased to 4.5 feet.

The approach velocity should be a minimum of 1.0 feet per second (fps) to avoid solids deposition within the approach channel, and a maximum of 4.0 fps to avoid pass-through of debris (*Design of*

Municipal Wastewater Treatment Plants, 4th Edition, Water Environment Federation, 1998; *Wastewater Engineering, Treatment, Disposal, Reuse*, 3rd Edition, Metcalf & Eddy Inc., 1991). Manufacturer-quoted capacities noted above result in velocities between 2.0 to 3.0 fps, depending on the upstream water surface elevation. Therefore, the two bar screens installed should be capable of handling the current rated flow of 18.0 mgd average and 36.0 mgd peak, and can continue to be effective at the build-out peak wet weather flow of 48.2 mgd (44.4 mgd or raw influent plus 3.8 mgd of recycle flows).

3.1.2 Influent Pumping Station

The hydraulic capacity of the influent pumping station (IPS) was evaluated briefly for this study. The existing IPS consists of a bifurcated wet well with three single-stage vertical centrifugal pumps installed in each wet well and a 24-inch by 36-inch cast iron sluice gate separating the two halves. Each pump installed in one wet well has a matching pump in the other wet well. This arrangement and the ability to isolate the wet wells with the gate were envisioned to allow cleaning of one wet well while the other remained active. However, the HARRF staff has indicated that the gate is not operable and the wet wells cannot be isolated.

The pumping station was designed with six pump bays, but only four pumps were installed initially – two in each wet well. In the 1980s, the largest pair of pumps were added. The pumps, drives, and motors were not changed, but some impellers were replaced with larger diameter impellers. The current IPS design criteria are summarized in Table 3-1 and the pump arrangement is shown on Figure 3-1 (source: HARRF-staff supplied information on May 31, 2006 and the 1981 *Hale Avenue WWTP Expansion* drawings [Sheet G-4]).

Table 3-1. HARRF Influent Pumping Station Design Criteria

Item	Unit	Pair 1	Pair 2		Pair 3
Pump Number	-	1 & 6	2 & 5		3 & 4
Manufacturer	-	Fairbanks Morse	Fairbanks Morse		Allis-Chalmers
Type	-	Vertical centrifugal angleflow	Vertical centrifugal angleflow		Vertical centrifugal mixed flow
Drive	-	Variable	Constant		Variable
Capacity, each	gpm	5060	4600		9000
Total dynamic head	feet	30	30		40
Maximum speed	RPM	855	875		880
Motor horsepower	hp	50	50		125
Impeller diameter	inch	15.40	14.9375		17.75
Operating strategy			Lead	Lag	
▪ Pump on (water depth)	inch	60	94	102	70
▪ Pump off (water depth)	inch	41	60	84	52

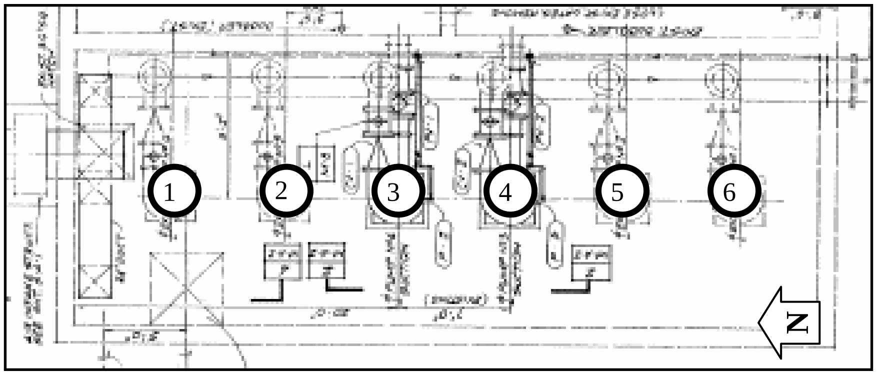


Figure 3-1. HARRF Influent Pump Station
Pump Arrangement

The *Final Letter Report for Capacity Rerating of the Hale Avenue Resource Recovery Facility (HARRF)* by MWH (hereto forth called the 2004 Capacity Rerating Study) stated that the IPS was rated for a peak capacity of 43.5 mgd with one pump out of service. Pump performance curves were not available for the smaller pumps, so the pumping station capacity could not be evaluated for this study. Verification of this capacity rating requires generating a system head curve and all pump curves to be available. Unfortunately, the study team did not have the pump curves for the smaller pumps and pressure sensing devices were not available, so a rigorous capacity evaluation of the pumps could not be performed. Instead, the following observations are provided (note that recommendations to improve performance and reliability are provided in Section 4):

- The discharge IPS force main is a combination of 30-inch and 36-inch-diameter pipes, transitioning from 30-inch to 36-inch about 280 feet downstream (Station 3+59.35 per Sheet C-14 of Phase 2 Drawings) from the IPS.

- Given the age of the existing equipment, a comprehensive evaluation of the current physical condition and remaining useful life of the equipment (including gates, operators, valves, etc.) must be conducted to determine the need for upgrades in system components or replacement of equipment.

3.1.3 Grit Removal

The grit removal system removes sand, gravel, and fine inorganic particles that can accumulate in downstream process units or cause excessive and premature wear in conveyance equipment. The installed system consists of two 24-foot-diameter Schloss forced vortex grit collectors (type CTP Grit Collector) with a 10-hp paddle mixer each, four Wemco horizontal recessed impeller grit pumps (two for each grit collectors) with 15-hp motors, two Schloss grit cyclone separators and classifiers, and two self-dump hoppers.

The reported capacity of the grit chambers differs according to the source. The 1999 Phase 2 Treatment Upgrades and Reclamation Facilities contract drawings indicate a peak flow and average flow capacity of 29.0 and 14.5 mgd, respectively, for each grit collector. The subsequent 2004 Capacity Rerating Study Letter Report rates each grit collector at 21.0 mgd average flow. The manufacturer recently stated the following capacity and performance information (*Telephone conversation between Brown and Caldwell and Schloss Engineered Equipment, Inc. on June 2, 2006*):

- Peak capacity of one 24-foot-diameter unit is 70 mgd peak flow.
- Grit collector is designed to provide the following particle removal efficiencies at the rated peak capacity, assuming a sand particle with a specific gravity of 2.65:
  - ✓ 95 percent of 50 mesh size
  - ✓ 85 percent of 70 mesh size
  - ✓ 65 percent of 100 mesh size

Based on past experience, Brown and Caldwell has found that the reliable capacity of this type of grit removal system is typically 50 percent of the manufacturer's stated capacity. Short periods of grit carryover will not have a significant impact on downstream unit processes, but continuous grit carryover during average flow conditions due to an undersized system can cause primary sludge pump wear, grit accumulation in aeration tanks, and/or grit accumulation in digesters. Although peak flow periods may have a short duration, they can represent a significant peak grit load due to "first flush" conditions. Other options are available for grit capture (e.g., DAFT bottom sludge degritting with co-thickening of primary and waste activated sludge (WAS), primary sludge degritting), but they should not be used to make up for an undersized raw sewage grit removal system.

We find that the hoppers are typically undersized, causing bridging of grit particles if the pump is not sized to pump the maximum expected rate of inflow of grit at less than about 1 percent solids concentration. There are improvements to the typical design that can be implemented to ensure that clumping of grit particles do not occur. Many designers, including Brown and Caldwell, include an air scouring system to fluidize the grit particles prior to pumping. The current system does not have this provision, but includes water agitation of the grit hopper. It is Brown and Caldwell's opinion that water agitation is not as effective as air scour for this purpose.

In summary, the currently configured grit removal system does not have sufficient capacity for build-out conditions, but it may have the necessary capacity to treat 18.0 mgd. The reliable capacity is likely between 14.5 and 21.0 mgd average flow; additional testing and verification is needed. In addition, implementing an air scour system will likely ensure that the capacity is at the upper end of the range.

3.1.4 Primary Treatment

On-site stress testing of the primary clarifiers was performed at surface overflow rates (SORs) ranging from 800 to 1,442 gallons per day per square foot (gpd/ft^2). This stress testing was achieved by taking tanks out of service. Field observations also showed that the flow split among the clarifiers is unequal, with more flow going to primary clarifier 4 (the eastern-most clarifier). In spite of the flow imbalance, the clarifiers demonstrated similar total suspended solids (TSS) and chemical oxygen demand (COD) removal with all clarifiers in service. At the highest SOR values, the water level in the effluent launders increased, as expected, but the effluent weirs were not submerged.

The stress testing showed that the HARRF primary clarifiers still remove 46 percent of the influent solids and 26 percent of the influent 5-day carbonaceous biochemical oxygen demand ($cBOD_5$) at a SOR of 1,400 gpd/ft^2. In addition, the maximum possible suspended solids removal is achieved under average flow conditions, corresponding to a SOR of between 800 and 849 gpd/ft^2. "Maximum possible suspended solids removal" refers to nearly 100-percent removal of primary influent settleable solids. A significant portion of primary influent suspended solids are not settleable and will pass through the clarifier with the primary effluent. Historical data (2000-2005) were evaluated to determine settling constants used to predict existing primary clarifier effluent quality. Settling constants were in good agreement with constants determined from on-site stress testing. Performance curves for COD and TSS removal as a function of influent flow were generated for use with the secondary process capacity assessment.

3.1.5 Secondary Treatment

The secondary treatment capacity was evaluated by using the suspended biological growth simulator, BioWin™ 2.2, and state point analysis (SPA) to model secondary clarifier performance A two-week wastewater characterization study was performed to determine wastewater characteristics and other inputs to calibrate the activated sludge simulator. The characterization study results showed that there is unequal flow distribution among aeration basins. We attribute this imbalance to unequal return activated sludge (RAS) flow and/or unequal primary effluent flow to an individual tank, and that the reported RAS flows are incorrect. In addition, the HARRF secondary effluent has high nitrite concentrations due to incomplete nitrification that is likely due to one or more of the following: (1) excessive RAS chlorination, (2) high ammonia loads in solids recycle streams, and (3) low dissolved oxygen (DO) concentrations in the aeration basins.

The activated sludge process capacity was evaluated using several criteria. First, BioWin™ 2.2 and SPA were used to determine whether the clarifier was underloaded, in thickening failure, or in clarification failure at the anticipated range of primary effluent flows and loads associated with a given average raw sewage flow. Thickening failure and clarification failure indicate insufficient process capacity and either the average flow is decreased, an additional primary clarifier(s) is added to reduce aeration influent loads, an additional aeration tank(s) is added, and/or an additional secondary clarifier(s) is added so the clarifier is underloaded. This underloaded condition minimizes the clarifier sludge blanket depth and the secondary effluent suspended solids concentration.

Second, BioWin™ 2.2 was used to determine process oxygen demands over the range of anticipated primary effluent flows and loads associated with a given average raw sewage flow. The simulated process oxygen demands were compared to the reliable capacity of the existing aeration panel diffusers and aeration air blowers. Off-gas testing was conducted in the HARRF aeration tanks to determine site-specific oxygen transfer coefficients for an accurate evaluation. The off-gas testing showed that the current value of αF (ratio of oxygen transfer rate in dirty [process] water to oxygen transfer rate in clean water) is 0.32 (see Appendix M for Off-Gas Testing Report).

Third, BioWin™ 2.2 was used to simulate secondary effluent quality over the range of anticipated flows and loads to ensure discharge permit compliance.

The secondary process capacity was based on parameters determined from historical performance data between 2000 and 2005. The average solids residence time of 2.75 days determined from the historical data was used to simulate activated sludge performance. It should be pointed out that in 2005 the average solids residence time was 3.5 days. The mixed liquor settleability (as measured by sludge volume index (SVI)) was used to evaluate secondary clarifier performance. A lower sludge volume index corresponds to a better settling sludge. Historical SVI values ranged between 65 and 600. The 90th percentile of this parameter is used as a measure of the reliable sludge settleability that can be anticipated under current activated sludge operating conditions. This value is calculated to be 203 mL/g. Use of the 90th percentile value means that current operating conditions will produce a sludge volume index of 203 mL/g or less approximately 90 percent of the time. Extraordinary control measures such as RAS chlorination or coagulant addition must be used during the time when the 90th percentile is exceeded. In other words, the secondary process will run without these extra measures approximately 330 days of the year. The other 35 days will require additional measures to keep the process within desired parameters.

The secondary process capacity, under current operating conditions as explained above, is calculated to be 14.8 mgd ADWF. The corresponding PWWF capacity is 29.3 mgd. The capacity of the HARRF is limited by the sludge settleability which limits the solids loading rate to the secondary clarifiers. In addition, the existing aeration system cannot produce air flow rates needed to meet process oxygen demands above 15.0 mgd at the measured value of αF. The aeration system is limited by the maximum airflow that can be tolerated by the existing fine-bubble aeration panels; the blowers have sufficient capacity.

3.1.6 Tertiary Treatment

The HARRF tertiary treatment facilities include:

- Three filter influent pumps for a reliable pumping capacity (2 duty/1 standby) of approximately 10.0 mgd
- An in-line pumped flash mix system for chemical addition
- Two 2-stage flocculation tanks, each with two mechanical flocculators
- Eight 200 ft^2 continuous backwash granular media filters
- A UV system with two channels, each with five banks of low pressure low intensity UV lamps (4 duty/1 standby)
- Chlorine disinfection with one multiple-pass contact tank (constructed using the three abandoned "squircle" secondary clarifiers).
- Associated chemical storage and feed systems.

All facilities except the chlorine disinfection system were constructed as part of the 1999 Phase 2 Treatment Upgrades and Water Reclamation Facilities project. The chlorine disinfection facilities were constructed as part of the 2005 Chlorine Contact Tank Design/Build Project. The tertiary treatment facilities are not needed for ocean outfall discharge permit compliance, but are used to treat a portion of the HARRF secondary effluent for use as reclaimed water.

The granular media filters were designed for a maximum influent flow of 10.0 mgd with loading rate of 4.34 gpm/ft^2 based on all the filters in service (*Parkson Corporation, Specification Section 11422*). The corresponding maximum hydraulic loading rate with one unit out of service is 5.0 gpm/ft^2 as allowed by California Department of Health Services (DHS) for recycled water treatment. A portion of the filter effluent is recycled continuously to the IPS as waste washwater. At the design waste washwater flow rate of approximately 80 gpm per filter, the total waste washwater flow is expected to be 0.8 mgd, which results in a net filtered water production of approximately 9.0 mgd. Under these conditions, the filters are specified to produce an effluent containing an average 5 mg/L TSS (2 nephlometric turbidity units [NTU]) for a typical secondary effluent containing an average of 20-30 mg/L TSS (10 NTU).

The filters, however, have never been able to operate at their rated hydraulic loading of 5.0 gpm/ft^2, and comply with the 2 NTU filter effluent limit for "disinfected tertiary" quality recycled water. Therefore, the full capacity of these filters has not been achieved despite efforts by plant staff, the manufacturer, and various chemical suppliers. In the recent past, the tertiary treatment processes at the HARRF have required excessive levels of coagulant (approximately 60 to 80 mg/L polyaluminum chloride (PACl) and approximately 3 mg/L polymer) to produce an effluent suitable for filtration and eventual use as reclaimed water. In addition, the current backwash rate is approximately 20 percent. Higher reject water quantity reduces the net filtered water production. Therefore, the maximum possible net filtered water production is reduced to 8.0 mgd at the maximum filter loading rate of 10.0 mgd.

It has been determined that the recent poor performance of the tertiary processes corresponded to high secondary effluent nitrite concentrations caused by incomplete nitrification and low mixed liquor suspended solids (MLSS) concentrations (less than 1.0 g/L) due to the relatively low SRT. At Brown and Caldwell's recommendation, the HARRF staff increased the operating SRT from approximately 2.75 days to approximately 5 days as an interim measure to improve tertiary system performance. This process change resulted in lower turbidity levels in the filter effluent and reduced chemical requirements. The improvement in tertiary performance is attributed to a completely nitrified effluent resulting from the longer SRT and due to higher MLSS concentrations resulting from the higher mixed liquor inventories in the aeration basins. Typically, activated sludge settles as a blanket resulting in the removal of smaller particles due to a "filtering" action of the settling biomass. Operating at MLSS concentrations less than 1.0 g/L results in a diluted sludge where benefits of the "filtering" action are lost and effluent can have higher levels of colloidal material.

The UV disinfection system was designed based on the 1993 NWRI guidelines (*UV Disinfection Guidelines for Wastewater Reclamation in California and UV Disinfection Research Needs*, National Water Research Institute, September 1993), which had been adopted by DHS for recycled water disinfection systems. These guidelines were superseded in 2000 by the 2000 NWRI/AWWRF guidelines (National Water Research Institute/American Water Works Research Foundation, *Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse*, December 2000). The 2000 NWRI/AWWRF guidelines required that the capacity of the HARRF UV disinfection system be tested at full-scale operating conditions before DHS would validate the recycled water treatment facilities. A series of three commissioning tests were conducted in early 2003 to validate the delivered UV dose. DHS developed an interim operations plan in the fall of 2003 that stated a maximum UV disinfection system capacity of 4.0 mgd. The San Diego Water Board approved the 4.0 mgd disinfection capacity based on the DHS interim operations plan.

The derated UV disinfection capacity was mitigated by conversion of the old “squircle” clarifiers to a chlorine contact tank with a capacity of approximately 10.0 mgd. While the chlorine contact tank has sufficient treatment capacity, the operations costs are relatively high because the high nitrite nitrogen concentration exerts a significant chlorine demand (approximately 10 mg chlorine/mg nitrite nitrogen) that must be met before recycled water disinfection can be achieved. Chlorine residual of 2 to 3 mg/L is required in the recycle water distribution system to prevent biological growth.

3.1.7 Solids Treatment

Evaluation of the capacity of unit processes and components for solids treatment at the HARRF is based on plans, operations and maintenance manuals, and previous reports and studies provided by the City. Additional information was collected during a site visit on March 2, 2006. Brown and Caldwell design guidelines and applicable regulatory requirements were used to evaluate the capacity of the solids treatment facilities. Table 3-2 summarizes design criteria used for this evaluation.

Findings and conclusions of the evaluation are summarized below. Capacity ratings for each solids processing component are reported in Table 3-3.

Dissolved Air Flotation Thickening. The capacity of the thickening facilities is inadequate to handle the range of projected solids loadings at 18.0 mgd average flow with one of the two dissolved air flotation thickeners (DAFTs) out of service.

Anaerobic Digestion. The capacity of the sludge stabilization facilities is inadequate to handle the range of projected thickened sludge loadings at 18.0 mgd average flow. This finding is based on the minimum SRT criterion of 20 days with one digester out of service during average loading conditions and historical thickened sludge concentration. The 20-day HRT at average with one digester out of service provides sufficient capacity to meet the 15-day HRT required by EPA 503 regulations for Class B biosolids in the event a peak two week flow occurs while one digester is out of service. Adequate capacity to handle thickened sludge loadings with the existing facilities can be provided by increasing the thickened sludge concentration, which will decrease the thickened sludge flow and increase the SRT. A minimum thickened sludge concentration of approximately 6.8 percent for the blended thickened waste activated sludge (TWAS) and primary sludge will meet the minimum SRT criterion at average loading conditions at 18.0 mgd average flow. The digesters currently meet vector attraction requirements by providing greater than 38 percent volatile solids reduction.

Table 3-2. Process Capacity Evaluation and Design Criteria

Process Unit	Criterion	Units	Value	Source
DAFT	Solids loading rates[a] ▪ Average, one unit out of service ▪ Peak, all units in service	lb/d-ft²	15 36	Brown and Caldwell Design Guideline
	Air to solids ratio	lb air/lb TSS	0.03	
	Minimum pressurized tank liquid retention time	min.	0.75	
	Saturation constant	mg/L	100.7	Henry's Law, air at 75°F
Anaerobic digestion	Vector attraction reduction (VAR)	volatile solids reduction (VSR)	38 percent	EPA 40 CFR 503 Part B
	Process to Significantly Reduce Pathogens (PSRP) for Class B biosolids Solids retention time (SRT)[b] at 35 to 55°C ▪ Average, largest unit out of service ▪ Peak 2-week, all units in service	days	20 15	EPA 40 CFR 503 Part B Brown and Caldwell Design guideline for average conditions to provide better VSR
	Active digester volume is based on number of digesters that are heated and mixed	---	---	EPA 40 CFR 503 Part B
Centrifuge dewatering	Hydraulic loading	gpm	75 to 150	Vendor information
	Solids loading	lb/hr	550 to 2800	Vendor information

Notes:
(a) Previous capacity assessment prepared by MWH used a solids loading rate of 45 lb/sf. Brown and Caldwell believes this loading rate could not be achieved without adding a significant amount of polymer.
(b) SRT based on active digester volume (i.e., digesters that are heated and mixed)

Centrifuge Dewatering. The capacity of the dewatering facilities is adequate to handle the range of projected stabilized sludge loadings at 21.2 mgd average flow. Additional capacity may be provided by running the centrifuges more than 12 hours per day. Emergency capacity may be provided by operating the third centrifuge and increasing the pressure capacity of the digested sludge transfer pumps or pigging the line between the secondary digester and the dewatering feed tank to reduce transfer pump discharge pressure.

Table 3-3. Existing Solids Processing Capacity at the HARRF

Evaluation Criterion	Treatment Capacity at Equivalent Average Plant Influent Flow (mgd)
DAF Thickening	
Average Solids Loading - one unit out of service	14
Average Solids Loading – all units in service	28
Average Saturation System Capacity	14
Anaerobic Digestion	
Average Solids Retention Time - one unit out of service (20-day minimum)	12
Average Solids Retention Time – all units in service (20-day minimum)	18
Vector Attraction Reduction (38% VSR)	Meets Requirements
Centrifuge Dewatering	
Hydraulic Loading – one unit out of service, 7 day/24 hr per day operation (150 gpm each)	21.2

3.1.8 Comparison of Current Capacity Assessment with the 2004 Capacity Rerating Study

The HARRF is currently rated at an average flow of 18.0 mgd based on the *2004 Capacity Rerating Study Letter Report*. The 2004 study was a desktop study using published design guidelines from textbooks and published process design manuals, was based on continued operation of the activated sludge process, and was based on one year of operations and performance data from February 2003 to January 2004. The *2004 Capacity Rerating Study* found that the aeration basins and secondary clarifiers were the limiting processes at the HARRF and were both rated for an average flow of 18.0 mgd (assuming a peak wet weather peaking factor of 2.0).

A similar finding was developed in the current capacity assessment – that the secondary treatment system limited the capacity of the HARRF. In addition, the existing solids treatment process has insufficient capacity to handle an average daily flow of 18.0 mgd. The current capacity assessment determined the maximum HARRF treatment capacity to be 14.8 mgd average flow based on limitations of the secondary treatment system.

While the capacity rating approaches in each study were fundamentally different, there are several assumptions in the *2004 Capacity Rerating Study* that would bring the two results closer together. The limiting secondary clarifier solids loading rate in the *2004 Capacity Rerating Study* was assumed to be 2.0 lb/ft^2-hr based on a textbook value. The limiting secondary clarifier solids loading rate in this study, 1.22 lb/ft^2-hr, was determined from historical SVI data. The 90th percentile of the SVI data was used as measure of reliable sludge settleability to evaluate secondary clarifier performance, so that SVI control (e.g., RAS chlorination, polymer addition) would be needed approximately 30 days per year. The difference in limiting secondary clarifier solids loading rate represents the majority of the difference in rated plant capacity.

Note that the reliable limiting secondary clarifier solids loading rate in this study was affected strongly by historical plant operations and performance. For example, SVI values in 2005 were significantly lower and were less variable than the historical SVI values from 2000 through 2004 that were used above. The 90^{th} percentile SVI value for 2005 was 135 mL/g, which corresponds to a limiting solids loading rate of 1.71 lb/ft^2-hr. The process capacity would be 18.4 mgd average flow – on a clarifier solids loading rate basis – if the reliable SVI value was reduced to 135 mL/g. This potential for a higher process capacity was demonstrated during wet weather events in January 2005 when the HARRF handled more than 14.8 mgd average flow at SVI values ranging from 82 to 177 mL/g.

Additionally, the DAFT solids loading rate used in the *2004 Capacity Rerating Study* was based on a textbook value. Brown and Caldwell believe that the textbook value can only be achieved with significant polymer addition. The value used in this study is based upon our experience with full-scale DAFT performance and can be achieved without polymer addition.

The total anaerobic digester volume used in the *2004 Capacity Rerating Study* included the volume of the 55-foot-diameter digested sludge holding tank in evaluating digester capacity. The digested sludge holding tank is not heated or mixed; therefore, the total anaerobic digester volume used in this study did not include the holding tank volume.

3.2 Capacity of Existing Disposal Systems

The capacity of existing storage and disposal facilities was evaluated. The process, findings, and results of the study are presented in Appendices I, J, and K. A summary is provided below.

3.2.1 Equalization

The equalization and storage facilities currently available and under construction at HARRF and their respective capacities are as follows:

- Existing secondary effluent equalization basin – 2 million gallons (MG)
- New secondary effluent equalization basin– 2 MG (under construction)
- Existing reclaimed water storage tank – 2 MG (Leslie Lane)
- New reclaimed water storage tank – 1 MG (under construction)

3.2.2 Escondido Land Outfall (ELO)

The hydraulic capacity of the existing land outfall is summarized as follows:

- Land outfall (gravity section) – 23.7 mgd
- Land outfall (pressurized section) – 21.4 mgd

The capacities of the gravity and pressurized sections of the land outfall result from different hydraulic behavior. The model developed for this study indicates that the gravity section is limited by siphons that restrict flow. At 23.7 mgd, the model predicts that a spill may occur at the inlet and outlet manholes of certain siphons. Further analysis and model tests demonstrate that the capacity of the ELO could be increased to 25.2 mgd if the siphon inlet/outlet manholes are sealed. Regarding the pressurized section, capacity improvements and spill reductions can be achieved by sealing all manholes downstream of Manhole 69. However, sealing these manholes may be challenging since most are difficult to access.

3.2.3 San Elijo Ocean Outfall (SEOO)

The hydraulic model developed for this study indicates that the existing SEOO has a hydraulic capacity of about 25.8 mgd. This capacity is consistent with the rating of 25.5 mgd reported by Tetra Tech, Inc., in 2001. The result of the 2001 evaluation was in fact included in the current NPDES permit (Order No. R9-2005-0101, NPDES No. CA0117981, June 8, 2005) and the Fact Sheet (Attachment F of the permit), noting that the total allowable monthly average effluent discharge from the HARRF and from the SEWRF cannot exceed 23.25 mgd – 18.0 mgd from the HARRF and 5.25 mgd from SEWRF.

3.2.4 Intermittent Live-stream Discharge

Intermittent live-stream discharge provides a mechanism for disposing of reclaimed water into the local creek in compliance with the stream flow conditions defined in the permit. The benefit of conducting live-stream discharge is the reduction of outfall capacity and additional secondary effluent equalization/storage capacity.

The City is allowed to discharge intermittently to the Escondido Creek up to 9.0 mgd of tertiary treated effluent as long as certain provisions have been met (see Section 2 for details). As noted previously, the HARRF is currently limited to 4.0 mgd of tertiary effluent production due to the limitation associated with the UV disinfection system (April 21, 2004 letter from the San Diego Regional Water Quality Control Board entitled *"Authorization to Discharge Title 22 Recylced Water, Order No. 93-70"*). A chlorination system capable of disinfecting 10.0 mgd of filtered effluent is currently in place. But its main purpose is to reliably produce disinfected recycled water for reuse. Consequently, it does not include a dechlorination system. Unless the chlorination system is retrofitted, the existing intermittent live-stream discharge system is limited to 4.0 mgd.

4.0 RECOMMENDED IMPROVEMENTS AT THE HARRF

In this section, improvements required to elevate the treatment and hydraulic capacity at the HARRF to the current rated average annual daily flow capacity of 18.0 mgd and the expected build-out flow of 27.5 mgd are presented.

4.1 Recommended Near-Term Improvements at the HARRF

Implementation of permanent improvements to make certain that the HARRF maintains adequate capacity to treat incoming flows can take several years to plan, design and construct. Meanwhile, interim, near-term improvements to the secondary and solids treatment system can be implemented to make certain that the plant can reliably treat an average daily flow of 18.0 mgd. These improvements include reducing the solids inventory in the aeration basins, improving sludge settleability, increasing oxygen transfer capacity, increasing the efficiency and hydraulic loading to the dissolved air flotation thickeners, and increasing the hydraulic retention time in the digesters. Recommended near-term improvements are further discussed below.

4.1.1 Bar Screen

The bar screen meets capacity and no improvements are needed at this time.

4.1.2 Influent Pump Station (IPS)

The existing IPS has sufficient capacity to convey the expected peak wet weather flow of 29.2 mgd ([average HARRF raw wastewater influent flow of 18.0 mgd – Rancho Bernardo contracted average daily flow of 5.3 mgd] * Peaking Factor of 2 + 3.8 mgd recycle flows). However, given the age of the existing equipment, a comprehensive evaluation of the remaining useful life of the equipment (e.g., gates, operators, valves) must be conducted to determine if any system components must be upgraded or replaced.

4.1.3 Primary Treatment

For the primary clarifiers, it is recommended that chemically enhanced primary treatment (CEPT) is implemented to reduce organic and solids loading to the secondary processes. For CEPT, chemical coagulants are added upstream of the primary clarifiers. By reducing the organic load to the secondary system, the mixed liquor suspended solids (MLSS) concentration is lowered, which reduces the solids loading rate to the secondary clarifiers. In addition, the aeration requirements are reduced due to the reduction in organic loading. Implementation of CEPT will allow the HARRF staff to take one primary clarifier out of service to perform preventative maintenance.

CEPT for the HARRF would require chemical dosing with ferric chloride (40 mg/L ferric chloride dose was assumed representative). Addition of a polymer will act to strengthen flocs and may provide better performance at doses ranging from 0.1 to 0.5 mg/L. For four primary clarifiers, TSS removal is estimated to be 72 percent with CEPT. The application of CEPT for the HARRF will result in elevated primary sludge loading to the solids handling processes due to the higher removal efficiency and chemical precipitates.

4.1.4 Secondary Treatment

The secondary treatment facilities (aeration basins, secondary clarifiers, and supporting systems) at the HARRF were determined to be limited by the existing aeration system and the sludge settleability. The 90th percentile sludge volume index (SVI) value used to rate the secondary clarifiers was determined to be 203 mL/g based on historic data (2000-2005). In developing a near-term solution for the HARRF, it was assumed that the 90th percentile SVI could be reduced to 150 mL/g with an improved chemical control strategy (RAS chlorination and polymer addition). Chemical addition for SVI control will require added operator attention involving frequent microscope analyses of mixed liquor and SVI analyses to prevent overdosing of chemicals. For instance, if SVI values are low (i.e., less than 80), additional RAS chlorination may break up floc and reduce effluent quality.

The existing blowers were found adequate to meet process oxygen demands over the range of anticipated primary effluent flows and loads associated with the current 15.0 mgd average flow. However, they don't have the capacity for 18.0 mgd flow under the existing operating conditions. The existing diffuser panels currently limit the amount of air added to the basin. Implementing CEPT will reduce aeration requirements. The SRT can also be reduced to 2 days to inhibit nitrification. However, the aeration capacity is still limited. Capacity can be increased by supplementing the existing diffuser capacity. This can be done by temporarily adding blowers and diffusers to the basins, adding high purity oxygen or air to the incoming return activated sludge stream, or adding surface aerators. The supplemental aeration would be used to increase dissolved oxygen concentrations in the aeration basins only when needed (such as during peak flow or load periods). Supplemental aeration in the aeration basins with high purity oxygen is considered feasible and included in the cost estimates.

As part of the near-term improvements, the existing RAS flow distribution should be improved. Improving the RAS distribution will equally distribute solids between aeration basins and provide a more balanced operation. In addition, the RAS flow meters should be recalibrated for improved process control.

4.1.5 Tertiary Treatment

Currently, the tertiary filters have been shown to operate poorly on the secondary effluent currently generated at the HARRF. The poor performance is attributed to the low SRT in the secondary treatment process. With the recommended improvements for the interim solution, the effluent water quality from the secondary clarifiers should improve. An improvement in the secondary effluent quality is expected to improve the tertiary filter performance. In addition, operation of the secondary process at a SRT of 2.0 day will suppress nitrification and prevent formation of nitrite. By eliminating nitrite formation, the chlorine demand of the tertiary effluent will be reduced allowing for more stable operation.

4.1.6 Solids Processing

Recommendations to improve the performance of the thickening, digestion, and dewatering processes are summarized in Table 4-1. These recommended improvements are intended to bring the existing facilities to the best performance level.

Table 4-1. Recommended Solids System Process Performance Improvements

Process	Item	Recommended Improvements	Purpose
DAF Thickening	Polymer feed	Move polymer feed closer to discharge point of pressurized flow	Improves mixing efficiency by using turbulence of rising bubbles
		Modify center feed piping to accommodate new polymer discharge point	Necessary to implement change noted above
	Thickener effluent	Replace thickener overflow weir with submerged launder pipe	Provides cleaner water for recycle to pressurized flow system
		Provide control valve on thickener effluent line for level control in the DAFT	Controls liquid level to maximize drainage through float
	Saturation System	Replace pressurized flow pumps to meet necessary recycle flow for solids loading	Provides sufficient flow for air saturation
		Add second pressurization tank or increase operating level	Provides sufficient residence time for air to dissolve; reduces possibility of vortexing
		Add continuous vent to purge excess nitrogen	Increases gas absorption and improves stability
		Modify inlet and outlet piping to prevent vortexing and inlet pipe flooding	Vortexing can bring in undissolved air to DAFT discharge point, disturbing small bubbles being released and break up floc as it forms with rising bubbles

Table 4-1. Recommended Solids System Process Performance Improvements

Process	Item	Recommended Improvements	Purpose
Anaerobic Digesters	Digester Feed Sequencing	Feed primary and secondary solids simultaneously to all digesters on the same day	Stabilizes operation through more consistent solids feed; prevents gas production spikes
	Digester mixing	Verify lances and draft tubes are clear	Ensures system is operating as designed
		Verify draft tube mixing capacity provides 16 to 24 turnovers per day	Verifies mixing capacity is sufficient to prevent solids deposition, surface matting, dead zones, and hot spots; provides efficient contact of existing biomass with new food
		Provide dedicated compressors for Digesters Nos. 1 and 2	Needed to provided balanced operation to draft tube gas mixing systems
		Perform dye study	Confirms mixing efficiency in digesters, particularly for Digester No.1
Centrifuge Dewatering	Polymer application	Provide sludge samples to centrifuge and polymer suppliers	Verifies that the sludge character has not changed since centrifuges have been placed into service
		Perform polymer trials	Establishes whether a new polymer should be used
	Scale Control	Perform periodic acid cleaning of centrate pipes and/or use polyphosphate scale inhibitors	Maintains centrate system hydraulic capacity to prevent backups from occurring

Even with the improvements listed in Table 4.1, the DAFT and aerobic digesters may not have the 18.0 mgd treatment capacity with one unit out of service. Therefore, the following additional improvements are suggested.

- Direct a portion of the waste activated sludge to co-thicken with the primary sludge in the primary clarifiers. This is not recommended for day-to-day operation, but may be considered in an emergency if both dissolved air flotation thickeners were out of service.

- Consider recuperative thickening when taking a digester out of service in order to maintain the solids retention time required to produce Class B biosolids. Recuperative thickening is a process where a portion of the partially digested sludge is removed from the digester, thickened, and re-inserted into the digester to increase the solids retention time of the sludge. This process is typically used to meet Class A or B solids retention time requirements. Since taking a digester out of service is an infrequent activity lasting about 3 to 4 weeks at a time, centrifuges or gravity belt thickeners can be rented rather than construction of another digester. (It should be noted the solid retention time required under Class B regulations can be met at 18.0 mgd average daily plant influent flow when all existing digesters are in service).

4.2 Recommended Long-Term Improvements to Achieve 27.5 mgd Average Flow Capacity

Improvements required to enable the HARRF to treat up to the ADWF of 27.5 mgd and the PWWF flow of 53.4 mgd of raw wastewater (including flows from Rancho Bernardo) are presented in this section. In-plant recycle flows from the thickening, dewatering, and filtration processes totaling 3.8 mgd have been added to the raw wastewater flows during the analysis. These flows represent the build-out conditions for the HARRF.

4.2.1 Bar Screens

Two bar screens in operation are capable of treating the peak wet weather flow of 48.2 mgd expected at build-out. However, a third bar screen is recommended for flexibility and to eliminate the difficult task of manually raking the manual bar screen during peak conditions should one of the Parkson Aqua Guard® units be out of service. The existing manual bar screen can be converted to a mechanical bar screen identical to those currently installed. This third bar screen can have its own bin for storage of dewatered screenings. Foul air from the new enclosure can be directed to the same odor control system currently treating the foul air from the existing bin enclosure. Cost estimates presented in Section 6.0 include the conversion of the manual bar screen to a mechanical bar screen.

4.2.2 Influent Pump Station

Based on the latest assessment in the 2004 Capacity Rerating Study, the existing pump station has a peak capacity of 43.5 mgd, 4.7 mgd short of the ultimate (build-out) peak wet weather flow expected of 48.2 mgd. Some of the existing pumps can be upsized or over speed to make up the difference (e.g., the 9,000-gpm pumps [Pumps 3 and 4] could be operated at 10-12 percent higher than the current design speed of 880 RPM to provide additional capacity). A field torsiograph test should be conducted to identify torsional resonance issues and determine if the existing variable frequency drives (VFDs) can operate at speeds greater than 60 Hz. In addition, a lateral resonance study should be conducted to determine if the pump foundation, frame and motor supports and rotating system can withstand the dynamic forces resulting from operation at the higher speeds. Other recommendations include the following:

- The motor manufacturer should be contacted to determine if the motor design is adequate to handle the additional electrical current and voltage at the higher speed. Additionally, the VFD manufacturer must be consulted regarding the capacity of the existing drives and their ability to over speed the system. Other checks of the electrical system will be needed to determine if there is sufficient capacity to carry the additional load.

- Any increase in motor size or overspeeding may require the upgrade of feeders to the IPS. The HARRF staff has reported the following:

- ✓ *"..the cabling to the influent pump stations MCC is single run (3 phase) of "500 MCM" type XHHW. The branch circuit breaker is set at 300 amps. It appears we would need to increase the size of the MCC feeder if we make a large change in the horsepower rating of any pumps.*

- ✓ *The 125 hp pumps use a 125 KVA Toshiba 130-H2 drive which appears to [be] short of the 150 hp rating. At this time the drives are only eighty percent loaded."*

- The ultimate (build-out) peak wet weather flow rate to the IPS is 48.2 mgd, consisting of 44.4 mgd raw sewage and 3.8 mgd of in-plant recycle flows. At the build-out flow rate, the velocities in the 30-inch and 36-inch pipes are approximately 15 fps and 10.6 fps, respectively. To avoid significant erosion of the pipe walls and excessive frictional energy loss, prudent design practice limits the velocity to between 8 and 10 fps. Accordingly, the 30-inch pipe should be replaced with a larger pipe to reduce the maximum velocity. Based on the 2004 Capacity Rerating Study, the larger pipe size will reduce the total dynamic head (TDH) at the pump discharge and the existing motors, drives, and pump impellers should be adequate. However, a new system-head curve should be developed as part of a detailed pumping station analysis to confirm (1) the revised capacity of the existing pumping station with the larger pipe, and (2) if the existing motors and drives are adequate for continued service.

4.2.3 Grit Removal

An additional grit removal system will be needed to accommodate future flows. For this study, two alternatives of providing additional grit removal capacity are considered. One alternative is to add a third vortex grit system, similar in size and capabilities to the existing Schloss vortex grit system. A conveyor and a centralized grit storage bin unit will be installed to collect grit from the existing and new classifiers/washers system. Another alternative is to allow the existing grit chamber to treat the incoming flows and allow any uncaptured grit to settle out in the primary clarifiers. This is a common practice in many plants that do not have a grit removal process.

For separate sludge thickening option, the raw sludge could be degritted before going to the digesters. Degritting of primary sludge will require solids concentration of approximately 1 percent solids. Feeding a lower concentration primary sludge will require more digester volume. Therefore, using sludge degritting system for separate sludge thickening option is not feasible for the HARRF. It is assumed that a third vortex grit system will be added when separate sludge thickening option is considered.

For co-thickening, DAFT provides a convenient place to remove the grit contained in the primary sludge when both primary and secondary solids are co-thickened in the DAFT. Co-thickening in the DAFTs is considered to enhance the performance of the solids processing facilities. The DAFT process removes grit from the primary sludge in much the same way as an aerated grit chamber does. Air bubbles released as a part of the flotation process cling onto the particles with a lower specific gravity, allowing them to float to the surface. The more dense particles that do not float would settle out to the bottom of the DAFT where it could be removed as a part of the bottom sludge.

Typically BC designs DAFT thickened bottom sludge pumping systems to recirculate from 4 to 10 percent of the raw sludge flow back to the influent feed to the DAFT. To keep from recirculating grit contained in primary sludge, this flow is generally passed through a sludge degritting system. Eutek makes a vortex grit removal system that can be utilized in the sludge degritting process. The City of San Diego at its Metro Biosolids Center (MBC) has been successfully using a Eutek Tea Cup™ degritting system for over seven years to remove grit from raw sludge prior to the thickening centrifuges. The size required for degritting the DAFT bottom sludge from the co-thickening process would be need to be coordinated with the size of the bottom sludge pumps. Since the bottom sludge is removed on an intermittent basis there is some flexibility in adjusting the size of the bottom sludge pumps to the optimum flow rate for the Eutek Teacup degritting system.

4.2.4 Primary Treatment

It is recommended that an additional primary clarifier be constructed to provide redundancy; plant staff would like to have the flexibility to take one clarifier out of service during dry weather. However peak flows cannot be treated with five primary clarifiers unless modifications are made to remove the hydraulic bottleneck the hydraulic pinch point downstream of the primary effluent launders. The hydraulic capacity of the primary clarifiers can be increased with the following improvements:

- Increase the size of the influent well orifice of the secondary clarifiers
- Increase the number of gates at the aeration basin influent and effluent channel to a total of eight gates per basin (currently, there are four gates per basin)

Intermittent CEPT application was considered for several treatment alternatives discussed below.

4.2.5 Secondary Treatment

Fifteen secondary treatment alternatives were identified for consideration for the HARRF as shown in Table 4-2. After preliminary evaluation, the list was reduced to eight potential options that were further examined. Of the eight alternatives listed below, only Alternative 3 and 6 were considered viable alternatives for the HARRF, and a subsequent cost analysis was performed.

- Alternative 1 - Conventional Activated Sludge (CAS) w/ Nitrification
- Alternative 2 - Bioaugmentation Reaeration (BAR)
- Alternative 3 - High-Rate Activated Sludge (HRAS)
- Alternative 4 - Membrane Bioreactor (MBR)
- Alternative 5 - Flow Equalization
- Alternative 6 - Moving Bed Biological Reactor (MBBR)

- Alternative 7 - Sludge Reaeration Activated Sludge (SRAS)
- Alternative 8 - Biological Contact Process (BCP)

Alternative 1 involved converting the secondary treatment system to nitrifying activated sludge by operating at a 5.0-d SRT. In addition, CEPT would be used to reduce organic loading to the secondary system. As a result, two additional secondary clarifiers and a 50-percent increase in aeration tank volume was required. The additional aeration basins and secondary clarifiers made Alternative 1 not feasible.

Table 4-2.
Summary of Secondary Process Alternatives for the HARRF

Preliminary Options	Potential Options	Viable Alternatives
Conventional Activated Sludge (CAS) w/ Nitrification	Conventional Activated Sludge (CAS) w/ Nitrification	High-Rate Activated Sludge (HRAS)
Bioaugmentation Reaeration (BAR)	Bioaugmentation Reaeration (BAR)	Moving Bed Biological Reactor (MBBR)
High-Rate Activated Sludge (HRAS)	High-Rate Activated Sludge (HRAS)	
Membrane Bioreactor (MBR)	Membrane Bioreactor (MBR)	
Flow Equalization	Flow Equalization	
Moving Bed Biological Reactor (MBBR)	Moving Bed Biological Reactor (MBBR)	
Sludge Reaeration Activated Sludge (SRAS)	Sludge Reaeration Activated Sludge (SRAS)	
Biological Contact Process (BCP)	Biological Contact Process (BCP)	
High-Purity Oxygen Activated Sludge (HPOAS)		
Sequencing Batch Reactor (SBR)		
Trickling Filter/Activated Sludge (TF/AS)		
Trickling Filter/Solids Contact (TF/SC)		
Integrated Fixed-Film Activated Sludge (IFAS)		
Biological Aerated Filter (BAF)		
Step Feed Activated Sludge (SFAS)/Contact Stabilization Activated Sludge (CSAS)		

Alternative 2 is similar to Alternative 1 with the addition of an aeration basin used to equalize recycle streams from the solids processing. By equalizing the recycle streams, stable nitrification is possible and only one additional secondary clarifier would be necessary. In addition, CEPT would be used to reduce organic loading to the secondary system. Similar to Alternative 1, the additional tank requirements made Alternative 2 not feasible.

For Alternative 3, the HARRF would be operated at a 2.0-d SRT to suppress nitrification and reduce MLSS concentration. Suppression of nitrification will reduce aeration requirements and the reduction in MLSS concentration will reduce solids loading rate to the secondary clarifiers. To control biological foaming, the front end of each aeration basin (approximately 25 percent of total volume) would be converted to an anaerobic selector so that a 90^{th} percentile SVI value of 125 mL/g can be achieved. Alternative 3 would require the construction of an additional aeration basin. Alternative 3 will require that CEPT is used on an “as needed” basis to maintain a MLSS concentration that will not overload the secondary clarifiers during a peak flow event. Alternative 3 was considered to be a feasible alternative, and a cost analysis was performed.

Alternative 4 would involve converting the HARRF to a MBR facility. For the MBR, the secondary clarifiers are replaced with membrane (either microfilter or ultrafilter) that perform solid-liquid separation. Because secondary clarifiers are eliminated, the MLSS concentration can be operated at increased values (e.g., 10,000 mg/L). The MBR would be operated at an 8.0-d SRT to produce a nitrified effluent. Alternative 4 will require that additional tanks for the membranes be constructed. In addition, it is possible that additional aeration basins would be required due to the high oxygen requirements. Because the MBR would nitrify and CEPT would not be used, the capacity of the existing aeration system would need to be increased to account the higher secondary treatment loading. Because of the additional tank requirements, Alternative 4 was determined to not be feasible.

Alternative 5 would involve operating the HARRF at a 2.0-d SRT with an anaerobic selector similar to Alternative 3. In place of intermittent CEPT, primary effluent equalization would be performed. Equalization would be performed using two, 4-MG tanks. Due to the space requirements and additional odor control that would be necessary, Alternative 5 was determined to not be feasible.

For Alternative 6, the front portion of the existing aeration basins would be converted to a MBBR system. The MBBR would consist of carrier media that sustain a biofilm capable of performing carbonaceous BOD removal. The MBBR portion of the aeration basins would require that a concrete wall be constructed to isolate the MBBR from the rest of the aeration tank. Sieves would be installed to retain carrier media and allow treated effluent to pass through the MBBR. The downstream portion of the aeration basins would be used as a solids contact process where mixed liquor is recycled from the secondary clarifier. The solids contact portion, operated at a 1.0-d SRT, provides for additional BOD removal and improved flocculation of sludge that sloughs off the carrier media. Alternative 6 was considered to be a feasible alternative, and a cost analysis was performed. However, it is recommended that pilot testing be performed to verify process performance and determine aeration and sludge production values.

Alternative 7 would require the construction of one additional aeration basin. The process would be operated at an aerobic SRT of 2.0 d and two of the aeration basins would be converted to reaeration zones where RAS would be sent. The head end of the each aeration basin would be converted to an anaerobic selector (approximately 20 percent of the total aeration basin volume) to mitigate sludge bulking and improve sludge settleability. For Alternative 7 to be cost effective compared with

Alternative 3, it was assumed that CEPT would not be used and no additional secondary clarifiers would be constructed. The peak OUR values are predicted to be as high as 167 mg/L-hr which is at the limit possible with typical aeration systems. In addition, using two aeration basins for reaeration may make project staging difficult. Therefore, Alternative 7 was determined to not be feasible.

For Alternative 8, a contact tank would be constructed where, during peak flow events, primary effluent and RAS would be combined for a short contact period. The overall effect is that the SLR to the secondary clarifiers is reduced. Several scenarios were investigated to determine the amount of flow that would require treatment through the contact tank and the volume of the tank. The conclusion was that the amount of flow bypassed and the volume of the contact tank were too high for the process to be feasible.

4.2.6 Tertiary Treatment

Three advanced treatment options were identified that could replace the existing tertiary filters: nitrifying biological aerated filter (BAF) (A); membrane filtration (B); and MBR (C). For Alternative 3, any of the three options could be implemented; for Alternative 6 only Options A and B were considered. Only Option A would require that the existing granular media filters be kept in service; the membrane-filtered effluent for B and C could be sent directly to disinfection. However, it is recommended that if Option A or B is selected, the selected process be pilot tested to verify full-scale design criteria and/or process performance. The BAF technology used in Option A is proven technology; however it is still new for the US. It is valuable to obtain site-specific operational and performance data. In Option B, the membrane filter system would be operating on a non-nitrified effluent. It is difficult to predict cleaning intervals, membrane fluxes, and water recovery without pilot testing. The difficulty of treating a non-nitrified effluent is evident in the existing performance of the Dynasand filters.

Although the existing chlorine contact tank provides adequate capacity for future recycled water demands, Brown and Caldwell recommends that the City revisit the 4.0 mgd UV disinfection capacity approved by DHS in the fall of 2003, as there are several potential opportunities to increase system capacity. First, the commissioning testing identified an unequal flow split between the two UV channels. The channel with the higher flow was tested to simulate conservative performance. The delivered UV dose (and system capacity) would increase if the flows were balanced equally. Second, the ambient filter effluent UV transmittance had to be reduced by adding decaffeinated coffee to conduct the tests at 55-percent UV transmittance. A higher UV transmittance (and increased system capacity) could be demonstrated by collecting six months of UV transmittance data based on three grab samples per day. Finally, the City should check with the manufacturer (Trojan) to see if there have been any changes in DHS-approved operations parameters (end of lamp life factor, quartz sleeve fouling factor) for their low-pressure, low-intensity UV system that could increase the rated capacity.

Demonstrating a higher secondary effluent UV transmittance and using UV bulbs from a specific manufacturer could increase UV capacity to 6.6 mgd. Other modifications (e.g., hydraulic improvements) could increase the UV system capacity further, but would require another series of commissioning tests. Even if the UV disinfection system were not used for recycled water production, the UV system could be used for disinfecting filtered effluent for live-stream discharge to avoid any concerns of a final effluent chlorine residual. Increased UV system capacity would provide an alternative to chlorine disinfection.

Supplemental treatment facilities (e.g., reverse osmosis) would be needed if additional recycled water-quality-based requirements were set. Supplemental treatment facilities also would be needed if additional biological treatment for nutrient removal and tertiary treatment were required for an alternative effluent disposal scheme, such as live-stream discharge.

4.2.7 Solids Treatment

Solids processing requirements for each alternative were evaluated and co-thickening of primary and secondary sludge was evaluated as a process alternative. In general, co-thickening would require larger DAFTs, smaller anaerobic digesters, and fewer dewatering centrifuges compared with separate sludge thickening. Infrastructure upgrades necessary to treat the solids produced when treating 27.5 mgd of average daily flow at the HARRF are presented in Table 4-3.

4.2.8 Hydraulic Capacity

The existing treatment facilities cannot pass peak flows associated with 27.5 mgd average flow capacity without flooding outlet weirs and/or inlet or outlet channels in one or more treatment processes. The following modifications are recommended to mitigate hydraulic bottlenecks:

- Additional aeration tank inlet gates to mitigate primary clarifier effluent weir flooding.
- Additional aeration tank outlet gates to mitigate primary clarifier effluent weir flooding.
- Increase secondary clarifier inlet column openings to mitigate mixed liquor splitter box weir flooding.

4.2.9 Summary of Recommended Long-Term Improvements at the HARRF

Improvements to provide sufficient capacity to treat the average daily flow of 27.5 mgd expected at build-out are summarized in Table 4-3. The recommended improvements are to supplement the process units already existing at the HARRF. It should be noted that the improvements are not additive; rather, they are stand-alone recommendations to enable the HARRF to treat the average daily flow indicated.

Table 4-3. Summary of Recommended Improvements at the HARRF for 27.5 mgd Average Dry Weather Flow Treatment Capacity

Description	Units	Needed Improvements for Alternative 6 [a]	Needed Improvements for Alternative 3 [a]
Bar Screen			
Number to Install	---	1 (convert existing manual bar screen)	1 (convert existing manual bar screen)
Type	---	Mechanical	Mechanical
Bar Spacing	mm	6	6
Peak Capacity	mgd	22	22
Influent Pump Station			
Pumps	---	▪ Operate the 9000-gpm pumps at 10-12 percent higher than the design speed to provide additional capacity. ▪ Determine if the pump foundation, frame, motor supports and rotating system can withstand the dynamic forces resulting from operation at the higher speeds.	
Motor	---	▪ Determine if the motor design is adequate to handle the additional electrical current and voltage at the higher speed	
Variable frequency drives (VFDs)	---	▪ Conduct a field torsiograph test to identify torsional resonance issues and determine if the existing VFDs can operate at speeds greater than 60 Hz.	
Discharge Force Main	---	▪ Upgrade of discharge force main to 36 inch pipe	
Grit Removal [b]			
Number to Install	---	1	1
Type	---	Vortex	Vortex
Diameter	ft	24	24
Average capacity	mgd	21	21
Primary Clarifiers	*Convert all primary clarifiers to CEPT* [c]		
Primary Clarifier Basins	number	1	1
Side water depth	ft	10	10
Surface area per tank	ft^2	5,250	5,250
Primary Sludge Pump Station			
Diaphragm Pumps	number	1	1
Pump capacity, each	gpm	150	150
Design head	ft	80	80
Activated Sludge System			
Aeration Basins	number	None	1
Side water depth	ft		16.5
Surface area	ft^2		75,000
Blowers	number		1
Pump capacity, each	scfm		10,300
Aeration Basin Modifications		*Convert 25 percent of the existing aeration basin to MBBR. One aeration basin will be dedicated to MBR in Alt 6C.*	*Convert 20 percent of the existing aeration basins into biological selector zone. One aeration basin will be dedicated to MBR in Alt 3C.*

Table 4-3. Summary of Recommended Improvements at the HARRF for 27.5 mgd Average Dry Weather Flow Treatment Capacity

Description	Units	Needed Improvements for Alternative 6 (a)		Needed Improvements for Alternative 3 (a)		
Submersible Mixers	number	None		7 (6 duty + 1 standby)		
Advanced Treatment System (to produce recycled water)						
		Alt 6A	Alt 6B	Alt 3A	Alt 3B	Alt 3C
Type of system		BAF	MF	BAF	MF	MBR
Average capacity	mgd	9.0	9.0	9.0	9.0	9.0
Return Activated Sludge Pump Station				Alt 3A and B		Alt 3C
Return Activated Sludge Pumps	number	3		3		4
Pump capacity, each	gpm	4,107		4,107		10,000
Mixed Liquor Pump Station						
Waste Activated Sludge Pumps	number	2		2		
Pump capacity, each	gpm	2430		2430		
Sludge Thickening						
Co-Thickening				Alt 3A and 3B		Alt 3C
Dissolved Air Flotation Thickener Basin	number	2		2		2
Diameter	ft	37		37		37
Thickened Sludge pumps	number	2		2		2
Pump capacity, each	gpm	260		260		260
Pressurization System(d)	number	4		4		4
Pump capacity (e), each	gpm	1,000		1,000		1,000
Pump pressure	ft	175		175		175
Compressor (f)	number	1		1		1
Compressor capacity, each	scfm	15		15		15
Separate Thickening				Alt 3A and 3B		Alt 3C
Dissolve Air Flotation Thickener Basin	number	1		1		2
Diameter Dissolve Air Flotation	ft	36		36		36
Thickened Sludge pumps	number	2		2		2
Pump capacity, each	gpm	260		260		260
Pressurization System	number	1		1		1
Pump capacity(f), each	gpm	500		500		500
Pump pressure	ft	175		175		175
Compressor	number	1		1		1
Compressor capacity, each	scfm	15		15		15
Sludge Degritting and Dewatering System(b)						
Slurrycup Grit Washing Units	number	2		2		
Diameter	ft	56		56		
Capacity, each	gpm	650 to 950		650 to 950		
Grit Snail	number	1		1		
Capacity	cu yd /hr	4		4		
Sludge Digestion						
Co-Thickening						
Anaerobic Digesters	number	1		1		
Tank Diameter	ft	109		109		
Side water depth	ft	25		25		

Table 4-3. Summary of Recommended Improvements at the HARRF for 27.5 mgd Average Dry Weather Flow Treatment Capacity

Description	Units	Needed Improvements for Alternative 6 (a)	Needed Improvements for Alternative 3 (a)	
Separate Sludge Thickening			Alt 3A and 3B	Alt 3C
Anaerobic Digesters	number	1	1	1
Tank Diameter	ft	141	141	142
Side water depth	ft	25	25	25
Dewatering System				
Co-Thickening				
Centrifuge number	number	1	1	
Average capacity each,	gpm	150	150	
Operating Centrifuge number	hrs/day	12	12	
Separate Sludge Thickening				
Centrifuge number	number	2	2	
Average capacity each,	gpm	150	150	
Operating schedule	hrs per day	12	12	

Notes:
(a) Needed improvements to the existing HARRF to enable treatment of 27.5 mgd average daily flow expected at build-out. Assumes that near-term improvements have not been implemented. Alternative 6A, 6B, 3A, 3B and 3C relate to improvements needed to produce secondary effluent with water quality characteristics appropriate for ocean discharge, plus the addition of an advanced treatment system that could produce up to 9.0 mgd of recycled water. Advanced treatment options for each alternative are as follows: Alternative A = nitrifying biological aerated filters; Alternative B = membrane filtration; Alternative C = membrane bioreactors.
(b)Grit removal system at the headworks is required for separate sludge thickening option only. Sludge degritter and dewatering system is required for co-thickening option only.
(c) CEPT will be used for near-term solution for the HARRF regardless which alternative is selected. For build-out condition, CEPT will only be used for Alternative 3 on a routine basis. For Alternative 6, CEPT is necessary during construction only, and will not be necessary upon completion.
(d) Each pressurized injection system consists of one tank and one pump. Compressors are operated on a common manifold that services all dissolved air flotation thickeners. Assumes that two (one duty and one standby) pressurized injection systems are added to existing dissolved air flotation thickeners. 100 percent redundancy is provided.
(e) Assumes that the compressed air system operates on a common discharge manifold to allow service to all dissolved air flotation thickeners. Additional compressors noted are required due to the new dissolved air flotation thickeners proposed.
(f) Assumes existing system is upgraded to match the capacity of the new proposed systems to provide uniformed sizing for redundancy.

4.3 Other Recommended Improvements

The existing odor control system at the HARRF treats 66,000 cfm of foul air from the Primary Clarifier Building (PCB). A two-stage odor control system (OCS) consisting of a bioscrubber followed by a mist scrubber was designed for handling hydrogen sulfide (H_2S) concentrations up to 200 parts per million by volume. However, current operational practice uses only the first stage for foul air treatment and recent monitoring results indicate that the bioscrubber is ineffective. Recommended improvements are briefly described below.

Immediate improvement in OCS operation can be achieved by replacing primary effluent with secondary effluent or reclaimed water as the bioscrubber wetting agent and by establishing continuous wetting agent recycle instead of using a single-pass system. It was discovered that the sulfide-laden primary effluent occasionally caused the outlet gas phase sulfide concentration to be higher than the foul air inlet. Secondary effluent and tertiary effluent have low sulfide concentration, thus the volatilization of sulfides should not occur. Nutrients must be added periodically to a bioscrubber operated using tertiary effluent to maintain a healthy microbial population in the bioscrubber.

Long-term improvements include reducing the amount of foul air requiring treatment by installing covers on the primary clarifiers and withdrawing foul air from beneath the covers rather than from the entire PCB. Covering and ventilating the primary clarifier should improve the air quality in the PCB.

A new odor control system can be installed to treat the reduced foul air volume as well as the foul air from the headworks. Approximately 30,000 cfm of foul air is expected from the primary clarifier and the headworks. Since foul air sampling and monitoring conducted during an evaluation of the OCS indicated an average H_2S concentration of less than 5 ppmv, a single-stage carbon adsorber system would provide adequate treatment. Because of its proximity to the primary clarifiers, improvements to the odor control system can be implemented when the fifth primary clarifier is installed.

4.4 Construction Phasing of the HARRF Improvements

The period when the average annual flows at the HARRF reach the current rated capacity of 18.0 mgd and the projected build-out flows of 27.5 mgd are presented in the *Flow Projection Analysis TM, Brown and Caldwell, December 1, 2006*. The projected flows were derived by plotting the annual average daily flow recorded from 2000 to 2005 and developing a trendline from the anticipated growth in 2006 to 2008 in the HARRF sphere of influence. The trendline was then extended to future years. The projected flows versus calendar years are plotted on Figure 4.1 as a red solid line. Based on this projection, the influent flow to the HARRF should reach 18.0 mgd by the year 2014 and 27.5 mgd by the year 2041.

The HARRF was determined to be less than the current rated average flow capacity of 18.0 mgd. Treatment capacity will need to be expanded to match increasing flows. Implementation of permanent improvements to ensure that the HARRF maintains adequate capacity to treat incoming flows can take several years to plan, design and construct. Meanwhile, interim, near-term improvements to the secondary and solids treatment system can be implemented to make certain that the plant capacity can treat an average daily flow of 18.0 mgd. These improvements are described in Section 4.1. It is estimated that incorporating the improvements will take approximately a year. With the assumption that planning and design starts in 2007, the improvements can be utilized starting 2009 (see Table 4.4).

Also depicted in Figure 4.1 is a stepped line that describes the proposed phased long-term improvements that must occur at the HARRF to keep pace with the population and development growth within the sphere of influence. The improvement phasing shown assumes that Alternative 3B (with co-thickening as the sludge thickening option) is implemented. Alternative 6 is still considered a viable option, however a pilot testing program has been recommended to verify process performance and determine design parameters. Phasing for Alternative 6, if selected, would be similar to Alternative 3. The exception is that during Phase 3, a portion of the existing aeration basins would be converted to MBBR facilities for Alternative 6 in lieu of construction of a sixth aeration basin necessary for Alternative 3. The tertiary treatment option selected for the phasing discussion was assumed to be Option B (membrane filtration).

Phase numbering designation begins with "Phase 3" since the plant has recently undergone Phase 1 and 2 improvements.

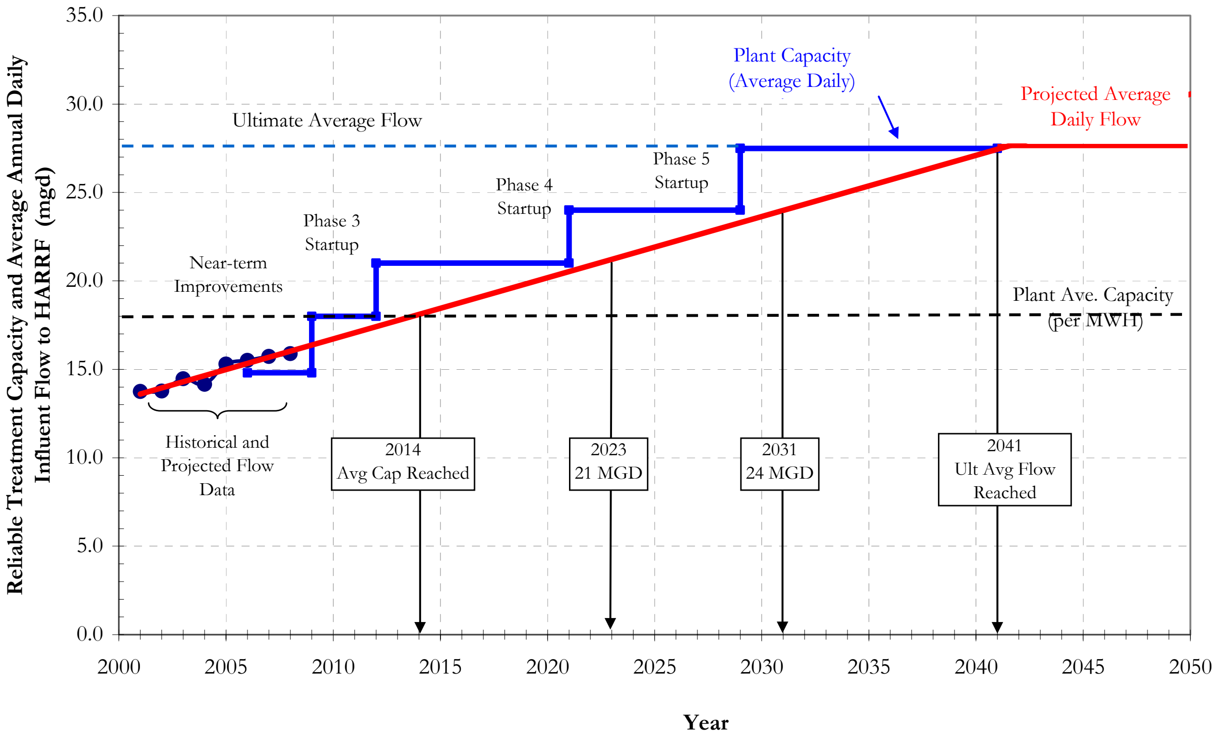


Figure 4-1. Projected Influent Flow at the HARRF

4.4.1 Long-Term Improvements for Expansion of Secondary Treatment Capacity

The plant improvements are proposed to occur in three phases to minimize overbuilding in any phase for efficient capital expenditures and to ensure that the reliable treatment capacity is always greater than the projected flow. An estimated schedule for planning, design, and construction for the four phases is shown in Table 4.4, but the timing will depend on the actual rate of development and increase in flows. The timing shown below is based on a single construction contract for each phase. Plant startup for a particular phase improvement is assumed to occur two years prior to the time when the anticipated flow reaches the plant capacity before the improvements. This is numerically presented in Table 4.4 and graphically shown in Figure 4.1. This time gap will allow operators to get familiar with the new process units and optimize the new systems. Figure 4.2 illustrates the capacity increases in individual unit processes with each phase. Specific improvements associated with each phase are described below.

Under the current NPDES permit, the City is required to submit a written report to the Water Board "within 90 days after the monthly average influent flow rate for any 30-day period equals or exceeds 75 percent of the design capacity (13.5 mgd) of the waste treatment and/or disposal facilities." The report must include the City's "intended schedule for studies, design, and other steps needed to provide additional capacity for the waste treatment and/or disposal facilities, and/or control the flow rate before the waste flow exceeds the capacity of present units." Considering that the HARRF received on average 15 mgd in 2005, this 75 percent of capacity trigger/criterion is in effect. However, this study, which considers a phased construction plan to make certain the HARRF continues to have adequate capacity to treat incoming wastewater flows until buildout, is believed to meet this criterion.

Table 4-4. Planning Schedule for Design and Construction Improvements at the HARRF

Phase	Reliable Treatment Capacity After Improvement, mgd[a]	Initiate Planning/Design	Initiate Construction	Plant Startup	Year When Capacity is Reached
Near Term	18.0	2007	2008	2009	2014
Long-Term-Phase 3	21.0	2007	2009	2012	2023
Long-Term-Phase 4	24.0	2017	2019	2021	2031
Long-Term-Phase 5	27.5	2027	2028	2029	2041

(a) Average dry weather flow capacity with one unit out of service for each process.

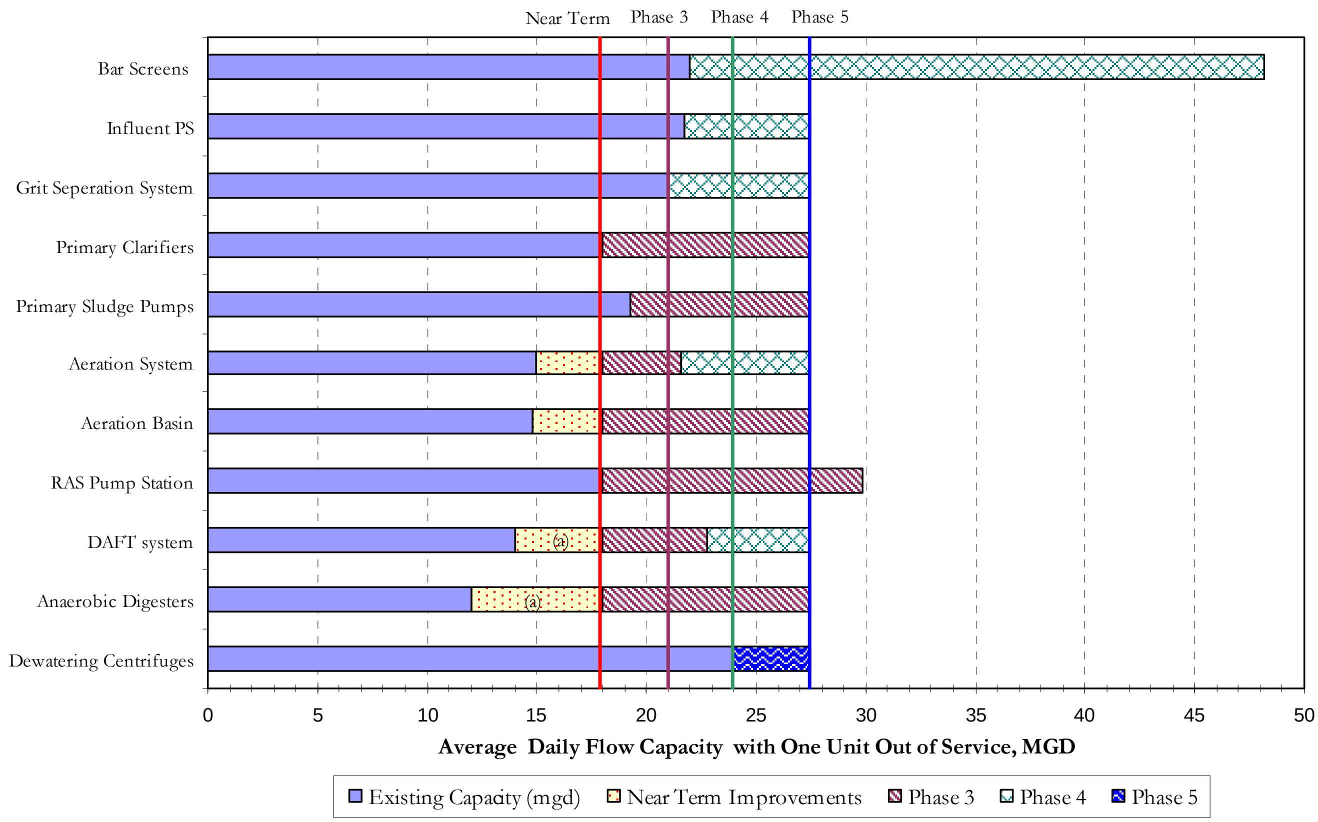


Figure 4-2. Capacity Increases for Each Phase at the HARRF

Note: (a) All the units are in operation during near-term improvements.

Phase 3 (21 mgd). Phase 3 will require the following improvements:

- Construction of one primary clarifier
- Addition of one primary sludge pump
- Addition of equipment to enable CEPT
- Construction of one aeration basin
- Replacement of the fine-bubble aeration system
- Installation of anaerobic selectors to all aeration basins
- Replacement of the existing RAS pumps with two larger capacity pumps
- Implementation of hydraulic improvements to aeration basin inlet and outlet gates, and secondary clarifier influent orifices
- Construction of a new WAS pump station
- Upsizing of the existing WAS pipe
- Construction of one new dissolved air flotation thickener
- Construction of one new anaerobic digester to handle 27.5 mgd.
- Improvement of the headworks and primary clarifier odor control system

The additional primary clarifier will provide redundancy and reliable capacity during dry weather if a tank must be taken out of service. The tank size will match the dimensions of the existing Clarifier No.4 (50-width, 12 feet deep). Note that with the addition of one primary sludge pump, the primary treatment system will be capable of treating flows received at build-out.

Conversion of all primary clarifiers to CEPT is needed. During dry weather operation, CEPT will not be necessary. However, during the PWWF conditions, CEPT is necessary to control MLSS concentration. This will require constructing a chemical storage area and a feeding system.

The addition of an anaerobic selector zone in each aeration basin, the reduction of the SRT from 2.75 to 2.0 days, and the replacement of the existing fine-bubble aeration system with aeration diffusers capable of operating at a higher flux (equivalent to the blower capacity) will improve sludge settleability and increase secondary process capacity to 21.6 mgd. The lowering of the SRT will suppress biological nitrification, reducing aeration requirements, resulting in an increased capacity rating for the blower system.

Replacement of the diffusers in an aeration tank requires removing one tank from service at a time. However, there is a potential for the effluent quality to degrade during this interim period, particularly because of the lack of aeration capacity. Therefore, construction of one aeration basin has been included in Phase 3. Not only does the added aeration basin provide operational flexibility, it will increase the capacity to enable the secondary treatment system to treat flows at build-out conditions.

Sludge wasting should be performed from the mixed liquor rather than settled sludge to provide better control on SRT adjustment in aeration basins. This would require increasing the WAS pumping capacity and replacing the existing 8-inch diameter WAS pipeline with a 12-inch diameter pipeline. The new WAS pump station can be constructed between the splitter box and the proposed new aeration basin. The existing WAS pumps and pipelines must remain in service while the new equipment and aeration basin is constructed. After completion, the 8-inch diameter WAS forcemain can be abandoned. Construction of the new WAS pump station will increase the WAS pumping capacity to final build-out capacity.

Replacing the existing smaller capacity RAS pumps with larger capacity pumps will increase the RAS pumping capacity to the final build-out capacity.

Construction of one additional anaerobic digester will increase capacity of the process to the required final build-out capacity. Construction of one more DAFT is also required to bring the DAFT capacity beyond 21.0 mgd.

Because of the sequencing requirements necessary for replacing the aeration equipment (only one tank out of service at a time) it is estimated that construction will take approximately three years. Planning and design is estimated to begin approximately two years prior to construction commencement. Considering two years for optimization and familiarization, the Phase 3 project should commence no later than 2007 if ensure that there is sufficient capacity by 2014.

Phase 4 (24.0 mgd). Phase 4 will require the following improvements:

- Conversion of the existing manual bar screen to mechanical bar screen
- Modifications to influent pump station and force main
- Addition of one sludge degritter system
- Addition of one aeration blower
- Construction of one new dissolved air flotation thickener

The modifications to the bar screen and the addition of sludge degritter unit increase the preliminary treatment capacity to beyond build-out flow. The pump station modifications and the additional blower and addition of the second DAFT unit will bring the capacity of these processes to 27.5 mgd.

It is estimated that the construction necessary for Phase 4 can be completed within two years and that planning and design will take two years.

Phase 5 (27.5 mgd). Phase 5 will require the following improvements:

- Addition of one dewatering centrifuge
- Modification of the existing centrifuge feed and drain piping

An additional centrifuge will be necessary to achieve build-out capacity if the centrifuges are operated 12 hours per day. If the centrifuges are operated continuously, then there is no need for an additional centrifuge. Feed piping and drain piping will need to be upsized regardless of centrifuge operation. The existing building needs to be reevaluated to determine if there is sufficient space to accommodate an additional centrifuge. If not, the building will be extended.

Phase 5 could be combined with Phase 4 which will reduce the overall cost due to elimination of additional mobilization/demobilization cost for the construction. If Phase 5 is kept as a separate phase, it is estimated that construction will take one year and design less than one year.

4.4.2 Long-Term Improvements for Expansions of Tertiary Treatment Capacity

Tertiary treatment requires filtration and disinfection steps designed and operated in accordance with Title 22 regulations to produce “disinfected tertiary” quality recycled water to meet projected recycled water demands of 9.0 mgd. The *1999 Phase 2 Treatment Upgrades and Water Reclamation Facilities Project* included continuous backwash granular media filters and UV disinfection to produce 9.0 mgd of disinfected tertiary recycled water. The former secondary clarifiers were modified in the *2005 Chlorine Contact Tank Design/Build Project* to provide a chlorine contact tank as an alternate means of recycled water disinfection.

The granular media filters were designed for a maximum influent flow of 10.0 mgd with a loading rate of 4.34 gpm/ft^2 based on all the filters in service (*Parkson Corporation, Specification Section 11422*). The corresponding maximum hydraulic loading rate with one unit out of service is 5.0 gpm/ft^2. At the design waste washwater flow rate of approximately 80 gpm per filter, the total waste washwater flow is expected to be 0.8 mgd, which results in a net filtered water production of approximately 9.0 mgd. Under these conditions, the filters are specified to produce an effluent at 2 NTU for a typical secondary effluent at 10 NTU.

The filters, however, have never been able to operate at their rated hydraulic loading of 5.0 gpm/ft^2 and comply with the 2 NTU filter effluent limit. Therefore, the full capacity of these filters has not been achieved despite efforts by plant staff, the manufacturer, and various chemical suppliers. The current backwash rate is approximately 20 percent. Higher reject water quantity reduces the net filtered water production. Therefore, the maximum possible net filtered water production is reduced to 8.0 mgd at the maximum filter loading rate of 10.0 mgd.

The UV disinfection system was designed for a maximum recycled water production of 9.0 mgd. The criteria for rating the capacity of installed systems changed between facility design and startup. The system is rated for only 4.0 mgd based on commissioning tests conducted in accordance with the new UV design guidelines. The chlorine contact tank was added because of the low rated capacity of the UV disinfection system. Chlorine contact tank performance testing demonstrated a capacity of more than 10.0 mgd.

Figure 4.3 shows the capacity increases for each tertiary treatment process to provide a recycled water capacity of 9.0 mgd. New filters, rated at 9.0 mgd, are included in Phase 3. Membrane filters are recommended as they can provide acceptable filter effluent quality throughout a range of secondary effluent quality. The existing chlorine contact tank capacity exceeds the 9.0 mgd recycled water demand, therefore no expansion is needed. However, modifications to the existing tank are recommended that partition the tank into two or three parallel sections. This will allow part of the contact tank to be taken out of service for maintenance (e.g., cleaning) while maintaining some contact tank capacity. In addition, part of the contact tank could be taken out of service when recycled water demands are significantly less than 9.0 mgd to optimize contact time and minimize chlorine use.

While the chlorine contact tank can meet recycled water disinfection needs, there are several potential opportunities to increase capacity of the UV disinfection system. Additional UV system capacity would eliminate the need for dechlorination when recycled water is discharged to Escondido Creek during high flows. Demonstrating a higher secondary effluent UV transmittance and using UV bulbs from a specific manufacturer could increase UV capacity to 6.6 mgd. Other modifications (e.g., hydraulic improvements) could increase the UV system capacity further, but would require another series of commissioning tests.

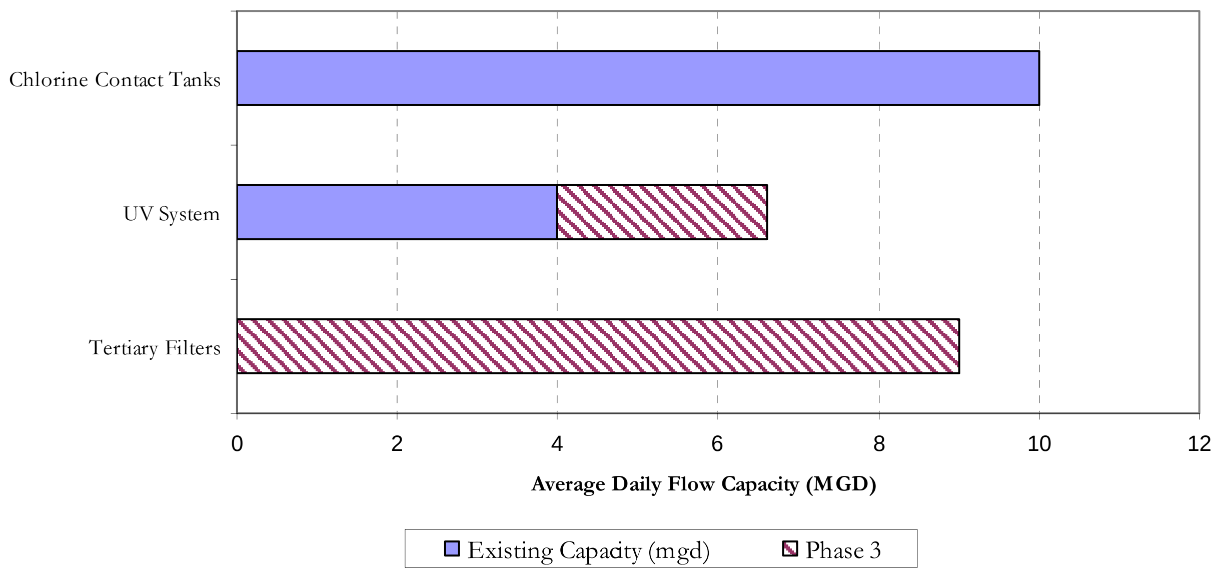


Figure 4-3. Capacity Increases for Tertiary Treatment at the HARRF

5.0 EFFLUENT DISPOSAL

Presented in this section are constraints that impact effluent disposal, derivation of disposal flow conditions, and description and cost estimates of the recommended disposal options.

5.1 Constraints Impacting Effluent Disposal

Developing disposal strategies requires knowledge of certain constraints that limit or even preclude certain options. In this section, these constraints are mentioned and discussed.

5.1.1 Total Suspended Solids (TSS) and 5-day Carbonaceous Biochemical Oxygen Demand ($CBOD_5$) Mass Emission Rate Implications

The current NPDES permit for the disposal of effluent to the ocean (Order No. R9-2005-0101, NPDES Permit No. CA0107981) contains "antidegradation" and "antibacksliding" policies that regulate the increase in discharges to the ocean. The antidegradation policy mandates that "existing quality of waters are maintained unless degradation is justified based on specific findings." The antibacksliding policy requires that "effluent limitations in a reissued permit to be as stringent as those in the previous permit, with some exceptions where limitations may be relaxed." As noted in these two policies, there is some room for slight modification to the discharge quantities prescribed in the permit, but it must be proven that modification will not impact the receiving waters. This was evident in the latest permit renewal in which the Water Board proved in the Fact Sheet that, although the renewal involved relaxation and deletion of some effluent limitations, there was no adverse impact found to the receiving water quality.

Recent discussions held with the Water Board indicate that an antidegradation analysis must be conducted if there are significant changes to the volume and mass loading related to the discharge of treated effluent to the ocean. However, the analysis may not be required if the effluent quality is such that the existing mass emission limitations are not exceeded significantly (less than 10 percent). To clearly avoid the antidegradation analysis altogether, the mass emission rate limitation associated with TSS and $CBOD_5$ and the 90^{th} percentile values for $CBOD_5$ and TSS concentration measured in the HARRF secondary effluent in the past 5 years were used as the basis to determine the "room under the cap." The following mass emission rate (MER) values were derived and are expressed in pounds per day (lb/d):

Permit Mass Emission Rate Effluent Limitations

- TSS Mass Emission Rate
  - ✓ Average Monthly = 4,500 lb/d
  - ✓ Average Weekly = 6,800 lb/d

- $CBOD_5$ Mass Emission Rate
  - ✓ Average Monthly = 3,800 lb/d
  - ✓ Average Weekly = 6,000 lb/d

Estimated Mass Emission Rates at 27.5 mgd Production Rate

- 90^{th} percentile TSS Concentration = 20.6 mg/L
- 90^{th} percentile $CBOD_5$ Concentration = 15.8 mg/L
- TSS Mass Emission Rate = 4,725 lb/d
- $CBOD_5$ Mass Emission Rate = 3,624 lb/d

The estimated effluent values for build-out presented above indicate that the TSS mass emission rate will be slightly exceeded, but the $CBOD_5$ will not. However, this analysis assumes 100-percent disposal to the ocean and no ongoing water recycling. More discussion is presented later about how this impacts disposal options.

5.1.2 Impact of Water Reuse

Water reuse allows the disposal of effluent that would otherwise have been discharged to the ocean. Prevailing climate in the San Diego area allows recycled water to be used for irrigation a majority of the year. In addition, there are other uses, such as cooling water for a power plant, that occur year-around. However, for conservatism, it was assumed that of 9.0 mgd of recycled water produced, 50 percent is disposed of land (or air via evaporation at the power plant) and the remaining 50 percent is discharged to the ocean. Subtracting the amount lost to recycling, the estimated MERs are as follows:

Amount Eliminated Through 9.0 mgd of Water Reuse

- TSS = 1,145 lb/d
- $CBOD_5$ = 878 lb/d

Total Mass Emission Rate at Build-out (including Impact of Water Reuse)

- Resultant TSS Mass Emission Rate = 3,580 lb/d
- Resultant $CBOD_5$ = 2,746 lb/d

It is expected that an average of 1.0 mgd and a maximum of 1.5 mgd of brine will be discharged through the outfall. However, the TSS and $CBOD_5$ contribution of the brine discharge is minimal. Therefore, the "room under the cap" allows about 35.1 mgd AADF to be discharged to the outfall, assuming that the effluent quality does not degrade beyond the 90^{th} percentile values reported earlier. This fact points out that the disposal options for the HARRF is not limited by the MER; rather, it is governed by the disposal of effluent during wet weather periods.

5.1.3 Continuous Live-stream Discharge

Another option for disposal of treated effluent is to continuously discharge to Escondido Creek an amount that exceeds the outfall capacity. Effluent standards are likely to be the same as those prescribed for the intermittent live-stream discharge (i.e., Order No. R9-2003-0394). The most notable effluent standards are those for phosphorus and nitrogen which reads as follows (for the Escondido Creek Hydrologic Subarea (HSA) 904.61 and 904.62):

"Concentrations of nitrogen and phosphorus, by themselves or in combination with other nutrients, shall be maintained at levels below those that stimulate algae and emergent plant growth. Threshold total Phosphorus (P) concentration shall not exceed 0.05 mg/L in any stream at the point where it enters any standing body of water, nor will 0.025 mg/L in any standing body of water. A desired goal in order to prevent nuisances in streams and other flowing waters appears to be 0.1 mg/L total P. These values are not to be exceeded more than 10 percent of the time unless studies of the specific water body in question clearly show that water quality objective changes are permissible and changes are approved by the Regional Board. Analogous values have not been set for nitrogen compounds; however, natural ratios of nitrogen to phosphorus are to be determined by surveillance and monitoring and upheld. If data are lacking, a ratio of N:P = 10:1 shall be used."

This standard essentially limits total P to 0.1 mg/L and total N to 1.0 mg/L, which is difficult to achieve. It is very restrictive, ultimately limiting the selection of the appropriate process to implement at the HARRF to nutrient removal processes. Biological methods are limited to certain effluent concentrations which are above the noted criteria. The standard will have to be met by biological treatment combined with physical/chemical treatment, likely requiring treatment by RO. Pursuing this option will need extensive work, including process and environmental impact evaluation. Furthermore, a pilot test will be needed to determine the effectiveness of certain treatment processes in achieving the discharge criteria at local conditions. Finally, the practice of continuous live-stream discharge will be a pioneering endeavor for the San Diego region and, consequently, will be a challenge for the City to implement.

For this study, the use of RO (and in some cases microfiltration ahead of RO for preconditioning) was included in some of the disposal options that require intermittent live-stream discharge. Based on the current permit allowing intermittent discharge, the City is required to redirect Escondido Creek flows back to the HARRF to recover during the dry weather periods the amount of nutrients (e.g., N and P) discharged to the creek during the wet weather season. There is a concern that the varying quality of Escondido Creek will not yield a sufficient amount of nutrients to recover the quantities discharged to the creek. Therefore, provisions were included to remove N and P in the tertiary effluent. RO was selected because it could easily be "switched on" when needed without the need for acclimation. However, it is strongly recommended to pilot an RO system to confirm its N-removal capability.

5.1.4 Groundwater Replenishment

A limited amount of information was available to the project team related to this disposal method. Information summarized below were extracted from the March 1999 *Hale Avenue Resource Recovery Facility Phase II Treatment Process Upgrades and Enhancements Facility Plan*, and conversations with City of San Diego staff.

The City evaluated the possibility of recharging the following three groundwater basins with tertiary effluent from the HARRF:

- Escondido Basin
- San Dieguito Basin
- San Pasqual Basin

The Escondido Basin was found to be too small and was not suited for cost-effective recharge. The San Dieguito Basin was too far from City facilities; it was discovered that it was more cost-effective for agencies closer to the basin to conduct the recharge operation. A majority of the San Pasqual basin is occupied by an agricultural preserve owned by the city of San Diego. San Diego staff indicated that recharge of tertiary effluent to the San Pasqual Basin was abandoned after receiving significant and very vocal opposition from farmers, citizens, and politicians in the area. It appears that groundwater recharge may not be a feasible alternative. In addition, depending on the groundwater management that occurs within the basin, it may not offer the year-around disposal opportunity needed to offset discharge through the land and ocean outfall.

5.2 Disposal Flow Conditions

The effluent flow conditions used to study the disposal options composed of dry and wet weather flows. This section describes the methods and assumptions used to derive these influent flows.

5.2.1 Dry Weather Flows

Dry weather flows (DWFs) are population-based flows entering the HARRF from the City of Escondido and Rancho Bernardo (via PS 77). Flow data obtained from the HARRF influent flume located at the headworks provided dry and wet weather flows for the 2005/2006 period. DWFs predicted at build-out were calculated by proportionally increasing the 2005 flows by the ratio of average dry weather flows estimated at build-out and today's average dry weather flow. The daily average DWF at build-out was obtained from the *Collection System Master Plan*. The following data, evaluated from observed meter readings, summarizes the DWF calculations:

- Average DWF from Escondido (existing) = 9.7 mgd
- Average DWF from Rancho Bernardo (existing and build-out) = 3.7 mgd[1]
- Annual average build-out flow (from Master Plan) = 27.5 mgd
- Annual averaged daily storm flow (from storm flow analysis) = 1.1 mgd
- Build-out DWF factor (applied to Escondido flow) = (27.5 – 1.1 – 3.7) / 9.7 = 2.33

Note 1: Rancho Bernardo (contractual) average flow = 5.3 mgd

The DWF multiplier derived above was applied to existing 2005 flow data to create time-varying hourly flow data formatted for the hydraulic model. Prior to routing the DWFs through the equalization storage basins / tanks, the flows were reduced by 2.7 mgd to account for the evaporation losses incurred by the local power generation facilities. The reduction due to evaporation also accounted for 1.5 mgd returned directly to the outfall. The final predicted dry weather flow at build-out has the following characteristics which are depicted on Figure 5-1:

- Average build-out dry weather flow = 23.7 mgd
- Peak build-out dry weather flow = 32.3 mgd

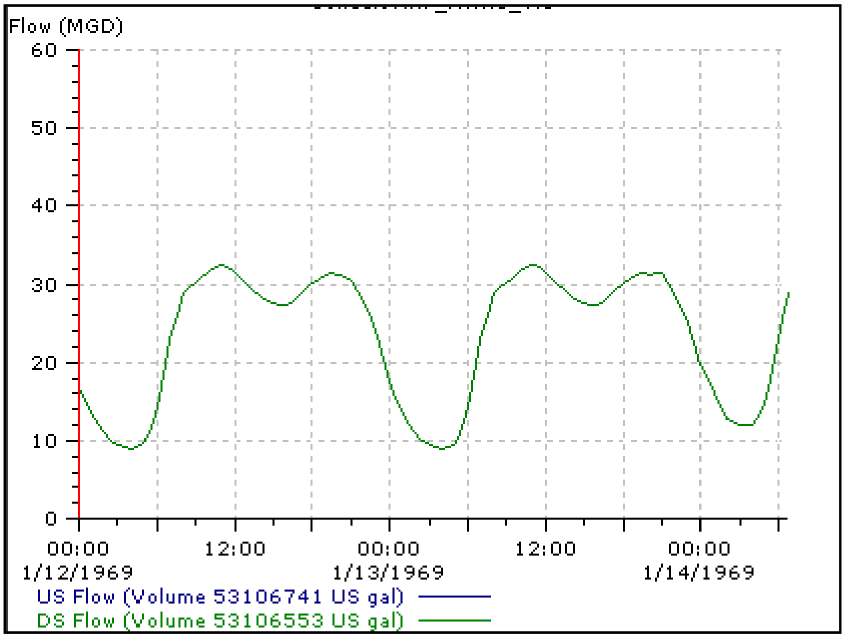


Figure 5-1. Predicted Dry Weather Flow Hydrograph for Build-out

5.2.2 Wet Weather Flows

The wet weather flows (WWFs) are generated from rainfall dependent inflow and infiltration (RDI/I) flows which comprise of groundwater infiltration (GWI) and storm peak flows. During the rainy season (typically from December through April), the base DWFs are elevated due to rising groundwater levels which manifest into the collection system as groundwater infiltration. In addition to the high base flows, peak flows generated from storm flows (entering the system via direct connections, cracks, and overflowing storm systems) are added to the base flows to create the 'worst-case' flow scenario.

The WWF condition provides the peak flow used to size the disposal facilities (e.g, outfall), and the storm volume is used to size the equalization facilities. The sizing of the disposal and equalization facilities is governed by the magnitude of the wet weather 'design' event. For this study, the WWF event used to size the facilities was selected from a historical rainfall time-series event. The event was selected on the basis that the disposal facilities (i.e. equalization basin) are filled once every 10 years (i.e. a 10-year return period). This '10-year' event was selected by ranking the peak flows and resulting storage volumes for a 41-year rainfall record. The storm flow component, as shown on Figure 5-2, was combined with the build-out dry weather flows to generate the time-varying influent design flows. As shown on Figure 5-2, the storm flows are composed of both peak storm flows and groundwater infiltration (shown as the elevated base flow). The groundwater infiltration is significant when sizing equalization facilities, as resulting GWI-based flows will fill the facilities if the outfall capacity is limited to peak dry weather flow.

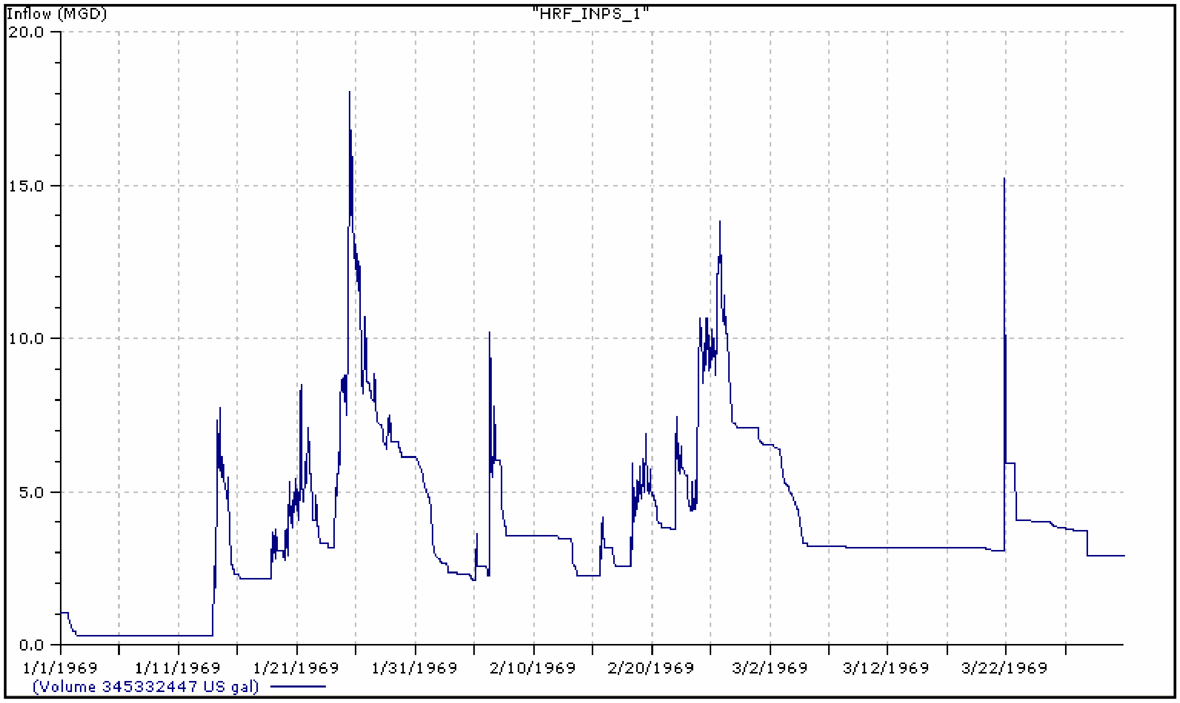


Figure 5-2. Design Storm Flows

The development and use of a 'design storm' to plan the disposal facilities differs from the use of an average dry weather flow and a corresponding peaking factor deployed in the analysis of the HARRF treatment processes. The two design-flow criteria were compared to ensure the plant and disposal upgrades were based on consistent design criteria. The following summarizes the comparison between the two design flow criteria.

Disposal Design Flow Criteria

- Peak wet weather flow of 10-year 'design' storm plus build-out DWF = 53.7 mgd
- Average daily flow (for storm period) = 30.4 mgd
- Peaking factor = 1.77

Process Design Flow Criteria

- Peak wet weather flow = 50.9 mgd (includes evaporation losses)
- Average dry weather flow = 24.8 mgd
- Peaking factor = 2.0

5.3 Viable Disposal Options

The three disposal and equalization options considered during this study include upgrading or building a new outfall (land and ocean), expanding equalization facilities at the HARRF, and conducting the live-stream discharges. The objective of this task is to determine the optimum solution satisfying the following criteria:

- Dispose and/or store the peak flow and volume for the 10-year design event
- Maximize the use of live-stream discharge
- Maximize the use of existing disposal and equalization facilities
- Minimize the cost of building new facilities

All disposal options assume flows from Rancho Bernardo continue discharging to the HARRF. Additional evaluation of the following disposal options will be required if Rancho Bernardo flows are re-directed into the City of San Diego's collection system, hence reducing the HARRF influent flow.

The following sections discuss the options in further detail along with the benefits and engineering implications associated with each option.

5.3.1 Escondido Land Outfall Options

The existing ELO capacity is limited to 21.4 mgd (pressurized) and 23.7 mgd (gravity), which is less than the current peak flow occurring during the wet season. Although minor improvements such as sealing the manholes local to the siphons will increase the capacity to approximately 25.0 mgd, significant improvements will need to be addressed to satisfy the ultimate peak flows. The study identified a series of options for increasing the land outfall capacity, including retrofitting the existing outfall, constructing a new outfall, or constructing a secondary outfall supplementing the existing outfall. Table 5-1 summarizes the land outfall capacity improvement options, identified as ELO-1 through ELO-5.

Table 5-1. Escondido Land Outfall Improvement Options

Option ID	Description	Capacity (mgd)	Notes
ELO-1	Existing improvements	25.0	Seal existing siphon manholes
ELO-2	Retrofit to force main	33.0	Seal manholes and line existing pipe
ELO-3	New or secondary outfall	33.0	New outfall size = 42- to 48-inch Secondary outfall = 30-inch
ELO-4	New or secondary outfall	45.0	New outfall size = 54- to 72-inch Secondary outfall = 42-inch
ELO-5	New or secondary outfall	49.0	New outfall size = 54- to 72-inch Secondary outfall = 42-inch

To meet the required capacity of the land outfall shown in Table 5-1, a new or secondary land outfall ranging in diameter from 36 to 72 inches would be needed. The new or secondary land outfall would start from the HARRF to the terminus point of the land outfall – the SEJPA Regulator Structure.

To evaluate the feasibility of constructing the new or secondary land outfall, Brown and Caldwell conducted a preliminary and cursory evaluation of possible alignment alternatives. The results of this evaluation are as follows:

- The existing 20-foot easement for the existing land outfall is inadequate to construct a 36- to 72-inch diameter pipeline. An additional 20-foot easement would be necessary.
- The majority of the existing land outfall is located in areas along the Escondido Creek in environmentally sensitive areas.
- The majority of the existing land outfall is inaccessible in many locations by maintenance and construction crews.
- The land outfall section from the HARRF to Harmony Grove Road, especially Manhole Nos. 6 to 8, is inaccessible.
- Alternative alignment from the HARRF to Harmony Grove Road could be constructed by trenchless technology methods from the HARRF to Avenida del Diablo, then to Harmony Grove Road.
- Harmony Grove Road may be considered as an alternative alignment for the new or secondary land outfall. However, it offers the following challenges:
    - ✓ Harmony Grove Road is a narrow two-lane road. A minimum of one lane will need to be closed during construction, requiring flagmen and most likely costly night-time construction.
    - ✓ There is very little room available on either side of the road for construction staging.
    - ✓ Harmony Grove Road is significantly higher in elevation at several locations where it parallels the existing ELO. If constructed on Harmony Grove Road, the new or secondary ELO will be deep and costly to build.
    - ✓ Based on field observations, blasting may be required to build the new or secondary ELO in Harmony Grove Road.

- From the intersection of Harmony Grove Road and Via Ambiente to the intersection of Calle Messina and Via de las Flores, there are no possible alignment alternatives except to obtain additional easement along the existing easement to build the new or secondary ELO.

- From the intersection of Calle Messina and Via de las Flores, the new or secondary ELO may be located in Via de las Flores up to Aliso Canyon Road. However, Via de las Flores is a narrow two-lane road and most likely heavily congested with utilities that service the surrounding developments.

- From the intersection of Via de las Flores Road and Aliso Canyon Road, the new or secondary ELO may be located in Aliso Canyon Road up to El Camino Del Norte. Aliso Canyon Road is a highly traveled road, congested with utilities, and is within the community of Rancho Santa Fe. The construction within Rancho Sante Fe community may be very challenging and costly due to the upscale residences within the area.

- From the intersection of Aliso Canyon Road and El Camino Del Norte, the new or secondary ELO may be located in El Camino Del Norte to Manchester Avenue. Camino Del Norte Canyon Road is a highly traveled road, congested with utilities, and is within the community of Rancho Santa Fe.

- From the intersection of El Camino Del Norte and Manchester Avenue, the new or secondary ELO may be located in Manchester Avenue to South El Camino Real. Manchester Avenue is a highly traveled road, congested with utilities, and is within the city of Encinitas. In addition, Manchester Avenue is a County of San Diego road and permitting may be difficult.

- The new or secondary ELO may then follow Manchester Avenue to the San Elijo Regulator Structure where a new parallel ocean can be constructed.

In summary, constructing a new or secondary ELO would have many challenges that need to be further evaluated in detail by the City. The proposed preliminary alignment presented in this discussion is based on cursory overview of the project area without the benefit of any hydraulic evaluation, utility research, geotechnical evaluation, environmental concerns, and permitting requirements. The proposed alignment is approximately 11 percent longer than the existing alignment. A detailed alignment study taking into consideration hydraulics, existing utilities, geotechnical issues, environmental constraints, constructability, traffic control, and permitting requirements is highly recommended.

The ultimate recommendation for upgrading the land outfall will depend on the cost-effective use of equalization and live-stream discharge. For example, ELO-1 will require a combination of 'significant' equalization and live-stream discharge to limit the disposal peak flow to 25.0 mgd.

Alternatively, a new secondary pipe could be constructed to supplement the existing outfall capacity providing a total capacity capable of disposing the predicted peak weather peak flow at build-out (i.e.; ELO-5).

The option of retrofitting the existing outfall into a force main was analyzed using the hydraulic model. A peak flow of 33.0 mgd (dry weather peak flow at build-out) was routed through the existing outfall to determine the maximum hydraulic head at Manhole 0 (i.e.; the effluent discharge location at the HARRF). The hydraulic profile, as shown in Figure 5-3, reveals a significant hydraulic head in the upper reaches of the outfall resulting in a maximum head of 130 feet at Manhole 0. The feasibility of retrofitting the existing outfall will have to resolve the following challenges:

- Costly and technically challenging pipe lining required
- Potentially high velocities (+8ft/s) impacting pipe lining joints
- Need to construct and maintain a new pump station at the HARRF
- Potential hydraulic issues (i.e.; cavitations) during low flows
- Minimum 12 MG of equalization required

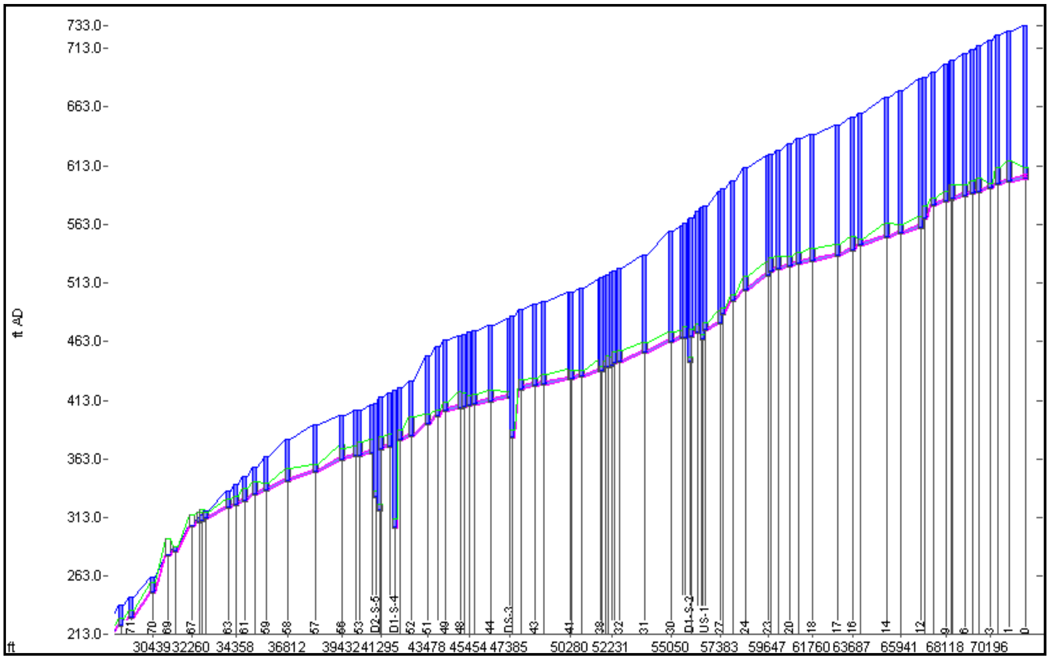


Figure 5-3. Hydraulic Profile of Existing Escondido Land Outfall with 33.0 mgd

ELO Improvement Projects and Phasing

The design and construction of large capital improvement projects, such as the new outfall, are typically conducted in phases driven by projected flow rates. Based on the analysis of the projected flows at HARRF, (see *Flow Projection Analysis TM, December 1, 2006*), the projected disposal flow at the build-out (49.0 mgd) is required by 2041. This flow includes both build-out dry weather flow and the 10-year wet weather flow.

The projected flow analysis also estimated the peak disposal flow of 27.6 mgd, which is generated from 2006 dry weather flow and the 10-year wet weather flow. This flow estimate assumes all available storage (existing plus under-construction) and live-stream discharge is utilized. The estimated peak disposal flow of 27.6 mgd exceeds the current outfall capacity of 23.7 mgd (21.4 mgd if Manhole 74 is allowed to spill). As the projected disposal flow increases approximately 0.6 mgd per year, immediate action is recommended to design and build a new outfall sized to discharge the ultimate build-out flow.

Prior to the design and construction of a new outfall (or an alternative storage facility), intermediate improvements to the outfall and operational practices of the disposal facilities are recommended. Table 5-2 lists the hydraulic and operational improvements necessary to effectively manage wet-weather flows and minimize potential non-compliant stream discharges and spills. Table 5-2 also displays the projected peak flows (based on the 10-year design wet-weather event) and the revised outfall capacities following the improvements. The largest capacity 'short-fall' (6.8 mgd) is predicted to occur in 2007 prior to any hydraulic improvements made to the existing outfall (Refer to the *Flow Projection Analysis TM, December 1, 2006* for further discussion of the projected peak flows).

Minor capacity improvements can be obtained by sealing Manhole 74, and sealing the inlet and outlet manholes for all siphon structures. Prior to conducting this work, further investigation of the pipe condition and hydraulic impact associated to these improvements is recommended.

The schedule for implementing the outfall improvement projects, as shown in Table 5-2, depicts a series of intermediate studies and projects which ultimately result in new disposal facilities (ie; new outfall or storage facilities). As the peak flow for the 10-year wet-weather event exceeds the current ELO capacity, the schedule is based on the earliest reasonable time for conducting the studies and construction projects. Prior to the estimated completion date (2010) of constructing a new outfall/storage facility, the outfall will continue to have in-sufficient capacity resulting in potential non-compliant spills. In order to alleviate these disposal constraints, the following operational improvements are recommended:

- Prepare for known capacity demands based on forecasted rainfall events
- Investigate creek flows during wet-weather events and evaluate live-stream dilution factors

Table 5-2. Recommended ELO Improvement Projects and Phasing

Phase	Description	Estimated Project Completion	Projected Peak Flow	Capacity Difference (mgd)	ELO Capacity (mgd)
Immediate	Optimize capacity management procedures	2006	27.6	-6.2	21.4
Near Term	Conduct condition assessment survey of ELO	2007	28.2	-6.8	21.4
Near Term	Seal Manhole 74	2007	28.2	-4.5	23.7
Near Term	Seal inlet and outlet manholes local to siphons	2007	28.2	-3.2	25.0
Near Term	Conduct outfall / storage alignment study	2007	28.2	-3.2	25.0
Near Term	Design new outfall / storage facilities	2008	28.8	-3.8	25.0
Long-Term	Construct new outfall / storage facilities	2010	30.1	18.9	49.0

5.3.2 San Elijo Ocean Outfall (SEOO) Options

Future growth in the area served by the HARRF will increase the average daily flow from an average annual daily flow of 15.3 mgd recorded in 2005 to 27.5 mgd. Build-out PWWF from the HARRF sewer service area is expected to be about 53.4 mgd. Combining this expected peak flow from the HARRF with the PWWF assumed for the SEWRF service area of 10.5 (5.25 * 2) mgd yields a total potential peak ocean discharge of about 64.0 mgd (depending on the peaking factor for the SEWRF service area). Subtracting the intermittent live-stream discharge flow allowed at the HARRF of 9.0 mgd (assuming improvements are made to the tertiary system), the ocean outfall must be capable of conveying at least 55.0 mgd. If the outfall is to be expanded and the allowable disposal split between the HARRF and SEWRF remains at 79 and 21 percent, respectively, the ocean outfall must be expanded to about 58.6 mgd (53.4/0.79 – 9).

The City has two principal options for increasing the hydraulic capacity for the discharge system: 1) make incremental changes such as paralleling the existing offshore and onshore sections; or 2) construct a new parallel outfall. The former approach could achieve a hydraulic capacity increase to about 35.0 mgd without constructing through the surf zone. With parallel construction through the surf zone, the overall capacity could be increased to the required capacity (combined discharge from the City and San Elijo JPA or separate discharge from the City).

The best apparent alternative would be to construct a parallel outfall with a diffuser located in deeper water, but on the same compass heading as the existing outfall. This approach should minimize overall costs for design, construction, and construction management. It would also require facing regulatory and public review only once. Since portions of the SEOO system would be approaching the end of their useful life at the end of the planning period, construction of full parallel capacity would be the most prudent approach for planning purposes. To assess the condition of the existing SEOO and determine its remaining useful life, forensic investigation of both the onshore and offshore sections, especially the asbestos cement pipe laid through the wetland, would be required. If the condition of the existing system is suitable for at least 50 years of additional service, then a phased approach or construction of the parallel system to carry the incremental flow above 25.8 mgd would cost less. Any new construction would require significant permitting approvals.

The alternatives considered for the ocean outfall are briefly described below and the capacities summarized in Table 5-3. For this study, two ultimate ocean outfall capacities were considered: 58.0 and 48.0 mgd. Each selected capacity is closely tied to various disposal alternatives reviewed for the city of Escondido. Detailed discussion is presented in the technical memorandum included as Appendix J.

Option SEOO-1: Phased Expansion

- Phase I
  - ✓ Parallel the existing 30-inch-diameter pipe beyond the surf zone (Station 15+00 to Station 40+00) with a 2,500 feet long, 54-inch-diameter pipe
  - ✓ Extend the diffuser section by about 500 feet into deeper water
- Phase II
  - ✓ Parallel or replace the land section of the SEOO (from the Regulator Structure to the beach)
  - ✓ Use a 42-inch-diameter pipe in parallel
  - ✓ Extend the diffuser section by an additional 700 feet for 48.0 mgd or 1,200 feet for 58.0 mgd, into deeper water
  - ✓ If the condition and durability for the existing 30-inch-diameter pipe through the surf zone are a concern, then replace this segment with a 48-inch- (for 48 mgd) or 54-inch- (for 58-mgd) diameter pipe.

Option SEOO-2: Parallel Existing SEOO with a New System

Build a completely new 30-36 inch- (for 48 mgd) or 42-inch- (for 58 mgd) diameter parallel outfall and diffuser with a hydraulic capacity to accommodate City and San Elijo flows in excess of the existing outfall hydraulic capacity.

Option SEOO-3: Replace Existing SEOO with a New System

Build a completely new 48-inch- (for 48 mgd) or 54-inch- (for 58 mgd) diameter parallel outfall and diffuser with a hydraulic capacity to accommodate the combined build-out flow.

Table 5-3. Ocean Outfall Improvement Options

Option ID	Description	Capacity Examined (mgd)
SEOO-1	Phased Expansion	35 Initial 48 or 58 Final
SEOO-2	Parallel Existing SEOO with a New 30-, 36-, and 42-inch-diameter System	48 or 58
SEOO-3	Replace Existing SEOO with a new 48-, and 54-inch-diameter System	48 or 58
SEOO-4	Construct New 42-, and 48-inch-diameter System to Convey Escondido Flows Only	33 to 49

Alternative SEOO-1 includes the risks associated with the condition and longevity of the existing system. It would also require permitting and offshore construction on separate occasions. Alternative SEOO-3 would provide a new system with build-out capacity for both communities. The overall ocean discharge permitting requirements and potential impacts are virtually the same for all alternatives since the dilution performance and construction impacts would be essentially the same.

Option SEOO-4: Construct a Ocean Outfall for Conveyance of Only Escondido Flows

Build a completely new 42-inch- (for 33 and 38 mgd) or 48-inch- (for 45 and 49 mgd) diameter ocean outfall and diffuser to convey and discharge flows solely from the City. The new Escondido Ocean Outfall will parallel the existing SEOO, along similar alignment as the SEOO-2 and SEOO-3 alternatives. The ultimate capacity and size of the ocean outfall will depend on the disposal option selected (described later in this section).

Construction Phasing of the SEOO

The SEOO has maximum hydraulic capacity of 25.8 mgd. The attenuated 10-year design peak wet-weather flow effluent flow from HARRF (27.6 mgd) combined with the maximum allowable peak flow discharged from SEJPA (5.3 mgd) exceeds the current capacity of the SEOO. Therefore similar to the ELO, it is recommended immediate steps are taken to expand or replace the existing SEOO based on the options outlined in Table 5-3.

The peak flow currently received at the SEOO is hydraulically limited by the upstream ELO capacity (21.4 mgd) and dependent on the operation of the SEJPA regulator structure. As the ELO capacity is limited by the SEOO (via the regulator structure) construction of a new ELO must be conducted in parallel with the expansion of the SEOO. Depending on the SEOO expansion options described in this report, the following key activities will need to be conducted in order to expand or replace the SEOO:

- Apply and obtain permits to construct both off and on-shore segments of the new SEOO (3-4 years)
- Conduct basis-of-design studies including (1- 2 year)
  - Geotechnical investigations
  - Alignment study
  - Capacity re-evaluation study
  - Environmental impact study
  - Pipeline pre-design
- Design on/off-shore outfall pipeline (1 year)
- Construct on/off-shore outfall (2 years)

5.3.3 Equalization Options

The HARRF comprises of existing secondary effluent and reclaimed water equalization facilities with additional facilities currently under construction. Upon completion of the new equalization construction project, the HARRF will be able to store 4.0 MG of secondary effluent and 3.0 MG of reclaimed water. The basis of providing equalization attenuates the peak flows hence reducing the peak flows disposed through the outfall.

A series of equalization options, listed in Table 5-4, were evaluated in conjunction with the outfall upgrades and live-stream discharge scenarios. The recommended equalization option will be based on the cost-effective choice of upgrading the outfall and/or live-stream discharge. For example, the first option (EQU-1), recommends building an additional 2.0 MG in addition to upgrading the outfall and increasing the live-stream discharge rate. Further details of this and other outfall, equalization, and live-stream discharge combinations are described later in this section.

Table 5-4. Equalization Improvement Options

Option ID	Description	Total Capacity (MG)	Notes
EQU-1	New SE equalization	2.0	Wet weather storage
EQU-2	New SE equalization	3.5	Dry weather storage
EQU-3	New SE equalization	12.0	Wet weather storage
EQU-4	New SE equalization	14.0	Wet weather storage
EQU-5	New SE equalization	25.0	Wet weather storage

The location and construction aspects of the new equalization basins will depend on the availability of land adjacent to the HARRF, geological conditions determining the cost and effort of positioning the basin below grade, and the method of filling and draining the basin, such as pumping. For example, two 7.0-MG basins with one replacing the existing equalization basin and the other constructed outside the plant boundary will provide 14.0MG of storage. However, this option will require the resolution of various land acquisition issues such as cost, environmental, etc. In order to mitigate land acquisition for one of the 7-MG basins, the existing equalization basin site could be used to locate a new storage tank (see Figure 5-4).

5.3.4 Live-stream Discharge Options

Three live-stream discharge options listed in Table 5-5 were evaluated. Similar to the outfall and equalization options, the three live-stream discharge options are tied closely to various combinations of outfall expansion and additional equalization which are described later in this report. LSQ-1 allows the currently permitted discharge of 9.0 mgd into the creek (assuming all conditions prescribed in the permit are met). Additional improvements will be required to the existing reclaimed water plant to achieve 9.0 mgd. The 'extreme' live-stream discharge option LSQ-3, disposing 20.0 mgd into the creek, was evaluated as an alternative to no equalization and an outfall capacity of 33.0 mgd.

The viability of conducting live-stream discharges is the knowledge, and proof, of the creek flows during stream discharge periods. Discharging during low creek flows (i.e.; below the permitted flow range) will violate the discharge permit. While the evaluation of creek flows during rainfall events (i.e.; periods when live-stream discharges are most likely required) is beyond the scope of this study, the equivalent return periods for the 2005 rainfall events with compliant live-stream discharges were evaluated and compared to the 10-year design storm event.

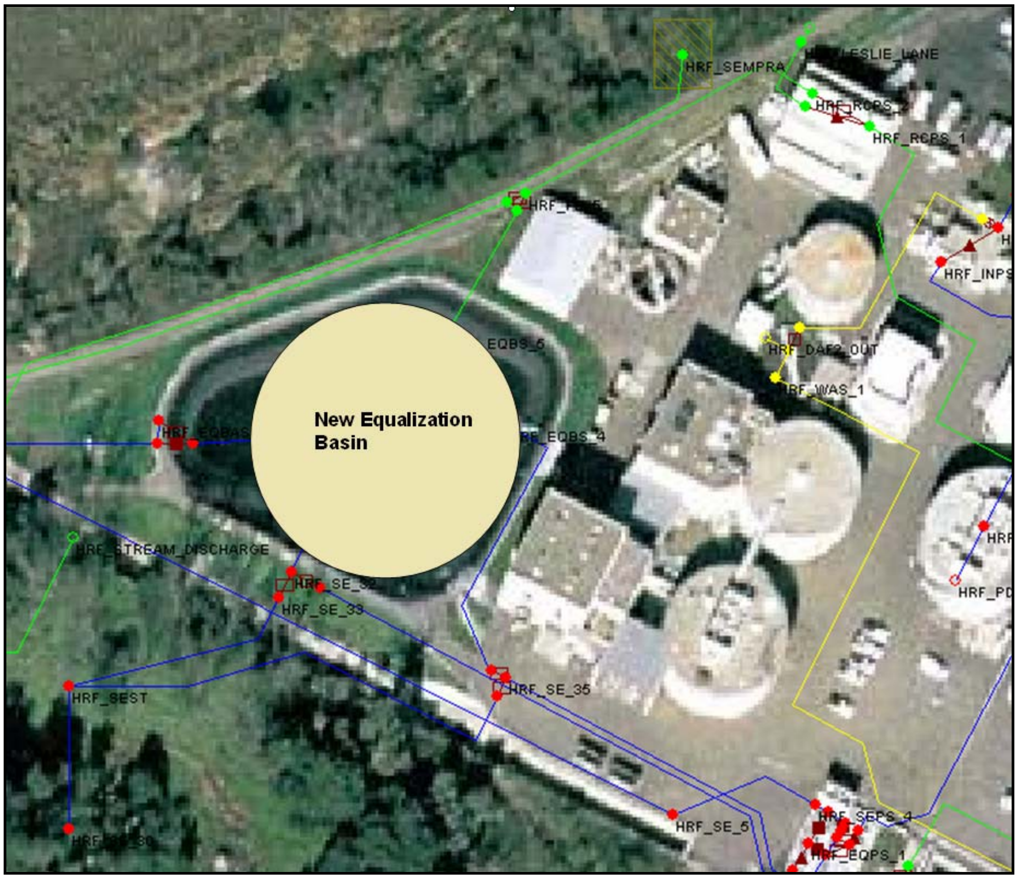


Figure 5-4. Proposed Site of New Equalization Basin

Table 5-5. Live-Stream Discharge Options

Option ID	Description	Capacity (mgd)	Notes
LSQ-1	Current permitted live-stream discharge	9.0	Existing Recycled Water plant improvements
LSQ-2	Proposed LSQ–1 Plus 6 mgd Additional Capacity	15.0	Additional Recycled Water plant capacity
LSQ-3	Proposed LSQ–1 Plus 11 mgd Additional Capacity	20.0	Additional Recycled Water plant capacity

2005 Live-stream Discharge Events

- January 10, 2005 storm – 1.5-year return period
- February 22, 2005 storm – 1.0-year return period

The relatively small return period events (derived by ranking the peak flows) as compared to the 10-year design storm suggest the creek flows present during the 10-year event will provide sufficient dilution to allow compliant live-stream discharges.

Live-stream Discharge Treatment Options

The live-stream discharge disposal scenarios identified in Table 5-5 were evaluated in terms of treatment requirements. The analysis used the total volume discharged to the creek predicted for the 10-year design event for each stream discharge scenario as listed in Table 5-6.

Table 5-6. Live-Stream Discharge Disposal Options

Alternative	Disposal Flow (mgd)	Total Discharge per Event (MG)
LSQ-1	9.0	18.1
LSQ-2	15.0	44.3
LSQ-3	20.0	65.9

When the HARRF discharges to the creek, the mass loading of nitrogen and phosphorus is determined. The plant is required to remove the equivalent mass loading (resulting from the discharge) from the creek at some time within that year. Therefore, there is a benefit to discharging an effluent low in nutrients. Two viable treatment alternatives were analyzed for the HARRF for the future condition. Table 5-7 summarizes the equivalent mass discharge of nitrogen and phosphorus determined from the BioWin™ model simulations under average flow and loading conditions.

Table 5-7. Estimated Nutrient Loading for Live-stream Discharge for Each Viable Treatment Alternative

Treatment Alternative	Total Nitrogen Discharge (lb-N/d)	Total Phosphorus Discharge (lb-P/d)	Nitrogen Concentration (lb-N/MG)	Phosphorus Concentration (lb-P/MG)
3A/3B	7,980	160	290	5.82
3C	6,240	345	227	12.5
6A/6B	7,980	610	290	22.2

Treatment Alternative 3C has the lowest estimated nitrogen concentration because the MBR will provide nitrogen removal. Treatment Alternatives 3A-3C have the lowest phosphorus discharge because of the presence of the anaerobic selector. For optimal selector performance to control filamentous bulking, an aerobic SRT of 2.0 days is recommended which will promote biological phosphorus removal. However, at this SRT, nitrification will not occur and there will be no nitrogen reduction for Alternatives 3A and 3B; Alternative 3C will provide some nitrogen removal because the MBR portion of the plant will achieve nitrogen removal. Alternatives 6A and 6B have the highest phosphorus discharge because there is no anaerobic selector for biological phosphorus removal.

Table 5-8 summarizes the nitrogen and phosphorus loading resulting from live-stream discharge for each disposal option. There is a concern that the amount of available nutrients in the stream during dry weather periods will not be sufficient to compensate for the total mass loading to the creek during the wet weather season. This fact will force the City to implement advanced treatment (such as RO) to remove the nitrogen and phosphorus. Since the amount of nutrients in the stream depends on the land improvements and irrigation and landscape practices along the Escondido Creek alignment upstream of the HARRF, Brown and Caldwell was unable to predict the limits of intermittent live-stream discharge that would preclude the construction of advanced treatment facilities.

Table 5-8. Nitrogen and Phosphorus Loading Resulting from Live-Stream Discharge

	Total Nitrogen Discharge per Event (lb-N)			Total Phosphorus Discharge per Event (lb-P)		
Treatment Alternative	Disposal Option D	Disposal Option E	Disposal Option F	Disposal Option D	Disposal Option E	Disposal Option F
3A and 3B	5,260	12,860	19,130	110	260	390
3C	4,110	10,060	14,960	230	560	830
6A and 6B	2,350	5,760	8,560	410	990	1,470

5.4 Outfall – Equalization – Live-stream Discharge Scenarios

The combined hydraulic use of the outfall, equalization, and live-stream discharge facilities were evaluated with the aim of identifying an optimum cost-effective scenario for disposing of the design flows. Table 5-9 summarizes the improvement. Note that the proposed outfall pipe sizes listed in Table 5-9 are for a new independent outfall.

Scenario A

The first scenario involves building a new outfall to discharge the peak design flow of 49.0 mgd. Pipe sizes for a new outfall will range from 54 to 72 inches in diameter depending on the alignment and profile. This scenario assumes no additional equalization and no live-stream discharges are required.

One alternative to building a new outfall is to construct a 42-inch-diameter secondary outfall supplementing the capacity of the existing outfall. This alternative would include the rehabilitation of the existing outfall. In addition, a secondary outfall will provide redundancy allowing one outfall to be shut down for operational, maintenance, or emergency repair needs. Again this scenario assumes no additional equalization and no live-stream discharges are required.

Scenario B

The second scenario involves building equalization facilities to store a total 28.5 MG of secondary effluent. The equalization facilities will compose of two to four separate tanks located at or near the HARRF, based on land availability and construction limitations. This scenario assumes minor improvements are made to the existing outfall to increase the capacity to 25.0 mgd. As this scenario requires equalizing both dry and wet weather flows, hence increasing the risk of non-compliant spills to the creek during periods of extended rainfall, this scenario was eliminated from further consideration.

Scenarios C, D, E, F, and G

These scenarios involve a combination of outfall, equalization, and live-stream discharge improvements. All of these scenarios will require either building a new outfall or building a secondary outfall supplementing the existing outfall capacity. The size (and cost) of the outfall will be reduced if additional equalization and live-stream discharge facilities are built at the HARRF. However, the cost of building these facilities may outweigh the cost of building a new outfall sized to carry the total peak design flow (Scenario A/H).

Scenarios C, D, and E depict the additional equalization needs (i.e.; not including the existing 7.0-MG equalization facilities) given an outfall capacity equivalent to the peak dry weather flow at build-out (i.e.; 33.0 mgd). In this case, Scenario C reveals the 'extreme' scenario where no live-stream discharge is required resulting in the need to store 25.0 MG. Similar to the practical limitations for Scenario B, as discussed above, this option is not considered viable and is only presented in this report to demonstrate the 'worst-case' scenario. Alternatively, Scenario D indicates that a 12.0-MG equalization basin is required while discharging the current permitted 9.0 mgd to the creek. If the live-stream discharge was further increased to 15.0 mgd (Scenario E), the equalization requirements are dramatically reduce to an additional 2.0 MG.

Scenarios F and G show the required outfall capacities with no additional equalization and live-stream discharges of 20.0, 15.0 and 9.0 mgd respectively. These scenarios show the relationship between live-stream discharge and outfall capacities; i.e., as live-stream discharge increases the required outfall capacity reduces.

Scenario H

Finally, Scenario H involves building a new outfall to discharge the peak flow design flow of 45.0 mgd while allowing the currently permitted live-stream discharge of 9.0 mgd. Pipe sizes for a new outfall will range from 54 to 72 inches in diameter depending on the alignment and profile. The solution only requires relatively minor modifications to the reclaimed water facility to ensure 9.0 mgd is available for live-stream discharge with no additional equalization.

Table 5-9. Outfall/Equalization/Live-Stream Discharge Scenarios

Scenario ID	Total Land Outfall Capacity (mgd)	New Land Outfall Size (inch)	Equalization Capacity (MG)	Live-Stream Discharge (MGD)	Facility Improvements	Notes
A	49.0	72	0.0	0.0	Outfall	1,6,7
B	25.0	N/A	28.5	0.0	Outfall (minor) + Equalization	2,3
C	33.0	42	25.0	0.0	Outfall + equalization	4,7
D	33.0	42	12.0	9.0	Outfall + equalization + live-stream-discharge	4,6,7
E	33.0	42	2.0	15.0	Outfall + equalization + live-stream-discharge	4,6,7
F	33.0	42	0.0	20.0	Outfall + live-stream-discharge	4,6,7
G	38.0	54	0.0	15.0	Outfall + live-stream-discharge	5,6,7
H	45.0	72	0.0	9.0	Outfall + live-stream-discharge	5,6,7

Notes:
Shaded boxes indicate most viable scenarios.
1. Land outfall upgraded by constructing a new outfall providing a total capacity of 49.0 mgd.
2. Capacity of existing land outfall increased to 25.0 mgd by sealing siphon manholes.
3. Equalization capacity includes 3.5 MG required to attenuate dry weather flows.
4. Outfall capacity designed to dispose of peak build-out dry weather flow (33.0 mgd).
5. Outfall capacities derived from peak flows reduced due to live-stream discharges.
6. Viable options accounting for equalization construction / site requirements.
7. New land outfall size will vary based on profile determined from alignment study.

6.0 COST ESTIMATE FOR RECOMMENDED IMPROVEMENTS

Brown and Caldwell was requested to develop order-of-magnitude construction cost estimates of the recommended improvements. Contained in this section are planning-level construction cost estimates with an accuracy of -35 and +50 percent. Developing operation and maintenance costs was not part of the scope of the study.

6.1 Cost Analysis of the HARRF Improvements

Near-Tem Improvements

The planning-level cost estimate for improvements that will ensure the capacity of the HARRF to 18.0 mgd average daily flow is estimated as $3.9 million. This cost includes only the cost related to the following improvements;

- Providing the equipment to enable chemical injection at the primary clarifiers,
- Providing two on-site oxygen gas generators, and oxygen dissolution aeration equipment to optimize oxygen use, and
- Rental cost for two 200 gpm-capacity centrifuges to be used as recuperative thickening of the digested sludge. It is assumed that the City will need to rent the centrifuges three times until the construction of Phase 3. Each rental period is assumed to be a month.

Currently, the City has the facilities in place to provide polymer injection at the splitter box and chlorinate the RAS line. Therefore, these improvements will not generate cost. Costs related to other solid process improvements are not included here. A detailed cost estimate is provided in Appendix L.

Long-Term Improvements

Planning-level cost estimates were also prepared for alternatives that can reliably treat 27.5 mgd average flow. The cost estimates incorporate the recommended process units and treatment identified for the alternatives described in Table 4-3. Cost estimates for improvement Alternatives 3 and 6 are presented in Table 6-1 for two sludge process options; co-thickening and separate sludge thickening.

The estimate illustrates the reduced cost associated with co-thickening. The higher cost of larger DAFT units needed to accommodate both the primary and biological solids for the co-thickening alternative is largely offset by the tremendous cost savings related to the construction of smaller digesters required for co-thickening. Smaller digesters are needed because thicker sludge is produced. Solids concentration up to 6.0 to 6.5 percent total solids has been achieved (compared to the current combined solids concentration of approximately 4 percent at the HARRF). More details on the differences between co-thickening and separate thickening are presented in Appendix G.

It should be noted that the estimate presented in Table 6-1 includes the installation of a new odor-control system to treat foul air from covered primary clarifiers (including the new, fifth primary clarifier) and the headworks. The cost presented includes a new single-stage activated carbon system (carbon, vessel, and fans), covers for the clarifiers, and the necessary ducting.

Construction cost of all the improvement alternatives were found close to each other. The cost difference between the highest cost alternative (Alternative 6B with sludge co-thickening) and the lowest cost alternative (Alternative 3A with sludge co-thickening) is about 20 percent.

Among these alternatives, 3A, 3B, 6A and 6B are considered the most practical (see Appendix H for more details on the comparison). Pilot testing of the nitrifying BAF (sub-alternative A), membrane filtration (sub-alternative B), or MBBR (Alternative 6) is highly recommended to determine the performance and design criteria for these systems at local conditions.

Although, developing operation and maintenance costs was not part of the scope of the study, annual operational and maintenance cost of sub-alternatives A, B, and C were estimated to understand the relative difference in their operation cost. Operational and maintenance cost of the 9.0 mgd BAF system is expected to be $620,000, which is 10 to 50 percent lower than the 9.0 mgd membrane filtration and the 9.0 mgd membrane bioreactor, respectively.

Table 6-1. Planning-Level Project Cost for Build-out Alternatives – Average Flow of 27.5 mgd (in 2006 End-of-Calendar Year Dollars)

Treatment Alternative	Description	Sludge Thickening Option	Total Project Cost ($Million)
3A	High Rate CAS with CEPT and BAF for 9.0 mgd	Co-thickening	**119**
		Separate	138
3B	High Rate CAS with CEPT and Micro filtration for 9.0 mgd	Co-thickening	**137**
		Separate	156
3C	High Rate CAS with CEPT and MBR for 9.0 mgd	Co-thickening	**139**
		Separate	166
6A	MBBR with BAF for 9.0 mgd	Co-thickening	**126**
		Separate	145
6B	MBBR with Micro filtration for 9.0 mgd	Co-thickening	**144**
		Separate	163

Main Assumptions:

- Planning-level estimates: -35 percent to +50 percent accuracy
- Costs includes all necessary earthwork, grading and sheeting/shoring necessary
- Electrical/Instrumentation included as 22 percent of mechanical, piping and building cost
- Construction Contingency = 40 percent of raw construction cost
- Contractor overhead and profit and general conditions = 22 percent of raw construction cost
- Misc. Markups (tax, material shipping and handling, travel/subsistence, etc.) = ~16 percent of raw construction cost
- Engineering = 20 percent of total capital cost
- SCADA = 10 percent of total capital cost
- Construction Management = 10 percent of total capital cost
- Legal and Admin = 10 percent of total capital cost

Phasing of the improvement to the ultimate capacity of 27.5 mgd will involve additional cost mainly associated with mobilization and demobilization, and the difficulties encountered when having to construct while keeping existing process units in service. Planning-level cost estimates for each phase of the secondary and tertiary treatment capacity improvements are listed in Table 6-2. Phasing is anticipated to add 3-5 percent to the costs reported in Table 6-1. As mentioned in Section 4.4, Alternative 3B (with sludge co-thickening option) is selected for the phasing discussion. The reported costs are in 2006 dollar values. Escalation to midpoint is not considered between phases since the exact time of construction is not known. However, the City should apply at least 8 percent escalation per year to account the uncertainty in pricing of building construction materials.

Table 6-2. Planning-Level Project Cost for Alternative 3B – Phased Construction (in 2006 End-of-Calendar Year Dollars)

Construction Phase	Total Project Cost ($Million)
Improvements for Expansion of Secondary Treatment Capacity	
Phase 3	75
Phase 4	11
Phase 5	3
Improvements for Expansion of Tertiary Treatment Capacity	
Phase 3	52

6.2 Cost Analysis of the Outfall Improvements

The near-term improvements to increase the capacity of the existing ELO are estimated to cost $5.8 million, which includes conducting a closed circuit television inspection of the entire length of the existing land outfall, and sealing Manhole 74 and the inlet and outlet manholes for all the siphon structures (total of 12 manholes).

Investigation of the ELO pipe condition might require either rehabilitation of the existing pipeline and constructing a storage facility or abandoning the ELO and constructing a new land outfall. Long-term ELO improvements assume that a new land outfall will be constructed to convey all the flows from the HARRF requiring ocean disposal. Cost of constructing the new land outfall is provided in Section 6.3 for different effluent disposal options. The presented costs do not include the cost to abandon or refurbish the existing ELO. The City must add the pertinent cost to the alternatives.

- Rehabilitation is estimated to cost about $27 million for the following:
  - ✓ Closed circuit television (CCTV) inspection of the entire length of the existing ELO,

  - ✓ Relining of 5 miles of the existing pipe,
  - ✓ 70 days of by-pass pumping,
  - ✓ Follow-up CCTV of the relined pipe, and
  - ✓ Rehabilitation of 53 manholes.

- Abandonment of the ELO is estimated to cost about $3 million for the following:

  - ✓ Filling of the existing ELO with sand
  - ✓ Removing the tops of manholes and filling them with slurry

Cost for environmental mitigation and monitoring are assumed to be $500,000 for abandonment and rehabilitation of the existing ELO. Detailed cost estimate is presented in Appendix L.

Construction cost of the new ocean outfall is provided in Section 6.3 for different effluent disposal options.

6.3 Cost Analysis of Effluent Disposal Options for Build-out Conditions

Presented in this sub-section are the costs related to disposing of the effluents produced at the HARRF. The planning-level costs shown in Table 6-3 include costs related to upgrading the HARRF to produce effluents (secondary or tertiary) suitable for the respective disposal strategy, upgrading the system to equalize dry weather or wet weather flows to ensure that the capacity of the outfall or other disposal means are not exceeded, and upgrading the land and/or ocean outfall to suit the disposal strategy. Land acquisition costs for construction of the facilities have not been included to the improvement costs listed in Table 6-3. This might increase the presented costs accordingly.

For all disposal scenarios, the cost for the HARRF improvements assumes that Alternative 3B is implemented. This is to facilitate the comparison between the disposal alternatives. Selection of a different treatment alternative for the HARRF will lessen or increase the total cost shown accordingly.

Comparatively, Scenario A had the lowest cost ($450 million) and Scenario H was the most expensive ($548 million). Detailed breakdown of the cost estimate is presented in Appendix L. Discussion of the key assumptions made in developing the costs for each scenario is presented below.

Table 6-3. Planning-Level Cost Estimate for Viable Effluent Disposal Scenarios (in 2006 End-of-Calendar Year Dollars)

Scenario	Summary of Improvements	Cost ($Million)	Total Cost ($Million)
A	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity using membrane filtration	137	450
	ELO Improvements: ▪ New 72-inch-diameter pipeline with 49.0 mgd capacity	233	
	SEOO Improvements: ▪ New 54-inch-diameter ocean outfall with 62.0 mgd capacity	80	
D	HARRF Improvements: ▪ 27.5 mgd secondary diameter ocean outfall with 62.0 mgd capacity	137	481
	Live-Stream Discharge: ▪ Membrane filtration with 9.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis with 9.0 mgd effluent capacity	98	
	Flow Equalization: ▪ Demolition of the existing 2 MG equalization basins ▪ Two 7 MG equalization basins construction	33	
	ELO Improvements: ▪ New 42-inch-diameter pipeline with 33.0 mgd capacity	142	
	SEOO Improvements: ▪ New 48-inch-diameter ocean outfall with 42.0 mgd capacity	71	
E	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	506
	Live-Stream Discharge: ▪ Membrane filtration with 15.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis with 15.0 mgd effluent production capacity	152	
	Flow Equalization: ▪ One 2 MG equalization basin construction	4	
	ELO Improvements: ▪ New 42-inch-diameter pipeline with 33.0 mgd capacity	142	
	SEOO Improvements: ▪ New 48-inch-diameter ocean outfall with 42.0 mgd capacity	71	
F	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	533
	Live-Stream Discharge: ▪ Membrane filtration with 20.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis for 20.0 mgd effluent production capacity	183	
	ELO Improvements: ▪ New 42-inch-diameter pipeline with 33.0 mgd capacity	142	
	SEOO Improvements: ▪ New 48-inch-diameter ocean outfall with 42.0 mgd capacity	71	

Table 6-3. Planning-Level Cost Estimate for Viable Effluent Disposal Scenarios (in 2006 End-of-Calendar Year Dollars)

Scenario	Summary of Improvements	Cost ($Million)	Total Cost ($Million)
G	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	530
	Live-Stream Discharge: ▪ Membrane filtration with 15.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis for 15.0 mgd effluent production capacity	152	
	ELO Improvements: ▪ New 54-inch-diameter pipeline with 38.0 mgd capacity	170	
	SEOO Improvements: ▪ New 48-inch-diameter ocean outfall with 48.0 mgd capacity	71	
H	HARRF Improvements: ▪ 27.5 mgd secondary and 9.0 mgd tertiary treatment capacity	137	548
	Live-Stream Discharge: ▪ Membrane filtration with 9.0 mgd tertiary effluent production capacity ▪ Chlorination and dechlorination ▪ Reverse osmosis with 9.0 mgd effluent production capacity	98	
	ELO Improvements: ▪ New 72-inch-diameter pipeline with 45.0 mgd capacity	233	
	SEOO Improvements: ▪ New 54-inch-diameter ocean outfall with 58.0 mgd capacity	80	

Note: Description of the discharge scenarios included here is given in Table 5-9

6.3.1 General Assumptions

- The reported costs are planning-level estimates with -35 percent to +50 percent accuracy.
- Costs includes all necessary earthwork, grading, and sheeting/shoring necessary
- Costs for alternatives requiring placement of the facilities outside of the current plant boundaries do not include land and/or easement acquisition costs. Furthermore, costs for ELO improvements requiring a secondary outfall do not include land and/or easement acquisition cost as well.

HARRF Improvements

- HARRF Improvement Alternative 3B was selected because it represents a mid-point cost of the three alternatives. Selection of Alternative 3A would result in a $18 million reduction in the overall cost. Selection of Alternative 3C would result in a $2 to $10 million increase.
- The HARRF improvement cost reported assumes co-thickening will be practiced. Implementing improvements to allow separate thickening will increase the cost

reported in Table 6-2 by $19 to $27 million, depending on the improvement alternative selected.

ELO Improvements

- Costs reported for ELO improvements assume that a new land outfall will be constructed to convey all the flows from the HARRF requiring ocean disposal. They do not include the cost to abandon or refurbish the existing ELO. The City must add the pertinent cost to the alternatives as indicated in Section 6.2.
- Cost for environmental mitigation and monitoring are assumed to be $2 million for construction of the new land outfall.
- The length of the new land outfall is assumed to be equal to 1.15 times the total length of the existing outfall (from the HARRF to the Regulator Structure). Two alignments were investigated (one described in Section 5). The longest route was found to be about 11 percent longer than the existing alignment. The longer length assumed for the new land outfall should cover any deviations from the alignments examined.

SEOO Improvements

- SEOO improvements assume that a new parallel ocean outfall will be constructed and that the 79:21 capacity ratio between the City and the San Elijo JPA is maintained in the future. The cost of downsizing to the next lower size (a reduction of 6 inches in diameter) to accommodate only Escondido flows results in a decrease in installed cost of between $2 to $9 million.

Live-stream Discharge

- The cost for RO is included in some of the disposal options that require intermittent live-stream discharge. Based on the current permit allowing intermittent discharge, the City is required to redirect Escondido Creek flows back to the HARRF during the dry weather periods to recover the amount of nutrients (i.e., nitrogen [N] and phosphorus [P]) discharged to the creek during the wet weather season. There is a concern that the varying quality of the Escondido Creek will not yield sufficient amount of nutrients to recover the quantities discharged to the creek. Therefore, provisions are included to remove N and P in the tertiary effluent. RO is selected because it can easily be "switched on" when needed without the need for acclimation. Pilot testing should be conducted to determine the effectiveness of RO in removing N under local conditions.

Flow Equalization

- Improvements noted in Table 6-3 are in addition to the 7.0 MG equalization storage basins already in place.

Soft Costs – HARRF Improvements

- Assumed soft costs for the HARRF and the outfall improvements are presented in Table 6-4. The difference between the assumed soft costs between the HARRF and outfall improvement is explained under the remarks section of the table.

Table 6-4. Soft Costs for the HARRF and Outfall Improvements

Item	HARRF Improvements	Outfall Improvements	Remarks
Electrical and Instrumentation	22 percent of mechanical, piping, and building cost	None	Outfall improvements does not require any electrical and instrumentation component
Construction Contingency	40 percent of raw construction cost	50 percent of raw construction cost	Currently there are more unknowns for the outfall improvements such as the geotechnical information
Contractor Overhead and Profit and General Conditions	22 percent of raw construction cost	17 percent of raw construction cost	Additional cost for construction equipment, trailers, and temporary utilities related to the HARRF improvements.
Escalation to End of 2006 Calendar Year	8 percent of raw construction cost	8 percent of raw construction cost	–
Miscellaneous Markups	16 percent of raw construction cost	16 percent of raw construction cost	–
Engineering	20 percent of total capital cost	15 percent of total capital cost	Less engineering time and disciplines are needed for the outfall improvement project
SCADA	10 percent of total capital cost	None	No SCADA system is needed for the outfall improvement
Construction Management	10 percent of total capital cost	6 percent of total capital cost	More construction management, i.e. additional inspection services, is needed for the HARRF improvements due to complexity of the construction.
Legal and Administration	10 percent of total capital cost	10 percent of total capital cost	–

6.3.2 Outfall Scenario-Specific Assumptions

Assumptions that are not covered above, but are important in developing the cost for the outfall scenarios, are discussed below.

Scenario D

- It is assumed that the existing 2-million-gallon equalization basin will be demolished and two new 7-million-gallon reservoirs will be constructed: one placed in the same location as the existing equalization basin and the other constructed outside of the current plant boundaries. Land acquisition cost is not included.

Scenario E

- An additional 2-million-gallon reservoir will be constructed outside of the current plant boundaries. Land acquisition cost is not included.

Scenario F and G

- The additional 9.0 mgd membrane filters are assumed to be located on the parking lot south of the existing tertiary filters. The remaining additional 6.0 mgd membrane filters and 15.0 mgd chlorine contact basins and RO system are assumed to be built outside of the current plant boundaries. Land acquisition cost is not included.

Scenario H

- The additional 9.0 mgd membrane filters are assumed to be located on the parking lot south of the existing tertiary filters. The remaining additional 11.0 mgd membrane filters and 20.0 mgd chlorine contact basins and the RO system are assumed to be built outside of the current plant boundaries. Land acquisition cost is not included.

7.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of the conclusions and recommendations from the various study elements are provided in this section.

7.1 Conclusions

7.1.1 HARRF

Bar screens

- The two bar screens installed should be capable of handling the build-out flows. However, a standby mechanical screen is required to provide flexibility.

Influent Pump Station

- Based on the latest assessment in the *2004 Capacity Rerating Study*, the existing pump station has a peak capacity of 43.5 mgd. This is 4.7 mgd short of the ultimate (build-out) peak wet weather flow expected of 48.2 mgd.

- Given the age of the existing equipment, a comprehensive evaluation of the remaining useful life of the equipment (including gates, operators, valves, etc.) must be conducted to determine the need for upgrades in system components or replacement of equipment.

Grit Removal

As currently configured, the grit removal system does not have sufficient capacity for build-out conditions, but may have the necessary capacity to treat 18.0 mgd. The reliable capacity is likely between 14.5 and 21.0 mgd average flow, but additional testing and verification are needed. In addition, implementing an air scour system will likely ensure that the capacity is at the upper end of the range.

Primary Treatment

- The primary clarifiers have sufficient hydraulic capacity for the plant at the rated flow of 18.0 mgd.

- Field observations also showed that there is flow imbalance in the flow splitting between primary clarifiers, with more flow going to Clarifier 4 (the eastern-most clarifier). In spite of the flow imbalance, the clarifiers demonstrated similar to TSS and COD removal with all clarifiers in service.

- The HARRF primary clarifiers still remove 46 percent of the influent solids and 26 percent of the influent $cBOD_5$ at a SOR of 1,400 gpd/ft^2. In addition, the maximum possible suspended solids removal (i.e., nearly 100-percent removal of primary influent settleable solids) is achieved under average flow conditions, between 800 and 849 gpd/ft^2.

Secondary Treatment

- There is unequal flow distribution between aeration basins that is attributed to uneven RAS flows and/or primary effluent flows to individual basin, and the reported RAS flows are incorrect.
- The plant is not completely nitrifying, possibly due to one or a combination of the following: elevated RAS chlorination, high ammonia loads in solids recycle streams, and/or low DO concentrations in the aeration basins.
- Historic SVI data were analyzed to assess sludge settleability. The 90th percentile SVI value was determined to be 203 mL/g.
- The wet weather capacity of the plant was found to govern plant capacity. Wet weather capacity, expressed in terms of ADWF, was determined to be 14.8 mgd with all units in service at the current operation conditions.
- Off-gas testing of the aeration system showed that the current αF factor is 0.32.
- Based on the measured αF, the existing aeration system has an estimated capacity of 15.0 mgd. The current fine-bubble aeration system was determined to be insufficient to meet current or future demands and it is recommended that the diffusers be replaced with diffusers capable of operating at higher airflow rates.

Tertiary Treatment

- The granular media filters have never been able to operate at their rated hydraulic loading of 5.0 gpm/ft^2 to produce 9.0 mgd filtered effluent and comply with the 2 NTU filter effluent limit for "disinfected tertiary" quality recycled water.
- It has been determined that the poor performance of the tertiary processes corresponded to high nitrite concentrations in the secondary effluent caused by incomplete nitrification and low MLSS concentrations (less than 1.0 g/L) due to the low SRT.
- Upon Brown and Caldwell's recommendation, the plant increased the operating SRT from approximately 2.75 days to approximately 5 days. This process change resulted in lower turbidity levels in the filter effluent and reduced chemical requirements.
- The chlorine contact tank has sufficient treatment capacity. The operations costs are relatively high because the high nitrite nitrogen concentration exerts a significant

chlorine demand (approximately 10 mg chlorine/mg nitrite nitrogen) that must be met before recycled water disinfection can be achieved.

Solids Treatment

- The DAFT units are currently operating below the rated plant capacity of 18.0 mgd on the basis of solids loading when one of two existing units is in service.

- Current digester volume does not meet SRT required by EPA 40 CFR 503 regulations when the largest unit is out of service. Increased thickened solids concentration will reduce hydraulic loading to the digesters and increase the available capacity.

- Centrifuge capacity is adequate for the solids generated at an average daily plant influent flow of 26.0 mgd.

Odor Control System

- The existing odor control system at the HARRF treats 66,000 cfm of foul air from the Primary Clarifier Building. A two-stage odor control system consisting of a bioscrubber followed by a mist scrubber was designed for handling hydrogen sulfide (H_2S) concentrations up to 200 parts per million by volume. However, current operational practice uses only the first stage for foul air treatment and recent monitoring results indicate that the bioscrubber is ineffective.

7.1.2 Escondido Land Outfall

The capacity of the land outfall must be increased to enable disposal of treated effluent from the HARRF at build-out conditions. Based on the cost estimates performed, the least costly effluent disposal options requires construction of a new 72-inch diameter land outfall capable of conveying up to 49.0 mgd of secondary or tertiary effluent from the HARRF. The existing outfall can be rehabilitated for a cost of $26 million.

7.1.3 San Elijo Ocean Outfall

In concert with the most cost-effective disposal scenario evaluated, the ocean outfall capacity must be expanded to 62.0 mgd. This outfall capacity includes 49.0 mgd allotted to the City and 13.0 mgd to the SEJPA and involves constructing a brand new 54-inch diameter ocean outfall to replace the existing SEOO. Constructing a new outfall dedicated solely for disposal of the HARRF effluent results in a pipe reduction of about 6 inches (to 48-inch diameter) and a reduction in total project cost of $9 million.

7.2 Recommendations

7.2.1 HARRF Improvements

Improvements recommended for the HARRF have been divided into two groups: items that will ensure the treatment capacity of the HARRF to 18.0 mgd average daily flow, and items that will enable the HARRF to treat up to the average daily flow of 27.5 mgd, which is expected at build-out. Considering that the annual average daily flow in 2005 was 15.3 mgd, upgrades to 18.0 mgd treatment capacity may quickly become inadequate if population growth occurs rapidly.

Recommended Near-Term Improvements for 18.0 mgd Average Daily Flow Capacity

Secondary Treatment Improvements

- Implement chemically enhanced primary treatment, where coagulants and flocculants are added upstream of the primary clarifiers to increase primary solids removal.
- Optimize the return activated sludge chlorination and polymer addition to the aeration basins to lower the sludge volume index and improve the settleability of the secondary.
- Increase aeration capacity by supplementing the existing diffuser capacity by adding air to the incoming return activated sludge stream.

Solids Processing Improvements

<u>*Dissolved Air Flotation*</u>

- Optimize polymer dosage to allow the dissolved air flotation thickeners to operate at higher solids loading rates and improve solids capture efficiency.
- Move the polymer injection point to turbulent areas to optimize mixing and contact of the polymer with the solids.
- Replace the thickener overflow weir with submerged launder pipe to provide cleaner water for recycle to the pressurized flow system.
- Provide control valve on the thickener effluent line to control the liquid level, maximize the drainage of water from the float, and increase the solids content of the thickened sludge.
- Replace pressurized flow pumps to meet necessary recycle flow for solids loading to provide sufficient flow for air saturation.
- Add second pressurization tank or increase operating level to provide sufficient residence time for air to dissolve and to reduce possibility of vortexing.

- Add continuous vent to purge excess nitrogen from the pressurization tank to increase gas absorption, and improve stability.

- Modify inlet and outlet piping to prevent vortexing and inlet pipe flooding.

- Direct a portion of the waste activated sludge to co-thicken with the primary sludge in the primary clarifiers. This is not recommended for day-to-day operation, but may be considered in an emergency if both dissolved air flotation thickeners are out of service.

Anaerobic Digesters

- Feed primary and secondary solids simultaneously to all digesters (on the same day) to ensure consistent solids feed to the digester, stabilize operation, and prevent gas production spikes.

- Verify lances and draft tubes are free of obstructions or buildup clear to ensure system is operating as designed.

- Adjust draft tube mixing capacity to provide 16 to 24 turnovers per day. This will provide sufficient mixing capacity to prevent solids deposition, surface matting, dead zones, and hot spots.

- Provide dedicated compressors for Digesters Nos. 1 and 2. This is needed to provide a balanced operation to draft tube gas mixing systems.

- Perform a dye study to confirm mixing efficiency in the digesters, particularly for Digester No.1.

- Consider recuperative thickening when taking a digester out of service in order to maintain the solids retention time required to produce Class B biosolids.

Centrifuge Dewatering

- Provide sludge samples to centrifuge and polymer suppliers to verify that the sludge character has not changed since centrifuges have been placed into service.

- Perform polymer trials to make certain that the correct polymer is being used.

- Perform periodic acid cleaning of centrate pipes and/or use polyphosphate scale inhibitors to maintain the centrate system hydraulic capacity and prevent backups from occurring.

Other Improvements

- Replace primary effluent with secondary effluent or reclaimed water as the wetting agent in the bioscrubber

- Establish continuous recycling of the wetting agent instead of using a pass-through effluent system.

Recommended Long-Term Improvements for 27.5 mgd Average Daily Flow Capacity

Barscreen

- Convert the existing manual bar screen to a mechanical bar screen that is identical to the existing mechanical bar screens to provide flexibility during peak conditions.

Influent Pump Station

- Conduct a field torsiograph test to assess torsional resonance issues associated with the IPS pumps and identify if the existing VFDs can operate at speeds greater than 60 Hz.

- Conduct a lateral resonance study to determine if the pump foundation, frame and motor supports, and rotating system can withstand the dynamic forces resulting from operation at the higher speeds.

- Contact the motor manufacturer to determine if the motor design will allow operation under the current and voltage required for the higher speed.

- Examine the electrical system to determine if there is sufficient capacity to carry the additional load. Any increase in motor size or overspeeding may require the upgrade of feeders to the IPS.

- Perform a comprehensive evaluation of the remaining useful life of the equipment (including gates, operators, valves, etc.) to determine the need for upgrades in system components or replacement of equipment.

- Increase the 30-inch IPS discharge pipes to 36-inch-diameter pipes and develop a system head curve to determine the final capacity of the existing pump station with the change in force main size, and whether the motors and drives need to be changed.

Primary Treatment

- Construct an additional primary clarifier to provide redundancy.

- Implement the following hydraulic modifications to de-bottleneck the primary clarifiers:

- ✓ Provide additional aeration tank inlet gates to mitigate primary clarifier effluent weir flooding
- ✓ Install additional aeration tank outlet gates to mitigate primary clarifier effluent weir flooding
- ✓ Increase secondary clarifier inlet column openings to mitigate mixed liquor splitter box weir flooding

- Implement intermittent CEPT to reduce the load on the secondary treatment facilities if high rate activated sludge option is selected. CEPT application is only needed during peak flow conditions.

Secondary Treatment

Alternative 3: High Rate Conventional Activated Sludge System

- Operate the existing activated sludge plant (with the addition of one primary clarifier and one aeration basin) as a high-rate system (SRT=2.0 d).
- Perform sludge wasting using mixed liquor rather than settled sludge. This would require that the waste activated sludge pumping capacity be increased and a new WAS pump station be constructed.
- Construct an additional aeration basin to provide redundancy and to ensure good effluent quality during replacement of the diffusers.
- Install a biological selector in each aeration basin that is estimated to reduce the 90^{th} percentile SVI value to 125 mL/g. In addition, it is recommended that the plant be operated in a high-rate operation (SRT = 2.0 d) to reduce the MLSS concentration and the subsequent loading on the secondary clarifiers.

Alternative 6: Moving Bed Bioreactor

- Convert 25 percent of the aeration basins to moving bed bioreactor.
- Modify the existing RAS line injection point to new configuration of the aeration basins.

Common Improvements

- Consider providing one additional blower as a potential improvement for the future.
- Replace the existing fine-bubble aeration equipment with one that allows for a higher quantity of oxygen to be added to the aeration basins.

- Upsize the current three small capacity RAS pumps with higher capacity RAS pumps.

- In general, when compared with separate sludge thickening, co-thickening would require larger DAFT units, smaller anaerobic digesters, and less dewatering centrifuges. Consider implementing the sludge co-thickening. Construct additional two 37-ft diameter DAFT units and one 109 diameter anaerobic digester for co-thickening option.

- Consider pilot testing of BAF, membrane filtration, or moving bed bioreactor, depending on the selected alternative.

Tertiary Treatment

- Implement one of the following two options to produce reclaimed water: (1) nitrifying BAF treating secondary effluent, or (2) membrane filtration treating secondary effluent.

- Although the chlorine contact tank provides adequate capacity for future recycled water demands, the City should revisit the 4.0 mgd UV disinfection capacity approved by DHS in the fall of 2003, as there are several potential opportunities to increase system capacity.

Other Improvements

- Install covers on the primary clarifiers and withdraw foul air from beneath the covers only to reduce the amount of foul air needing treatment from 66,000 cfm to 30,000 cfm.

- Install a new odor control system to treat the reduced foul air volume as well as the foul air from the headworks (total of 30,000 cfm). A single-stage carbon adsorber system would provide adequate treatment.

7.2.2 Outfall Improvements

- Initiate an alignment study to determine possible routes for a new land outfall and determine the constraints involved.
- Conduct condition assessments of the land and ocean outfalls.
- Conduct near-term capital improvements to maximize the existing capacity of the ELO.
- Evaluate the costs of acquiring land to construct a new land outfall.

- Periodically update the projected build-out flows using accurate available land use data.
- If considering expanding equalization at the HARRF, evaluate land and construction costs.
- Maintain the ability to dispose of 9.0 mgd to Escondido Creek during extreme conditions to provide disposal flexibility by renewing the current permit for intermittent live-stream discharge.
- Evaluate the advantages and disadvantages of constructing a separate ocean outfall that conveys solely effluent from the HARRF.
- Consider a regional land and ocean outfall that can benefit other communities.

APPENDIX A

PROCESS OBJECTIVES MEETING MINUTES

City of Escondido
HARRF Capacity Study
Process Objectives Meeting

Meeting Minutes

Date/Time: **December 19, 2005 1:30 – 4:30 pm**
Location: **Hale Avenue Resource Recovery Facility**
Notes By: **Josh Newman**

Attendees:

City of Escondido	Brown and Caldwell
John Burcham	Victor Occiano
Pete Klein	Eric Wahlberg
Jim Larzalere	Seval Sen
	Josh Newman

Note : Items in italics represent information that was filled in after the meeting. Some of this was not discussed directly at the meeting.

I. INTRODUCTIONS

Eric Wahlberg started the meeting with introductions.

II. HYDRAULIC AND PROCESS CAPACITIES

Eric discussed the differences between hydraulic and process capacities.

III. PLANT OVERVIEW AND PROCESS OBJECTIVES

a. Headworks

The process objective of the screens is to remove rags and stringy material that would otherwise clog downstream pumps and equipment.

The plant has two Parkson Aqua Guard® traveling screens with 6 mm bar spacing. Screenings are processed through a washer-compactor (also by Parkson). The washer-compactor unit(s) plugs up frequently and is a source of excessive maintenance. Loading to the washer compactors is high due to the large amount of screenings removed by the 6-mm Parkson screens.

b. Influent pumping

The process objective of the influent pumps is to lift the incoming wastewater to an elevation of 626.25 ft. The wastewater then flows through the plant by gravity for treatment and ultimately reaches the secondary effluent pump station. Plant flow averages around 16 mgd.

According to 1981 expansion drawings, influent pump design data are as follows:

Pump Type	*Quantity*	*Capacity (gpm)*	*TDH (ft)*	*Hp*
Constant Speed	*2*	*4600*	*30*	*50*
Variable Speed	*2*	*6000*	*30*	*50*
Variable Speed	*2*	*9000*	*40*	*125*

The City would like to keep 100% redundancy for influent pumps in the future (per discussion with John Burcham on January 9, 2006). However, the hydraulic loading condition was not discussed. BC will follow up with the City to determine if this pump redundancy should be evaluated at average, peak day, or some other peak hydraulic loading condition.

c. Grit removal

The process objective of grit removal is to remove abrasive grit that would otherwise damage equipment, and prevent accumulation of grit in digesters thus extending the required digester cleaning interval.

There are two vortex grit chambers (PISTA® by Smith and Loveless), each rated at 29 mgd. There are four recessed impeller grit pumps each with a rated capacity of 220 gpm at 40 ft of head. The grit slurry is processed through two grit cyclone/classifiers. Each cyclone is rated for 220 gpm.

d. Primary clarification

The primary clarifiers separate and remove suspended solids (SS) that easily settle and/or float under quiescent conditions. A fraction of SS contains organics that contribute to the organic loading to the secondary treatment system. The process objective of the primary clarifiers is to remove enough solids and organic material from the wastewater so that the oxidative capacity of the downstream secondary treatment system is not overloaded.

i. *Primary clarifiers – There are 4 primary clarifiers with design data as follows:*

Type	*Qty.*	*Flow (mgd)*	*SWD (ft)*	*Surface Area (SF)*	*Surface Overflow Rate (gpd/ft^2)*		*Weir Overflow Rate (gpd/ ft^2)*
					Avg.	*Peak*	*Avg.*
Rectangular	*3*	*4.25*	*8*	*5250*	*810*	*1,595*	*42,500*
Rectangular	*1*	*4.25*	*10*	*5250*	*810*	*1,595*	*21,250*

Sludge blanket in the primaries is typically maintained at about 6-inches (on the deck) with the sludge depth in the hoppers typically around 5-ft deep. The total solids (TS) content of the sludge is typically 3-4% and is rarely greater than 4%. The City would like to be able to take one unit out of service at any time and still meet process objectives.

The distribution of flow from the primary tanks into the aeration basin influent channel is seen as potentially problematic with conditions in certain primary tanks disproportionately impacting certain aeration basins. In addition, the effluent weir elevations have not been verified since the basins were installed and may be out of adjustment. John asked BC to move forward with a survey of the weir elevations to document the results and for use during the plant flow modeling task.

ii. Primary sludge pumps -- There are 6 primary sludge diaphragm pumps, each rated at 152 gpm at 80 ft of total dynamic head (TDH). The pump stroke and speed is adjustable. Raw sludge is pumped directly to the digesters at a TS content of 3-4%. The City would like to have 100% redundancy for the primary sludge pumps.

e. Activated Sludge Reactors

The process objective of the reactors is to (a) convert soluble, colloidal, and particulate organics in the primary effluent to biomass (i.e., activated sludge) and, (b) produce an activated sludge that flocculates well, settles well, and compacts well. Part (a) is generally not difficult and soluble BOD of the secondary effluent is typically 15 mg/L or less. Part (b) is not so easy.

i. Aeration basins – There are 5 identical aeration basins, each with an anaerobic selector that is partitioned with redwood baffles. The basins are equipped with Parkson membrane panel diffusers, except in the selector zone that has course bubble diffusers for mixing only. Dissolved oxygen (DO) is measured at the end of the basins. Plant staff do a DO profile of the basins weekly. Typically, the measured DO in the front, middle, and end portions of the basin are approximately 1.5, 2.5-3, and 1.2 mg/L, respectively. At peak flow, the DO in the latter portion of the basins can drop to 0.6 mg/L. The City finds that effluent quality deteriorates if the mixed liquor suspended solids (MLSS) concentration is allowed to increase above approximately 1,200 mg/L. The target mean cell residence time (MCRT) is 5 days. Larzalere and Burcham indicated that while they aren't required to nitrify, higher MCRTs and higher effluent nitrate concentrations seem to correspond to better secondary effluent quality and, therefore, they more easily meet Title 22 requirements. The City would like to be able to take one aeration basin out of service at any time.

ii. Aeration system -- Upgraded with Parkson panel membrane diffusers in 1980s. Multistage blowers were replaced with Turblex single stage blowers. Design data on these items were not available in the meeting and is not featured in the data provided to BC. These data must be collected. There are three blowers, (two duty plus one spare). Pressure limit (of the Parkson membrane panels) limits the plants ability to run more than two blowers at a time. The City indicated that air flow distribution to the basins is problematic and difficult due to air piping configuration. Air flow is measured in the main air header and in each individual basin header. The cumulative total air flow for the basins is ~ 21,000 scfm while at the main header the totalized air flow is ~ 16,000 scfm (i.e., up to 30% error). The City believes the individual basin values are more accurate due to straightening vanes and better meter location relative to pipe fittings. Based on the discussion, BC believes that it is possible that the aeration basins may be DO limited at times. It will be helpful for BC to review the data (i.e., specs, drawings, construction submittals) associated with the upgraded aeration system.

iii. WAS pumping – Waste activated sludge (WAS) is pulled off of the return activated sludge (RAS) header. There are two variable speed progressing cavity WAS pumps (1 duty plus 1 spare), each with a rated capacity of 515 gpm at of 35 ft of TDH. The WAS wasting rate is paced off of the RAS flow. Typically it is around 8.4 percent of RAS flow.

f. Secondary clarification

The process objective of the secondary clarifiers is to remove settleable solids. This is accomplished by providing for flocculation of the mixed liquor to occur, followed by a quiescent volume where settling and compaction can occur. The settled activated sludge can then be pumped back to the aeration basins RAS.

i. Secondary clarifiers – There are 4 secondary clarifiers (two large and two small). Each is circular clarifier with a center floc-well, sloped bottom, sludge collection mechanism, , and scum collection box. *Later review of the clarifier drawings showed a sludge collector mechanism with scrapers in a "V" configuration, which typically means a sludge withdrawal header at each V plumbed to a well at the center of the clarifier that empties by gravity to a RAS wet well.* For Title 22 water production , the turbidity of secondary effluent cannot be greater than 10 NTU. This limitation is linked to the ability of the Dynasand filters to produce Title 22 filtered effluent with a turbidity of 2 NTU or less. Larzalere indicated that it is typically easier to achieve these requirements when nitrification occurs in the aeration basins. The desire to nitrify and maintain a MLSS concentration less than 1,200 mg/L are conflicting. As a result, nitrification is usually only accomplished during the summer. The sludge volume index (SVI) is typically between 100 and 130 mL/g.

ii. RAS pumping – RAS pump design data is as follows:

Pump Type	***Quantity***	***Capacity (gpm)***	***TDH (ft)***	***Hp***
Variable Speed, Non-Clog	*3 (2+1 spare)*	*4,107*	*37*	*60*
Variable Speed, Non-Clog	*3 (2+1 spare)*	*2,173*	*35*	*25*

The RAS flow rate is flow-paced with the influent flow. The RAS flow rate cannot be maintained lower than 40% of plant flow because at low RAS flows (corresponding to low influent flows), the RAS pump suction piping in the small clarifiers plugs up due to a low scouring velocity.

g. Advanced treatment system – 8 Dynasand filters, each with a filtration area of 200 ft^2, are rated for 1,000gpm. Polymer and poly-aluminum chloride is added to the secondary effluent for coagulation/flocculation. However, the filter influent, after chemical addition, is subjected to a 5-foot fall as it enters the filters. This flow drop is suspected to cause significant floc shear. Seven filters are needed to produce 4 MGD of Title 22 water. This is only 400 gpm per filter. Therefore, the Dynasand filters are performing at less than half the rated capacity. The permit requires that maximum turbidity in the filtered effluent is 2 NTU. Title 22 also requires that the upper limit on filter influent turbidity is 10 NTU for direct filtration without upstream flocculation.and sedimentation.

h. UV Disinfection – *The UV system was not discussed extensively, but BC later confirmed that the UV system is rated for 4 mgd.*

i. Dissolved air flotation thickens (DAFTs) – There are two DAFTs. The City operates only one at a time. There is 100 percent redundancy under current conditions. *In a later discussion with John Burcham, BC determined that keeping 100 percent redundancy for the DAFTs is not required for the capacity study.* The DAFTs produce thickened sludge at 3-4% TS content and have a capture efficiency of approximately 98%.

j. Digesters – There are two primary and one secondary digester. One primary digester is typically offline during the summer months. The City is in the process of upgrading the digester heating system. Digester gas is collected from all three digesters. Currently, a majority of the gas is flared because the micro turbines are out of service.

k. Dewatering – There are 3 dewatering centrifuges (2 duty + 1 spare). Solids capture efficiency is ~ 95%. Centrifuges run approximately 16 hours per day on swing and graveyard shifts. There are 6 hours of dewatered biosolids storage in the truck loading hopper. Biosolids are also stored in covered trailers. There is no problem with biosolids storage and the plant has a sufficient number of trailers for storage to meet their biosolids program needs.

l EQ Basins – The existing equalization basin (EQ) basin will be used for brine storage in the future and will not be available for secondary effluent flow equalization. The City plans to construct a 2-MG secondary effluent EQ Tank and a 1-MG Reclaimed water EQ tank.

IV. PEAK FLOWS AND LOADS

Peaking factors will be developed based on historical data analysis and storm frequency. City of San Diego uses 10 year storm event for design purposes.

V. REDUNDANCY

In general, the City wants 100 percent redundancy for pumps and the ability to take one basin or tank out of service at any time for process units.

VI. STAFF OBSERVATIONS/OPERATIONAL CONSTRAINTS

1. If MLSS increases above 1,200 mg/L the secondary effluent quality deteriorates.

2. At peak flow (typically occurring on weekdays in the afternoon), the DO in the latter portion of the aeration basins can drop to 0.6 mg/L.

3. No more than two blowers can be operated at a time because of pressure limit associated with Parkson membrane panel diffusers.

4. RAS flow rate cannot be lower than 40% of plant flow because at low flow, the RAS pump suction piping plugs up due to low scouring velocity.

5. 7 of the 8 Dynasand filters installed are needed to produce 4 MGD of Title 22 water. This is only 400 gpm per filter. Therefore, the Dynasand filters are performing at less than half the rated capacity. Production of Title 22 water drives how the secondaries are operated. The plant must nitrify to enable production of tertiary effluent with turbidity of 2 NTU or less.

6. Flow distribution of primary effluent to aeration basins is poor and aeration basins get fed disproportionately from the primary basin closest to it. Therefore, if there is a septic primary, it will disproportionately affect the closest aeration basin.

7. There was some discussion regarding a new brewery discharge to the plant. John Burcham indicated that the brewery discharge limits for flow and BOD are 20 gpm and 1,200 lb/day, respectively. *John mentioned in a later conversation that that the flow was in the range 1,000 to 3,000 gpd initially but went up to around 13,000 gpd recently. This increase seemed to coincide with problems with chemical coagulation and flocculation upstream of the filters. The plant is in the process of having several chemical suppliers evaluate the problem. This brings up the question of industrial discharges in general. As part of the capacity analysis, BC will need to understand the range of industrial users on the City's collection system.*

VII. <u>EFFLUENT LIMITS</u>

Effluent limits are currently 30 mg/L TSS and 25 mg/L CBOD for ocean discharge with no nutrient limit. However, this study must also consider full nutrient removal to address live stream discharge given the ocean outfall will be capacity-limited in the future. The Biowin model must consider two scenarios: Conventional AS, and AS with nutrient removal.

<u>Action Items</u>

- As approved by John Burcham in the meeting, BC will provide a survey of weir and water surface elevations under the Additional Services task.
- BC will work with City staff to obtain specifications, drawings, and construction submittals associated with the secondary treatment aeration system (i.e., blowers, piping, and diffusers, etc.)
- BC will work with City staff to obtain the significant industrial users list.

APPENDIX B

HARRF HYDRAULIC PROFILE TM

TECHNICAL MEMORANDUM – FINAL

DATE: July 5, 2006

TO: ANGELA MORROW, CITY OF ESCONDIDO

FROM: VICTOR OCCIANO, BROWN AND CALDWELL

PREPARED BY: SEVAL SEN, BROWN AND CALDWELL
RON APPLETON, BROWN AND CALDWELL

SUBJECT: CITY OF ESCONDIDO
HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF)
– PLANT HYDRAULIC PROFILE ANALYSIS

The City of Escondido has engaged Brown and Caldwell to determine the capacity of the Hale Avenue Resource Recovery Facility (HARRF). One of the tasks is to evaluate the hydraulic capacity of the facility. The purpose of this memorandum is to (1) present the results of hydraulic analysis of the liquid stream unit processes at the HARRF for varying flow conditions; (2) identify the flow restrictions; and (3) summarize findings and recommendations.

INTRODUCTION

Brown and Caldwell performed the hydraulic analyses using a proprietary hydraulic and energy profile generation software developed by Brown and Caldwell entitled PROFILE ™. This program is a useful tool which allows the design engineer to model various hydraulic configurations by changing the influent flow, return flows, and number of process units in service.

PROFILE ™ operates on the conservation of energy principle and calculates the hydraulic profile from the energy gradient. The energy grade is the fundamental reference for the profile calculations. The program requires identification of the controlling downstream energy grade line. Then all the energy losses are applied to the downstream energy grade to identify the upstream energy grade. The hydraulic grade is then calculated by subtracting the velocity head from the energy grade.

Headloss Equations

Assumptions for headlosses for major hydraulic elements included in PROFILE™ are defined below. Note that not all these hydraulic elements are used to model the hydraulic profile at HARRF.

1. **Pipe Friction**. Headlosses for pipe friction are assumed to conform to the Darcy-Weisbach equation simplified by using roughness factors from the Colebrook equation (from Moody diagram), a copy of typical Colebrook friction factors taken from Handbook of Hydraulics, Brater and King, 6th edition, pg 6-12 is provided in the Appendix for reference:

   $$H = (f * l/d)*(V^2/2g)$$

   where
   H = headloss (feet)
   f = Colebrook pipe friction factor
   l = pipe length (feet)
   d = pipe diameter (inches)
   $V^2/2g$ = velocity head (feet)

   Colebrook roughness factors f are assumed as follows:
   Old concrete 0.0025
   Old steel pipe 0.005

2. **Minor Losses**. Headlosses for fittings and valves are assumed to conform to the following:

   $$H = K*(V^2/2g)$$

   where
   H = headloss (feet)
   K = an empirically developed headloss coefficient
   $V^2/2g$ = velocity head (feet)

   Headloss coefficients (K) are assumed as follows:

Standard 90 elbow	0.3	
Standard 45 elbow	0.2	
Standard tee	1.0	Flow through
Entrance coefficient	0.5	Sudden area change
Exit coefficient	1.0	Sudden area change

3. **Open Channels or Conduits Flowing Part Full.** Headlosses in open channels or conduits flowing part full are assumed to conform to the Manning Equation.

   $$H = L*[n*V/(1.49*R^{0.67})]^2$$

   where
   H = headloss (feet)
   L = channel or conduit length (feet)
   n = Manning coefficient

V = velocity (feet per second)
R = hydraulic radius

Manning coefficients (n) are assumed as follows:

Finished concrete	0.013

4. **V- Notch Weirs.** Headloss for rectangular notch weirs are assumed to conform to the following equation:

$$H_n = (Q_n/2.5 \tan(\theta/2))^{0.4}$$

where H_n = headloss per notch (inches)
θ = notch angle (degrees)
Q_n = flow per notch (cubic feet per second)

5. **Cutthroat Flumes.** Headloss for cutthroat flumes are assumed to conform to the following equation for standard length flumes including 1.5, 3, 4.5 and 9 feet with maximum allowable submergence equal to 85% (ref. Skogerboe, et al, "Selection and Installation of cutthroat flumes"):

$$H_a = (Q/(K{*}W\ 1.025)^{(1/n)}$$

where H_a = upstream water depth
Q = flow through flume (cubic feet per second)
K = flume length factor
W = flume throat width
n = flume length coefficient

Flume length factor (K) are assumed as follows

Flume length 1.5 feet:	6.4
Flume length 3.0 feet:	4.5
Flume length 4.5 feet:	3.98
Flume length 9.0 feet:	3.5

Flume length coefficient (n) are assumed as follows

Flume length 1.5 feet:	2
Flume length 3 and 4.5 feet:	1.72
Flume length 9 feet:	1.56

6. **Launder/Effluent Collector**. Headlosses in launders and effluent collectors consider both friction losses and momentum effects. Flow along the launder/effluent collector is considered to be increasing along the length as flow is added.

7. **Submerged Orifice.** Headloss for submerged orifices are assumed to conform to the following equation:

$$H = (Q/C{*}A)^2/2g$$

where H = headloss (feet)
Q = flow per orifice (cubic feet per second)
C = orifice coefficient (assumed to be 0.6)
A = area of orifice (square feet)

8. **Sharp-Crested Weir.** Headloss for sharp crested weirs are assumed to conform to the following equations:

For end contractions:

$$Q = C\,(L-0.2\,H)*H^{1.5}$$

where Q = flow across weir (cubic feet per second)
C = weir coefficient
L = length of weir (feet)
H = height of water above crest (feet)

9. **Broad-Crested Weir.** Headloss for broad-crested weirs are assumed to conform to the following equation:

$$Q = C\,LH^{1.5}$$

where Q = flow across weir (cubic feet per second)
C = weir coefficient
L = length of weir (feet)
H = height of water above crest (feet)

Data Collection

The hydraulic model developed for HARRF is based on information provided in the following design drawings:

- HARRF Phase 2- Treatment Upgrades and Water Reclamation Facilities, Volume 2, June 1999
- HARRF Phase 1B- Aeration Upgrades, Volume 3, May 1998
- Expansion Hale Avenue Wastewater Treatment Facilities, Volume 2, October 1981
- Plans for the Construction of 1973 Water Quality Control Plant, 1971

In addition, Brown and Caldwell identified and surveyed 57 points consisting of several weir crest and water surface elevations at the flow controlling structures. Surveyed elevations were used instead of drawing elevations wherever available for the hydraulic model. A comparison of the surveyed and drawing elevations is attached.

Identification of the Downstream Energy Grade Line

The hydraulics of the headworks, primary clarifiers, aeration basins and the secondary clarifiers are dictated by the water surface elevation in the Secondary Effluent Pump Satiation (SEPS). The SEPS influent channel is equipped with a motorized downward opening gate which may be positioned up or down to control the amount of water which is discharged to the outfall. As the weir is lowered, more flow goes over the weir gate and the flow to the outfall increases. As the weir gate is raised, the flow to the outfall decreases which forces more water into the equalization basin pump wetwell.

As the worst case scenario in the hydraulic simulation, it was assumed that the motorized gate in the SEPS influent channel is at its highest elevation and could not be lowered due to operational problems. This condition will force the flow to go to the equalization pump wetwell through fixed weirs between the SEPS influent channel and the equalization pump wetwell. As part of the worst case scenario, it was also assumed that the equalization basin is full; therefore equalization basin pumps are turned off. The flow will go through the fixed weir at the north end of the equalization pump wetwell and be discharged to the outfall. As a result, the controlling downstream water level for the worst case condition will be this weir crest elevation (EL 608.93 according to the survey conducted by Brown and Caldwell).

Hydraulic Simulations

Two flow conditions, the current maximum rating capacity of 18 mgd and the future build-out capacity of 27.5 mgd were simulated using PROFILE™. Both average dry weather flow (ADWF) and peak wet weather flow (PWWF) were simulated for each flow capacity. Model configuration is described in the following sections.

Hydraulic Simulation at 18 mgd Capacity

Average Dry Weather Flow

The HARRF is currently rated to treat and discharge (on the average) up to 18.0 mgd of secondary treated wastewater. Currently, the tertiary filters and the UV system rated treatment capacity is 4 mgd. Although it is not measured at the plant, filter backwash water generation is expected to be about 15 percent of the filtered influent. This means that the average filter backwash generation is 0.6 mgd, which is recycled back to the headworks. Additional recycle streams are DAFT subnatant and centrate. Total recycle stream is expected to be about 1.4 mgd. Therefore, the average plant influent of 19.4 mgd is used for the average flow hydraulic simulations. RAS recycle flow is assumed as 40 percent of the plant influent flow.

It is assumed that one process unit is always out of service wherever there is more than one unit for conservative simulation of the average flow hydraulic condition. The units in service for the average flow simulation are summarized below.

Flow: 19.4 mgd	
Process Unit	**Number of Units in Operation**
Grit Chambers	1
Primary Clarifiers	3
Aeration Basins	4
Secondary Clarifiers	3

Peak Wet Weather Flow
According to Wastewater Collection System Master Plan Upgrade Report, November 2005, the wet weather peaking factor for HARRF is 2.0. Therefore, the peak flow using for this model scenario is 37.4 mgd, including the 1.4 mgd recycle flow.

For the peak flow hydraulic simulation, it is assumed than all the process units are in operation as listed below.

Flow: 37.4 mgd	
Process Unit	**Number of Units in Operation**
Grit Chambers	2
Primary Clarifiers	4
Aeration Basins	5
Secondary Clarifiers	4

Hydraulic Simulation at 27.5 mgd Capacity

Average Dry Weather Flow
Projected average dry weather wastewater flow from areas served by a treatment plant is expected to be 27.5 mgd in year 2030 (reported in Brown and Caldwell's *HARRF System Integration and Optimization Technical Memorandum*). Ultimate capacity of the tertiary filters and UV system is expected to be 9 mgd to supply the future reclaimed and utility water demand. Several recycle flows are generated at the plant and discharged back to the headworks, including the DAFT subnatant, centrate and tertiary filter backwash water. For the built out condition, the total recycle stream flow is expected to be around 3.8 mgd. To be conservative, this recycle flow is added to the average flow of 27.5 mgd and therefore, the average plant influent of 31.3 mgd is used for the average flow hydraulic simulations. RAS recycle flow is assumed as 50 percent of the plant influent flow.

Several processes were evaluated to meet the future flows and loads at HARRF as reported in Brown and Caldwell's *HARRF System Integration and Optimization Technical Memorandum.* Among four viable secondary process improvement alternatives, Alternative 3 was selected for hydraulic simulation. Alternative 3 includes addition of one primary clarifier and one aeration basin for the liquid stream treatment. Addition of one more grit chamber is needed to accommodate the future peak flows.

It is assumed that one process unit is always out of service wherever there is more than one unit for conservative simulation of the average flow hydraulic condition. The units in service for the average flow simulation are summarized below.

Flow: 31.3 mgd

Process Unit	Number of Units in Operation
Grit Chambers	2
Primary Clarifiers	4
Aeration Basins	5
Secondary Clarifiers	3

<u>Peak Wet Weather Flow</u>
Expected peak flow is 53.6 mgd (Table 2.2 in Brown and Caldwell's *HARRF System Integration and Optimization Technical Memorandum)*. With addition of the recycle streams, total plant peak flow is 57.4 mgd.

For the peak flow hydraulic simulation, it is assumed than all the process units are in operation as listed below.

Flow: 57.4 mgd

Process Unit	Number of Units in Operation
Grit Chambers	3
Primary Clarifiers	5
Aeration Basins	6
Secondary Clarifiers	4

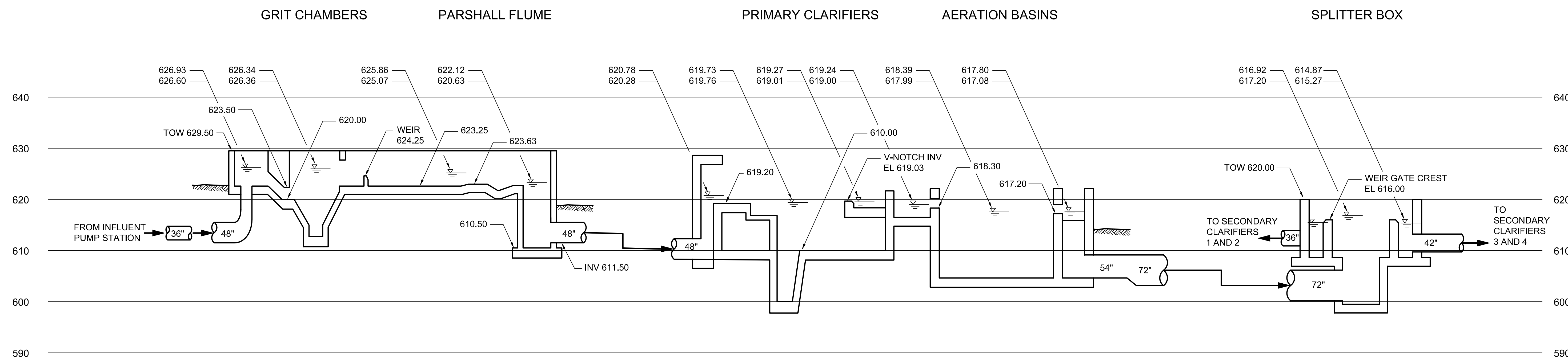
Results and Discussion

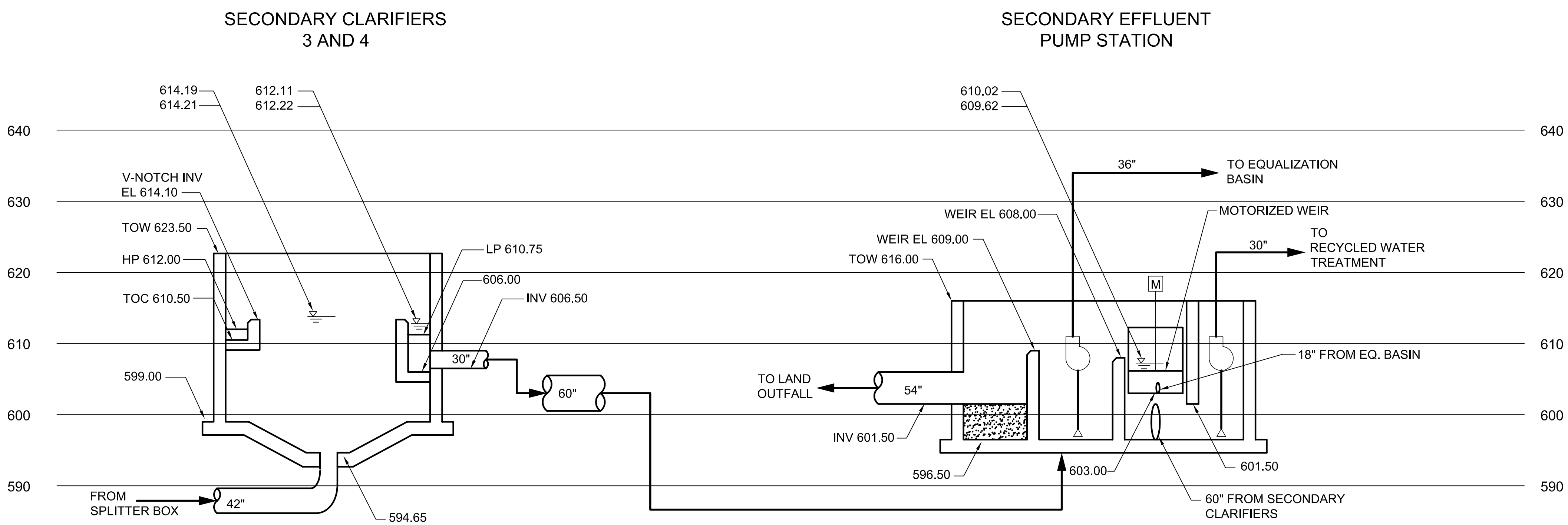
The HARRF hydraulic profile for average dry weather and peak wet weather flow scenarios is shown on Drawing G-1 and G-2 for 18 mgd and 27.5 mgd treatment capacities, respectively. The plant influent flow, RAS flow, and number of process units in service for each flow scenario are shown on the drawing.

The profile does not indicate any "hydraulic bottlenecks" that would limit flows to less than the design values of 18.0 mgd ADWF and 36.0 mgd PWWF. All tank water surface elevations (WSELs) are lower than the respective top of wall elevations, so there is no danger of overflowing. Additionally, WSELs downstream of primary clarifier and secondary clarifier weirs are lower than the respective weir elevations, so clarification performance and scum removal is not compromised. Finally, the WSEL downstream of the Parshall flume results in "free flow" conditions, so influent flow rate measurement is not compromised.

At the future built-out condition, the primary clarifier launders become inundated as well as the splitter box to a point that a true nappe does not exists. The aeration basin inlet and outlet gates and the outlet orifices of the central column of the secondary clarifiers are identified as the "hydraulic bottlenecks." Doubling the number of openings in the inlet and outlet channels of the aeration basin will reduce the headloss at high flows, consequently reducing the water level at the primary effluent launders. In addition, increasing the size of the outlet orifices of the central column of the secondary clarifiers will limit the headloss at 0.5 ft at the peak flow conditions, thus improving the conditions at the splitter box weir. These improvements are expected to lower the WSELs downstream of primary clarifier and secondary clarifier weirs below the respective weir elevations, therefore, clarification performance and scum removal are not compromised.

As a result, with additional one primary clarifier, one aeration basin, and one grit chamber and with some improvements to the aeration basin inlet and outlet channel openings and the central feed column of the secondary clarifiers, the existing plant will have hydraulic capacity to handle ADWF of 27.5 mgd and PWWF of 53.6 mgd.





LEGEND:

PEAK WET WEATHER FLOW (PWWF)
AVERAGE DRY WEATHER FLOW (ADWF)

FLOW RATES:

	ADWF	PWWF
PLANT INFLUENT FLOW, MGD	18.6	36.6
RAS FLOW, MGD	7.44	18.1

PROCESS UNITS IN SERVICE:

GRIT CHAMBERS	1	2
PRIMARY CLARIFERS	3	4
AERATION TANK	4	5
SECONDARY CLARIFIERS	3	4

Photo Image Filename:
Path: P:\Projects\Escondido, City of (CA)\129259 Filename: 129295-G1 Plot date: Jun 12, 2006-11:35:58am CAD User: trigor

BROWN AND CALDWELL
SAN DIEGO, CALIFORNIA

SUBMITTED: ____________ DATE: ______
PROJECT MANAGER
APPROVED: ____________ DATE: ______
BROWN AND CALDWELL

LINE IS 2 INCHES AT FULL SIZE (IF NOT 2" - SCALE ACCORDINGLY)

DESIGNED: RA
DRAWN: HK
CHECKED:
CHECKED: FB
APPROVED: RA

EXTERNAL REFERENCE FILES
129259BDR.dwg

REVISIONS

ZONE	REV.	DESCRIPTION	BY	DATE	APP.

CITY OF ESCONDIDO
CALIFORNIA

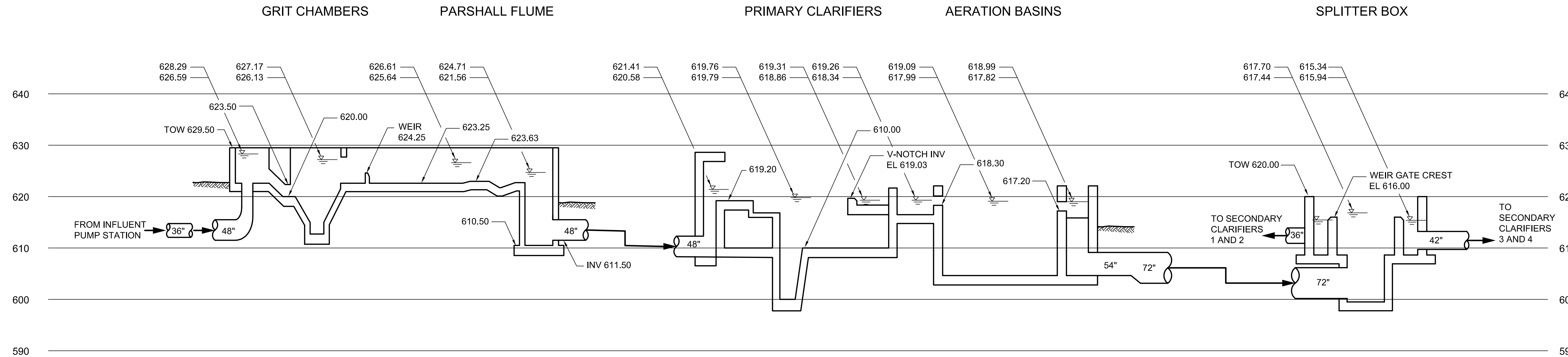
HALE AVENUE RESOURCE RECOVERY FACILITY CAPACITY STUDY

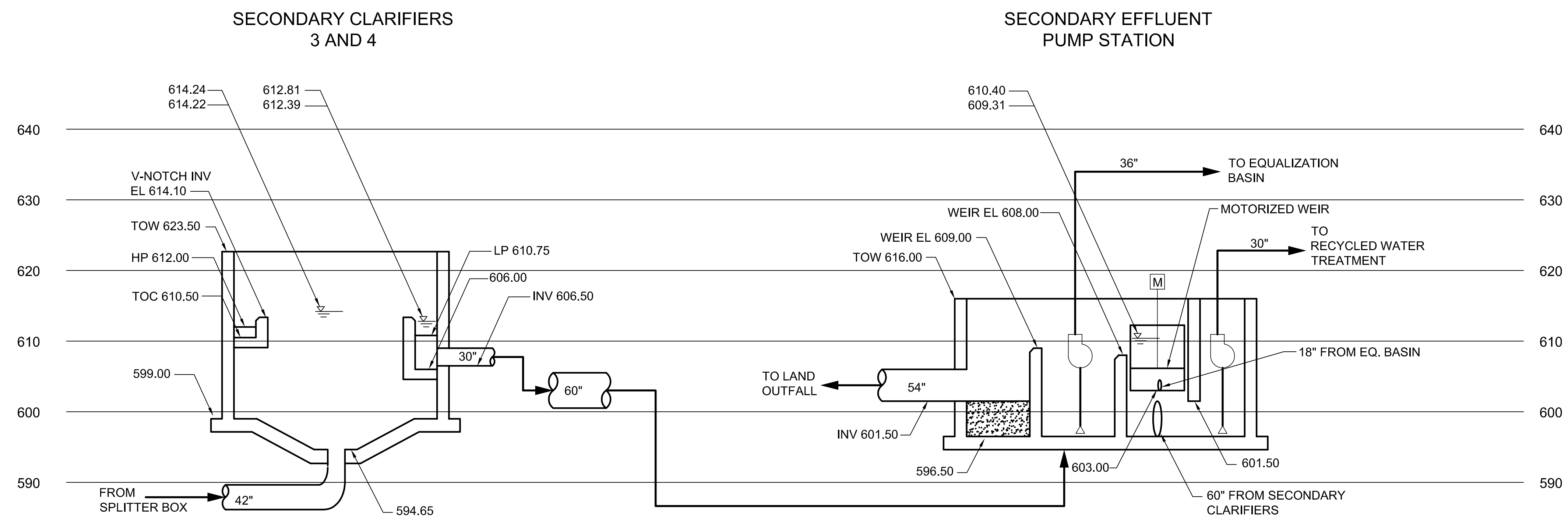
HYDRAULIC PROFILE FOR CURRENT RATED CAPACITY

CAD FILENAME
FILE NUMBER
SCALE
NO SCALE
DRAWING NUMBER
G-1
SHEET NUMBER

04-27-06

GRIT CHAMBERS PARSHALL FLUME PRIMARY CLARIFIERS AERATION BASINS SPLITTER BOX





LEGEND:

PEAK WET WEATHER FLOW (PWWF)
AVERAGE DRY WEATHER FLOW (ADWF)

FLOW RATES:

	ADWF	PWWF
PLANT INFLUENT FLOW, MGD	31.3	57.4
RAS FLOW, MGD	12.5	28.7

PROCESS UNITS IN SERVICE:

GRIT CHAMBERS	2	3
PRIMARY CLARIFERS	3	5
AERATION TANK	5	6
SECONDARY CLARIFIERS	3	4

BROWN AND CALDWELL

SAN DIEGO, CALIFORNIA

SUBMITTED: PROJECT MANAGER DATE:

APPROVED: BROWN AND CALDWELL DATE:

LINE IS 2 INCHES AT FULL SIZE (IF NOT 2" - SCALE ACCORDINGLY)

DESIGNED: RA
DRAWN: HK
CHECKED:
CHECKED: FB
APPROVED: RA

EXTERNAL REFERENCE FILES
129259BDR.dwg

REVISIONS

ZONE	REV.	DESCRIPTION	BY	DATE	APP.

CITY OF ESCONDIDO
CALIFORNIA

HALE AVENUE RESOURCE RECOVERY FACILITY CAPACITY STUDY

HYDRAULIC PROFILE FOR FUTURE BUILD-OUT CONDITION

CAD FILENAME
FILE NUMBER
SCALE
NO SCALE
DRAWING NUMBER
G-2
SHEET NUMBER

04-27-06

Photo Image Filename:
Path: P:\Projects\Escondido, City of (CA)\129259 Filename: 129295-G2 Plot date: Jun 12, 2006-11:36:40am CAD User: trigor

APPENDIX C

PRIMARY CLARIFIER EVALUATION TM

TECHNICAL MEMORANDUM - FINAL

DATE: JULY 5, 2006

TO: ANGELA MORROW, CITY OF ESCONDIDO

FROM: VICTOR OCCIANO, BROWN AND CALDWELL

PREPARED BY: RION MERLO, BROWN AND CALDWELL
ERIC WAHLBERG, BROWN AND CALDWELL
JOSE JIMENEZ, BROWN AND CALDWELL
SEVAL SEN, BROWN AND CALDWELL

SUBJECT: CITY OF ESCONDIDO
HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – PRIMARY CLARIFIER STRESS TESTING AND CAPACITY ASSESSMENT

SUMMARY

The purpose of this memorandum is to (1) present the results of data analysis of historical primary clarifier data at the Hale Avenue Resource Recovery facility (HARRF); (2) present results of the on-site stress testing performed on the primary clarifiers; and (3) present the estimated capacity of the primary clarifiers. Historic data (2000-2005) was evaluated to determine existing primary clarifier performance and to determine settling constants used to predict clarifier performance. The average carbonaceous 5-day biochemical oxygen demand ($cBOD_5$) removal was 31 percent and the total suspended solids (TSS) removal was 64 percent over the five years of historic data.

The historically determined settling constants were verified with on-site stress tests of the primary clarifiers and showed good agreement. Stress testing was performed at surface overflow rates (SOR) ranging from 800 to 1442 gallons per day per square foot (gpd/ft^2) achieved by taking tanks out of service. Field observations showed that there is flow imbalance in the flow splitting between clarifiers, with more flow going to Clarifier 4 (the eastern-most clarifier). Regardless of the flow imbalance, the clarifiers demonstrated similar TSS and COD removal with all clarifiers in service. The stress testing showed that HARRF's primary clarifiers still remove 46 percent of the influent

solids and 26 percent of the influent $cBOD_5$ at a SOR of 1,400 gpd/ft^2. In addition, the primary clarifiers remove all the settleable TSS, just as they are supposed to, at flow rates "typical" of average flows equating to a SOR ranging from 800-849 gpd/ft^2.

Using the settling constants determined from the historic data and validated with the stress testing, performance curves for COD and TSS removal as a function of influent flow were generated. These curves will be used with the analysis of the activated sludge process to determine the overall HARRF capacity.

INTRODUCTION

The City of Escondido is currently engaged in a study to determine the overall treatment capacity of the Hale Avenue Resource Recovery Facility (HARRF). One of the tasks under the study is to evaluate the capacity of the existing primary clarifiers.

The HARRF is designed to treat an average daily flow of 18 mgd. The original plant was constructed with three primary clarifiers, each with a surface area of 5,250 square feet (ft^2) and a side water depth of 8 feet. The fourth primary clarifier was added in the 1982 expansion, and has the same surface area (i.e., 5,250 ft^2) and a side water depth of 10 feet. According to the design criteria presented in Table 1, the primary clarifiers were designed to operate at an average surface overflow rate (SOR) of 810 gpd/ft^2 and a peak SOR of 1,595 gpd/ft^2. With all primary clarifiers in service, this equates to an average flow of 17.0 mgd and a peak flow of 33.5 mgd corresponding to an average-to-peak-flow peaking factor of 1.97. Table 1 summarizes the design criteria for the existing primary clarifiers.

In a letter report to determine the HARRF capacity, the primary clarifiers were evaluated at an average SOR of 1,000 gpd/ft^2 and a peak SOR of 2,500 gpd/ft^2. At these SOR values and with all the clarifiers in service, the average and peak capacities reported were 21.0 and 52.5 mgd, respectively. With the largest unit out of service, the average and peak capacities reported were 15.8 and 39.4 mgd, respectively. (Final Letter Report for Capacity Rerating of the Hale Avenue Resource Recovery Facility (HARRF), MWH, 2004).

Table 1. Existing primary clarifiers at HARRF.

Process Unit	Design Criteria
Number	4
Side Water Depth	3 @ 8 ft 1 @ 10ft
Surface Area/tank	5,250 ft^2
Average Design SOR	810 gpd/ft^2
Peak Design SOR	1,595 gpd/ft^2

The design criteria in Table 1 oversimplifies the capacity determination of the HARRF primary clarifiers. Primary clarifiers are suspended solids removal devices. Because physical forces predominate in the removal of suspended solids in primary clarifiers, many in the wastewater treatment profession consider them constant-percentage removal devices. This is simply not the case; rather, the total suspended solids (TSS) removal efficiency decreases with increasing flow. While primary clarifiers can only remove settleable TSS and the chemical oxygen demand (COD) or 5-day carbonaceous oxygen demand ($cBOD_5$) associated with those removed TSS (from the standpoint of the capacity of the downstream activated sludge process) it is the amount of COD (or $cBOD_5$) in the primary effluent that is of tantamount concern. Note that when measuring organic removal in a primary clarifier, $cBOD_5$ is the preferred measurement over total BOD_5 because a total BOD_5 value may be influenced by the presence of ammonia (ammonia is soluble and will pass through the clarifier). Therefore, the capacity of the primary clarifier to remove organics (i.e., COD or $cBOD_5$) is inextricably tied with the capacity of the activated sludge process to oxidize the organics discharged in the primary effluent. For this reason, the capacity of the two cannot be evaluated independently and can sometimes be manipulated to increase the combined capacity without the need to add additional structures. For example, if there was unused capacity in the activated sludge system, the primary clarifiers could be operated sub-optimally (i.e., at a higher hydraulic loading), because the increase in primary effluent organics would be compensated for by the excess capacity in the activated sludge system.

Within practical limitations, raw wastewater suspended solids can be classified as either settleable (TSS_{set}) or non-settleable (TSS_{non}). The effluent from an ideal primary clarifier would contain TSS_{non} and those TSS_{set} having settling velocities less than the SOR existing at the time the sample was collected. Raw wastewater COD can be classified as either soluble (sCOD or $sBOD_5$), particulate associated with settleable suspended solids ($pCOD_{set}$), or particulate associated with non-settleable suspended solids ($pCOD_{non}$). Non settleable COD (COD_{non}) is comprised of sCOD and $pCOD_{non}$. Primary clarifiers can only remove settleable suspended solids and the COD associated with those solids. The optimum performance of a primary clarifier, therefore, is dictated by the characteristics of the raw wastewater; that is, the relative amounts of settleable-versus-non settleable suspended solids and soluble-versus-particulate COD. Effluents from full-scale primary clarifiers, which are not ideal, also contain particulate COD associated with settleable suspended solids that escape.

Historically, primary clarifier performance has been based on the removal efficiencies of total suspended solids (TSS) and total COD (or BOD_5) with no regard to the ratio of settleable-to-non-settleable suspended solids and soluble-to-particulate COD in the influent. In fact, primary clarifier performance should be based on the removal efficiencies of settleable suspended solids and settleable COD.

Poor primary clarifier performance, based on the quantity of escaping settleable suspended solids and settleable COD, can be attributed to one or more of the following:

- poor flocculation of the incoming solids;
- inadequate primary sludge removal; and/or
- poor tank hydraulics causing flow and solids short circuiting.

The performance of a primary clarifier can be determined by collecting TSS and COD data from the influent and effluent. These results can be used to determine if the clarifier can be optimized. In addition, stress tests can be performed to determine the capacity of the clarifiers. Stress testing was performed on December 6 and 7, 2005 by taking clarifiers out of service to increase the SOR. After the one clarifier was taken off line, approximately two hours were allowed to elapse before a set of samples, as described above, was collected. The performance of primary clarifier 1 and 4 was monitored during the stress testing. Influent samples to the test primary clarifiers were collected at the influent structure and analyzed for TSS_{non} and COD_{non}. Dispersed and non-settleable samples were collected at the clarifier inlet and effluent. Dispersed samples are collected by settling a sample for 30 minutes and measuring the contents of the supernatant. Non-settleable contents are determined by mixing a sample for 30 minutes to induce flocculation followed by a 30 minute settling period. The non-settleable portion is measured in the supernatant Effluent samples were collected as the primary effluent flowed over the weir at the same sample location as the effluent dispersed samples. These samples were collected three times over the course of the two days. In addition, stress testing experiments were performed on-site to observe the performance of the test primary clarifiers at elevated flows.

Identifying the fact that primary clarifiers remove only seattleable particulate material, the following equations were adapted from Wahlberg *et al.* (2005)[1] to describe the performance of the primary clarifiers at the NDWWTP.

- $C_{PE}=C_{non}+(C_{PI} - C_{non})e^{-\lambda/SOR}$ (1)
- $E=[1 - (C_{non}/C_{PI})] - [1 - (C_{non}/C_{PI})]e^{-\lambda/SOR}$ (2)
- $E= E_{max}\ (1 - e^{-\lambda/SOR})$ (3)
- $E_{max} = 1 - (C_{non}/C_{PI})$ (4)

where:
C_{PE} = primary effluent COD, $cBOD_5$, BOD_5, or TSS concentration (mg/L)
C_{non} = non-settleable COD, $cBOD_5$, BOD_5, or TSS concentration (mg/L)
C_{PI} = primary influent COD, $cBOD_5$, BOD_5, or TSS concentration (mg/L)
λ = settling constant (gpd/ft^2)
SOR = surface overflow rate (gpd/ft^2)
E = removal efficiency
E_{max} = maximum removal efficiency

Equations 1 through 4 describe the performance of primary clarifiers. By determining λ and C_{non} values from historic data, the performance of the primary clarifiers can be determined at a given flow rate, or SOR. From a series of these determinations, a performance curve can be developed so that the oxidative capacity of the activated sludge system can be matched with the performance of the primary clarifiers thereby fixing their capacity. Also, by performing field stress tests, the λ and C_{non} values can be verified as representative of current operation.

[1] Wahlberg, E., Stallings, R., and Appleton, R (2005) Primary clarifier design concepts and considerations. *WEFTEC*, Washington, D.C.

RESULTS AND DISCUSSION

Historic Data

Data for the primary clarifiers for the period between January 2000 to September 2005 were analyzed and the TSS and $cBOD_5$ removal efficiencies calculated. TSS removal efficiency ranged from 11 to 97 percent with an average value of 64 percent; $cBOD_5$ removal efficiencies ranged from 0 to 74 percent with an average of 31 percent (Figure 2). The daily SOR ranged from 530 to 1,100 gpd/ft^2 with a median value of 680 gpd/ ft^2 (Figure 3a). Primary influent TSS (TSS_{PI}) values ranged from 141 to 987 mg/L and had a median value of 277 mg/L (Figure 3b). Plant influent $cBOD_5$ ($cBOD_{PI}$) values ranged from 122 to 584 mg/L with a median value of 228 mg/L (Figure 3c).

The SOR did not appear to have a significant effect on TSS or $cBOD_5$ removal efficiency (Figure 4). Due to the nature of Equation 2, TSS removal is a function of both influent TSS and SOR. Because both of these parameters are varying in the historic data, a distinct relationship is not immediately evident. Equation 2 was fit to the historic data to determine values for λ_{TSS} and TSS_{non} (Figure 5). The values for λ_{TSS} and TSS_{non} were determined to be 2,347 gpd/ft^2 and 92.2 mg/L. Equation 2 also was fit to historic $cBOD_5$ data to determine λ_{cBOD} and $cBOD_{non}$, resulting in 970 gpd/ft^2 and 132.2 mg/L, respectively (Figure 7). Lower λ values have been observed for $cBOD_5$ removal compared to TSS removal in other analyses and is likely due to the presence of colloidal $cBOD_5$, which would be measured as "soluble", and can be removed in primary clarifiers by incorporation into settleable flocs.Using the lower λ value determined from the cBOD historic data will under-predict clarifier removal. Therefore, the analysis below uses λ from the TSS results.

On-Site Stress Testing

Stress testing of the primary clarifiers was performed on December 6 and 7, 2005 by Brown and Caldwell process engineers. Elevated SOR conditions were achieved by taking primary clarifiers out of service and treating more flow through the on-line clarifiers. Two step increases in SOR were performed, first by taking clarifier 2 out of service, followed by clarifier 3. The SOR was determined from plant flows and ranged from 800 to 1442 gpd/ft^2 during the testing. During the stress tests, COD, TSS, BOD_5 and $cBOD_5$ were measured. Samples were collected of primary influent and effluent, dispersed suspended solids and non-settleable suspended solids. The results of the stress tests are summarized in Table 2.

One of the objectives of the on-site stress tests was to determine if there are performance differences between the four primary clarifiers. Table 3 summarizes the influent and effluent concentrations measured with all primary clarifiers in service (SOR=800-849 gpd/ft^2). There was not a significant difference in COD and TSS removal between the tanks. However, during the stress testing, it was noted that clarifier 4 was receiving more flow based on visual inspection (water surface was higher in the effluent launder in clarifier 4 compared to clarifier 1). There were significant differences in $cBOD_5$ and BOD_5 removal, both between primary clarifiers and between sample duplicates for individual clarifiers. The COD and TSS results appear more reliable and are satisfactory enough to conclude that the performance in each tank is very similar. Because of the similarity between the COD and TSS data, clarifier analyses were performed using COD and TSS data rather than BOD_5 or $cBOD_5$ data.

In an ideal primary clarifier, the effluent TSS and COD concentrations will be equal to the non-settleable TSS and COD, respectively, plus the TSS and COD associated with those TSS that have settling velocities less than the SOR at the time of the sampling. The escaping TSS at the SOR during the tests (i.e., 800-849 gpd/ft^2) was expected to be very low. Indeed, results from Student's t-tests performed on the values in Table 2 for the base loaded conditions indicated no significant difference in between the TSS_{PE} and TSS_{non}. This result demonstrates that at these SORs, the HARRF primary clarifiers were performing as well as expected given the characteristics of HARRF's influent wastewater.

The TSS and COD removal efficiencies both decreased as SOR increased as expected (Figure 5). At the highest SOR (1377-1442 gpd/ft^2) which corresponds to an influent flow of 28.9-30.2 mgd with all clarifiers on-line, the water surface elevation in the effluent weirs was elevated indicating that the clarifier was approaching a hydraulically overloaded condition (Figure 6). However, the actual hydraulic capacity of the primary clarifiers was determined to be sufficient for the current plant rated peak flow of 36 mgd (see *Plant Hydraulic Profile Analysis Technical Memorandum)*. Large clumps of solids were observed in the effluent, possibly due to a rising sludge blanket, again indicating overloaded conditions and/or insufficient sludge withdrawal. As mentioned previously, Clarifier 4 received more flow than Clarifier 1 indicating flow imbalances. The rising sludge blanket observed during peak flow conditions can be mitigated by increasing primary sludge pumping. By increasing primary sludge production, there is less chance of solids carry over during the peak flows.

The data from the clarifier stress tests were analyzed to determine λ_{TSS} and TSS_{non}: 92.0 mg/L and 1,954 gpd/ft^2, respectively. These values agreed very well with the historically determined values of 92.2 mg/L and 2,347 gpd/ft^2.

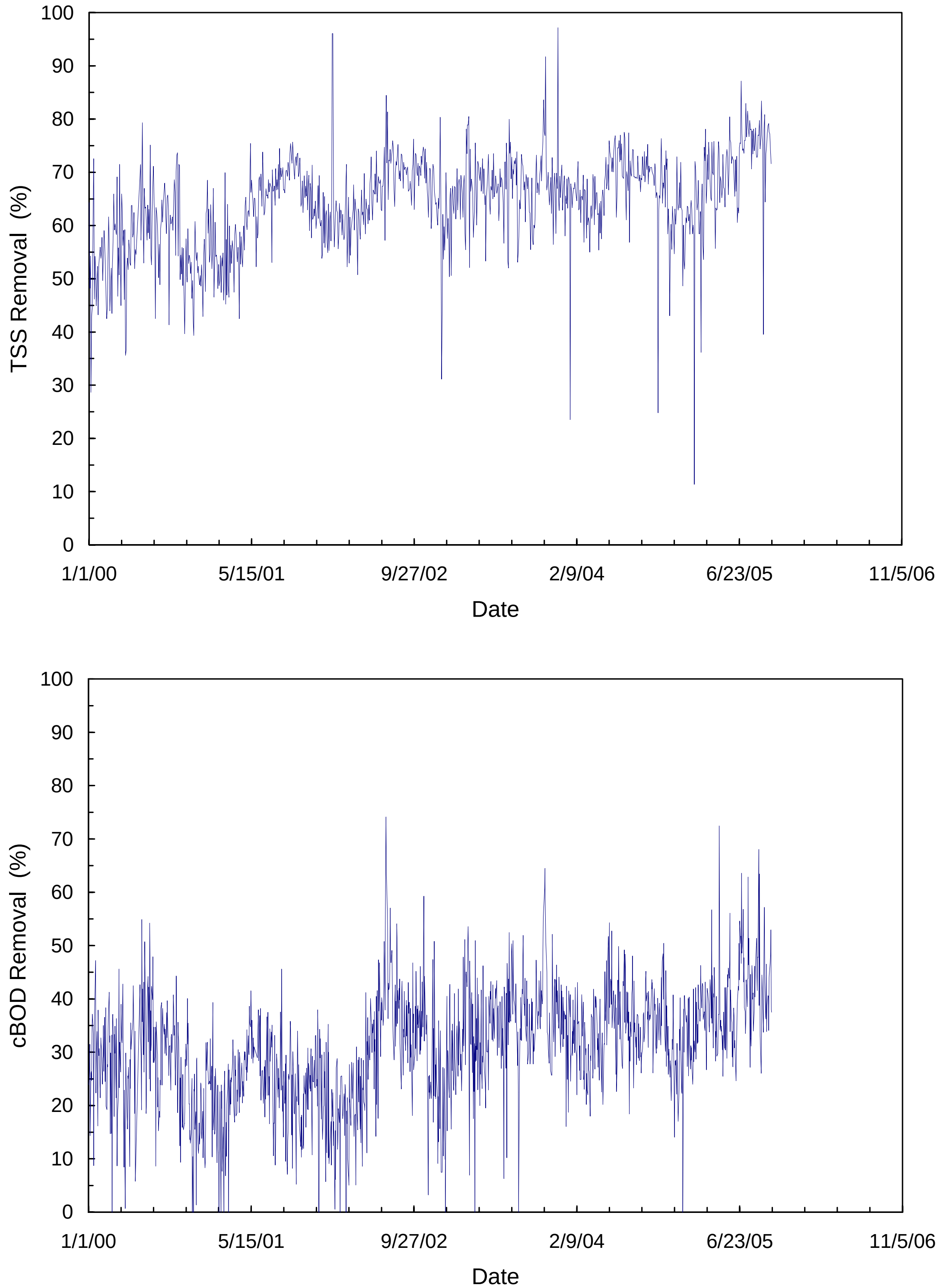


Figure 2. Historic TSS (top) and $cBOD_5$ (bottom) removal by HARRF primary clarifiers, January 1, 2000 to September 30, 2005.

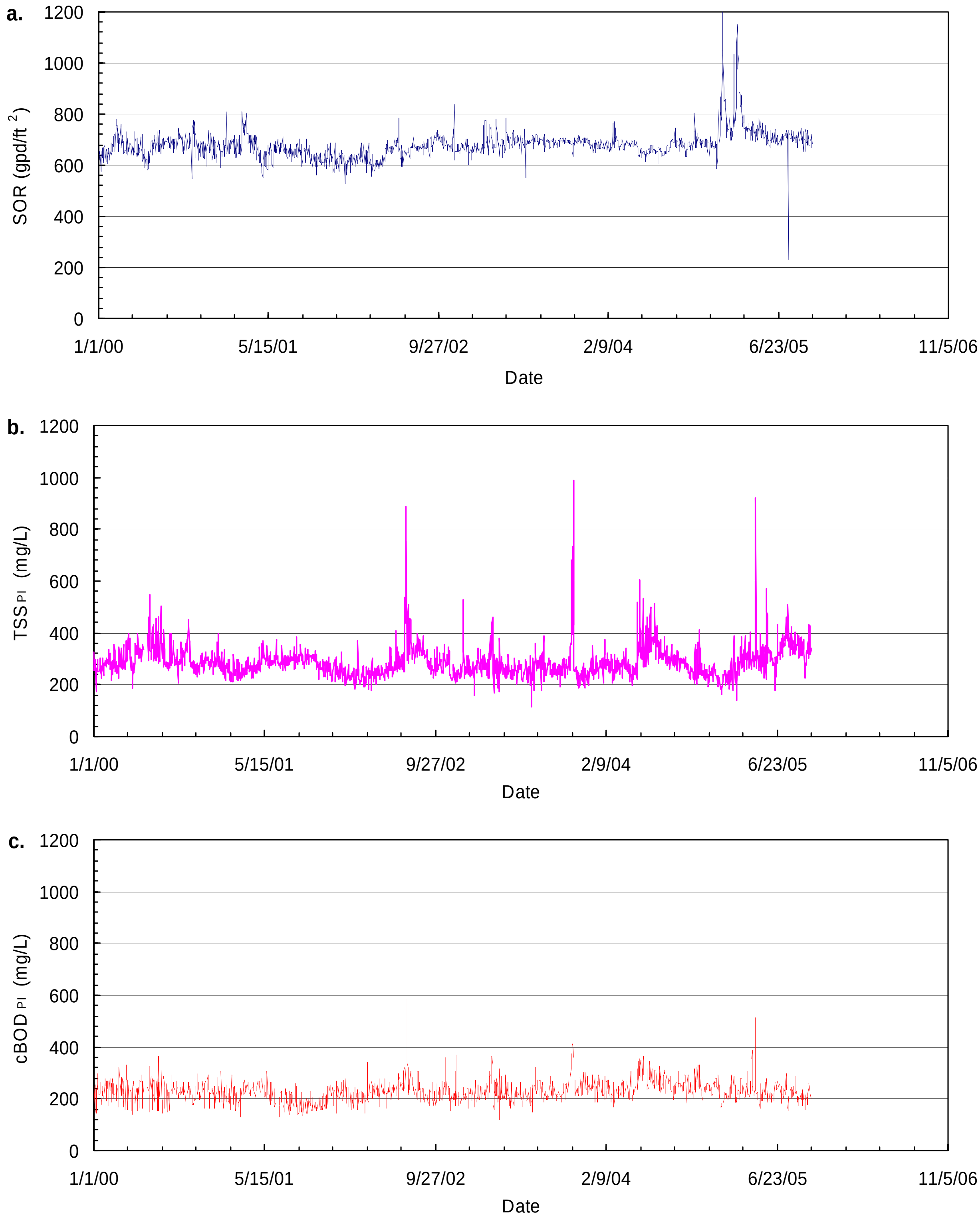


Figure 3. Historic (a) SOR, (b) TSS_{PI}, and (c) $cBOD_{PI}$ at HARRF, January 1, 2000 to September 30, 2005.

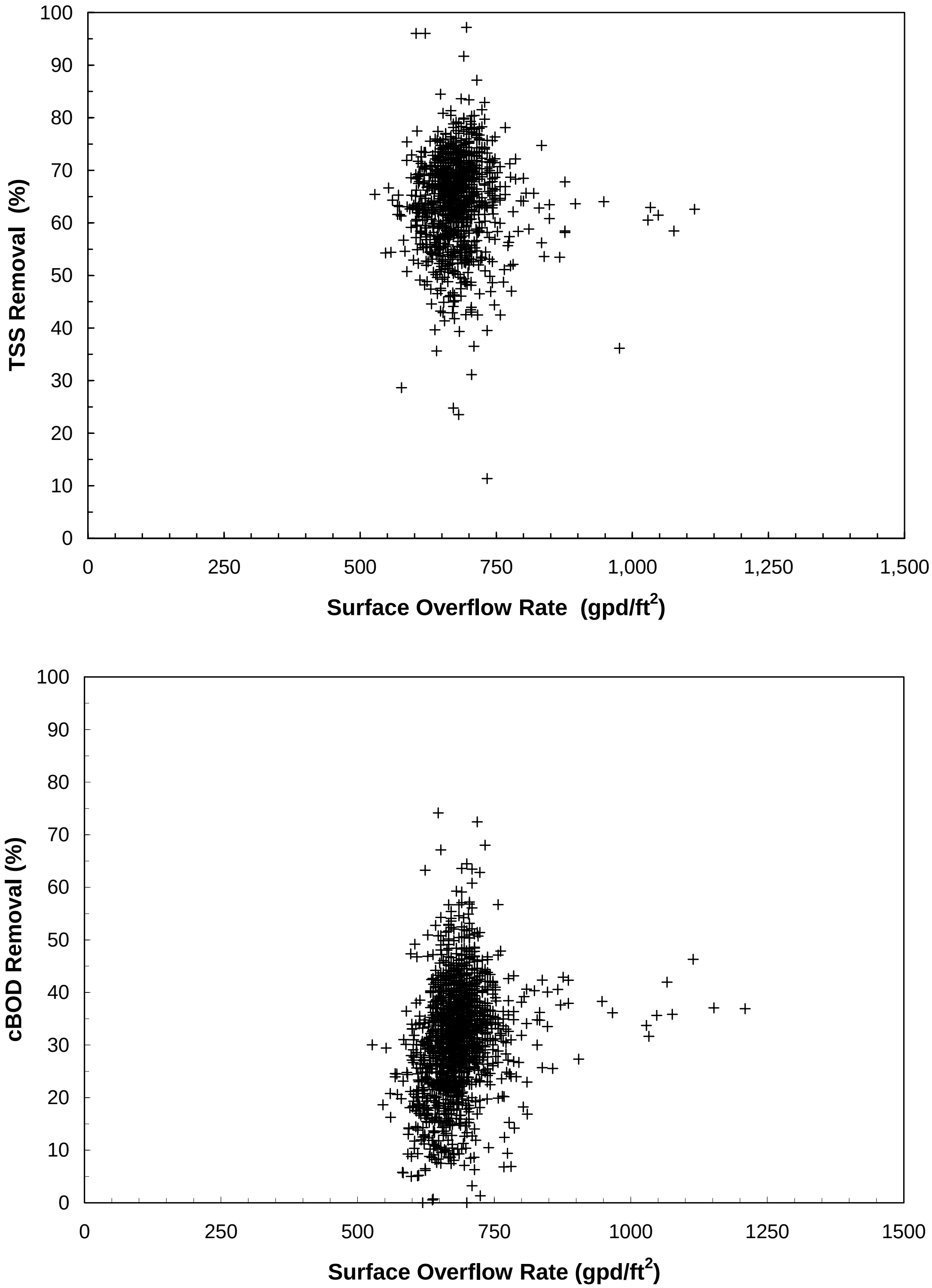


Figure 4. Effect of SOR on TSS (top) and $cBOD_5$ (bottom) removal efficiencies at HARRF, January 1, 2000 to September 30, 2005.

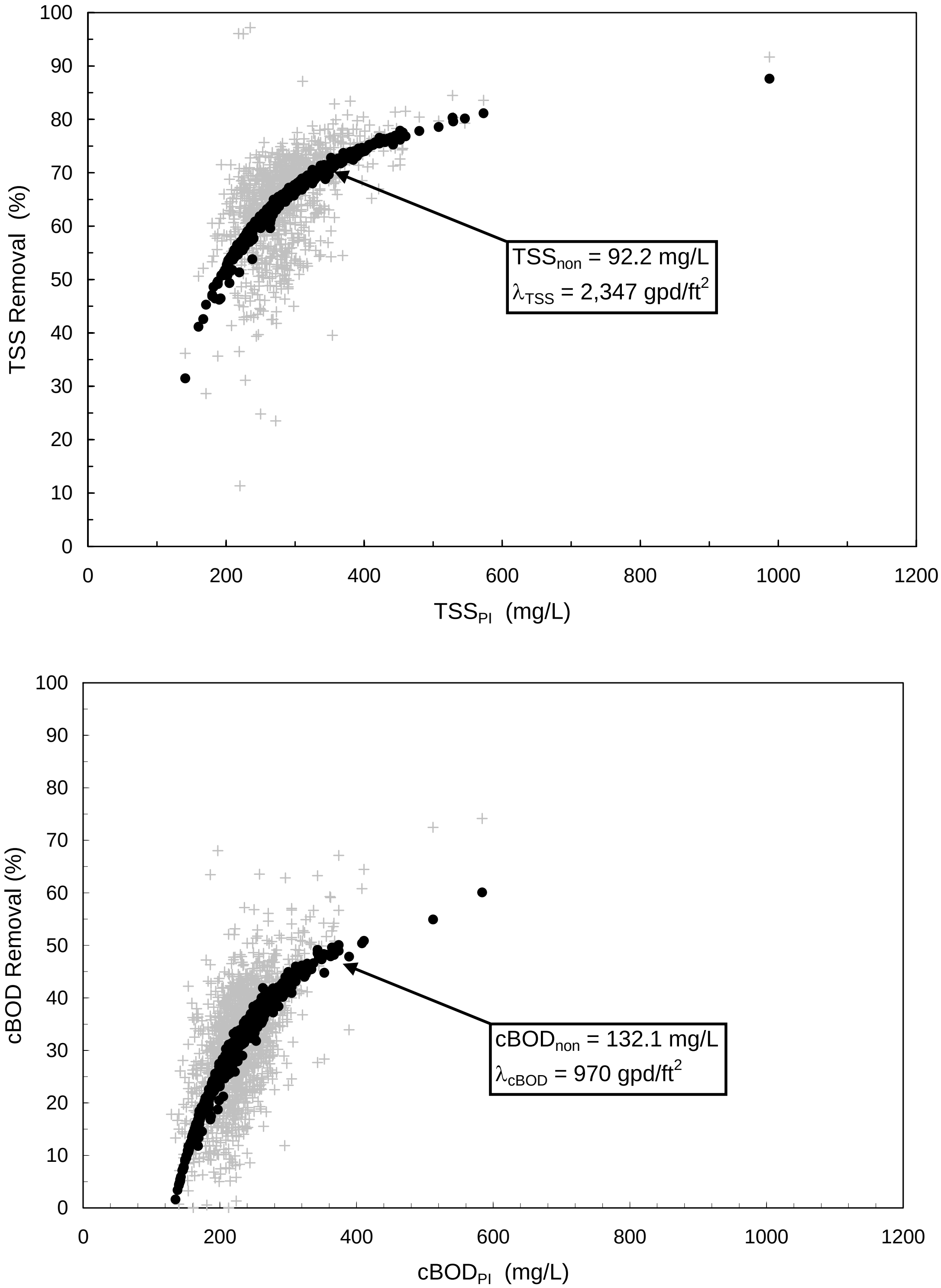


Figure 5. Fitting Equation 2 to historic data for TSS (top) and cBOD (bottom) removal.

Table 2. Results from on-site stress tests.

Test Condition	SOR, gpd/ft^2	Influent	Influent Dispersed	Non-Settleable	Effluent	Effluent Dispersed	Effluent Non-Settleable
TSS, mg/L							
All Clarifiers in Service	800-849	288	112	96	88		
		328	140	80	104		
					92		
					100		
					92		
					92		
					96		
					104		
Clarifier 2 out of Service	1007-1083	308	112	116	124	124	104
		264	160		108	104	
					152	116	96
					120	112	
Clarifiers 2 and 3 out of service	1377-1442	240	104	88	152	128	124
		244			148	124	
					124	108	80
					132	124	
cBOD, mg/L							
All Clarifiers in Service	800-849	186	133		149		
		178	143		168		
					137		
					141		
					134		
					174		
					132		
					149		
Clarifier 2 out of Service	1007-1083	288	193		234	89	
		244	167		111	121	
					243	114	
					157	93	
Clarifiers 2 and 3 out of service	1377-1442	270	182		222	152	
		239			229	202	
					230	217	
					270	189	
COD, mg/L							
All Clarifiers in Service	800-849	548	394		348		
		630	418		379		
					327		
					370		
					348		
					388		
					336		
					403		
Clarifier 2 out of Service	1007-1083	636	461		457	391	
		597	455		421	400	
					439	403	
					452	433	
Clarifiers 2 and 3 out of service	1377-1442	670	473		455	445	
		642			500	479	
					479	464	
					512	497	
BOD, mg/L							
All Clarifiers in Service	800-849	232	171		281		
		222	152		209		
					146		
					151		
					189		
					254		
					185		
					177		
Clarifier 2 out of Service	1007-1083	383	232		247	177	
		363	220		152	190	
					297	179	
					308	166	
Clarifiers 2 and 3 out of service	1377-1442	389	255		250	222	
		371			249	288	
					333	349	
					280	252	

Table 3. On-site data collected to determine differences in primary clarifier performance.

		Primary Clarifier 1 (Western-most Unit)		**Primary Clarifier 2 (West Central)**		**Primary Clarifier 3 (East Central)**		**Primary Clarifier 4 (Eastern-most Unit)**	
	Influent Conc. Mg/L	Effluent Conc., mg/L	Percent Removal	Effluent Conc., mg/L	Percent Removal	Effluent Conc., mg/L	Percent Removal	Effluent Conc., mg/L	Percent Removal
COD	548	348	36	327	40	348	36	336	39
	630	379	40	370	41	388	38	403	36
cBOD	186	149	**20**	137	**26**	134	**28**	132	**29**
	178	168	**6**	141	**21**	174	**2**	149	**16**
BOD	232	281	**-21**	146	**37**	189	**19**	185	**20**
	222	209	**6**	151	**32**	254	**-14**	177	**20**
TSS	288	88	69	92	68	92	68	96	67
	328	104	68	100	70	92	72	104	68

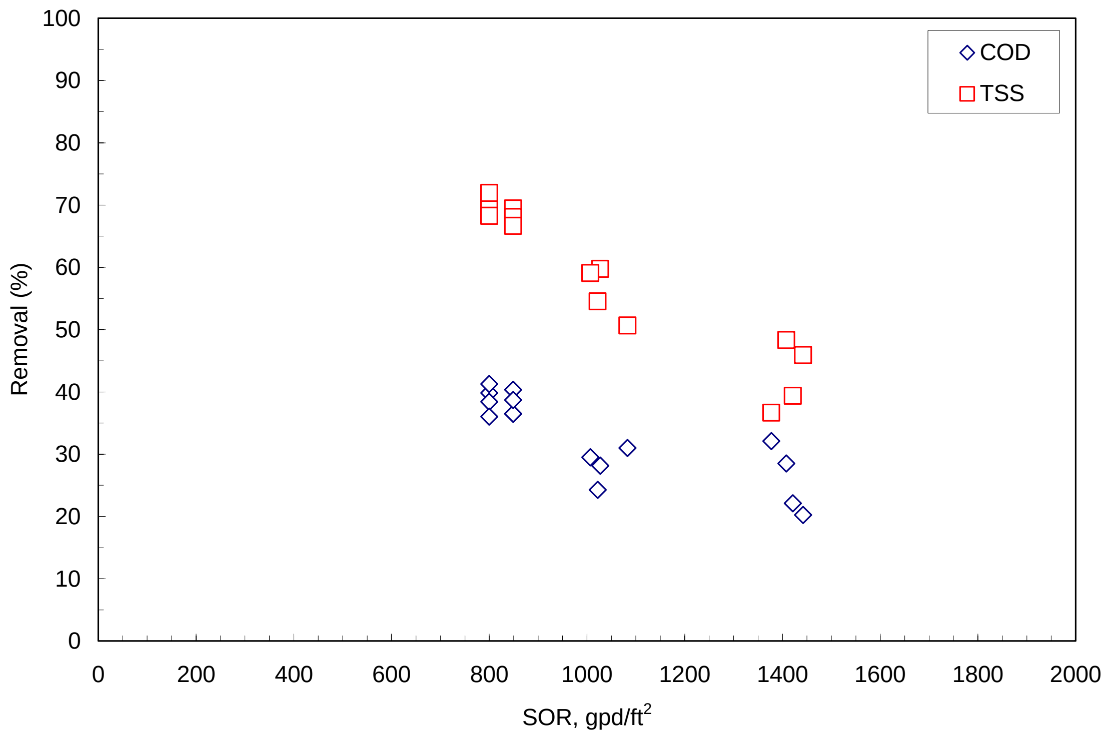


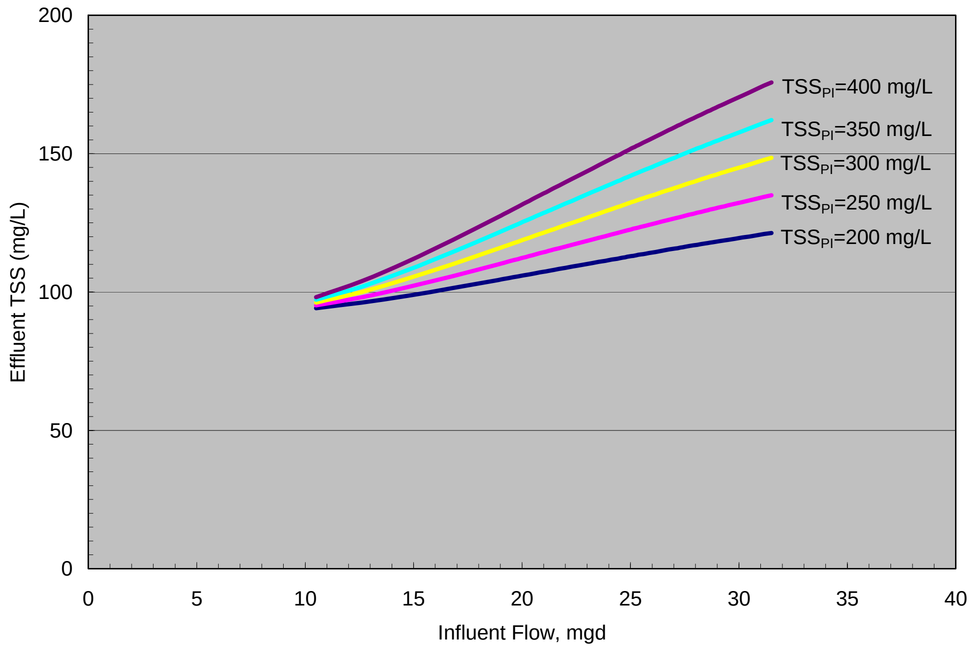
Figure 5. Results of on-site primary clarifier stress testing.

Figure 6. Primary clarifier effluent weirs with two clarifiers in service.
(Clarifier 4 shown, SOR=1377-1442 gpd/ft^2)

Capacity Assessment

Using the λ_{TSS} and TSS_{non} values determined from the on-site stress testing, the concentration of TSS in the primary clarifier effluent at increasing flows for varying influent TSS concentrations was determined. Performance graphs were created for conditions when all primary clarifiers are in service and when one is out of service (Figure 7). As discussed above, the most important parameter for determining the capacity of the primary clarifier/activated sludge couple is the primary effluent COD (or $cBOD_5$). On this account, COD performance curves, similar to the curves as shown in Figure 7, were developed (Figure 8). Results from the wastewater characterization study were used to convert the TSS performance curves to COD performance curves.

The capacity of the activated sludge system will be determined, through modeling, in terms of activated sludge influent COD concentration. Based on the data in Figures 7 and 8 and the influent characteristics, the capacity of the primary clarifier/activated sludge couple will be determined.



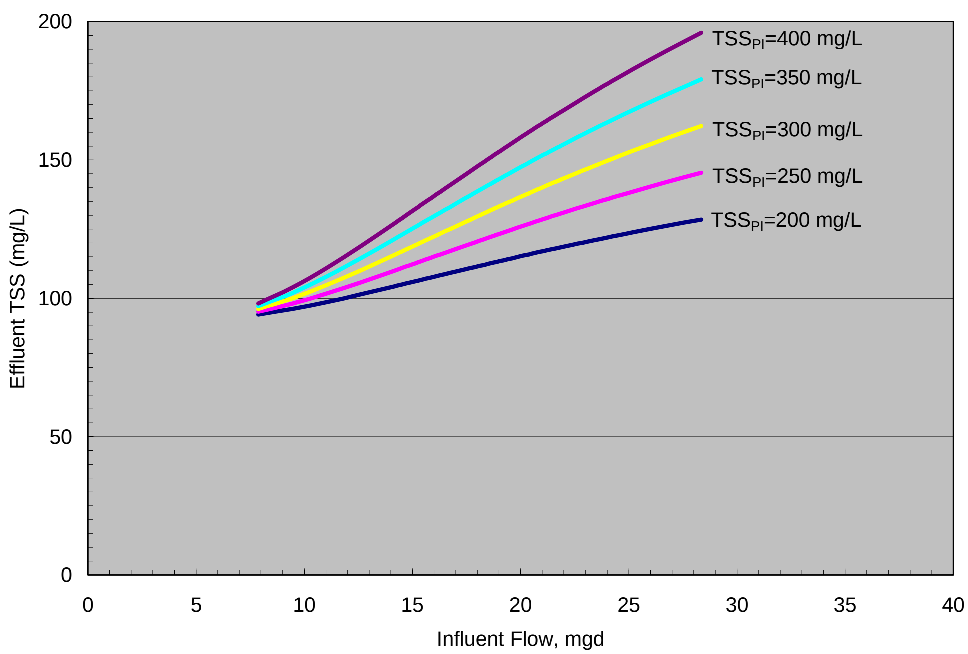


Figure 7. Performance curves of TSS removal for 4 on-line primary clarifiers (top) and 3 on-line primary clarifiers (bottom).

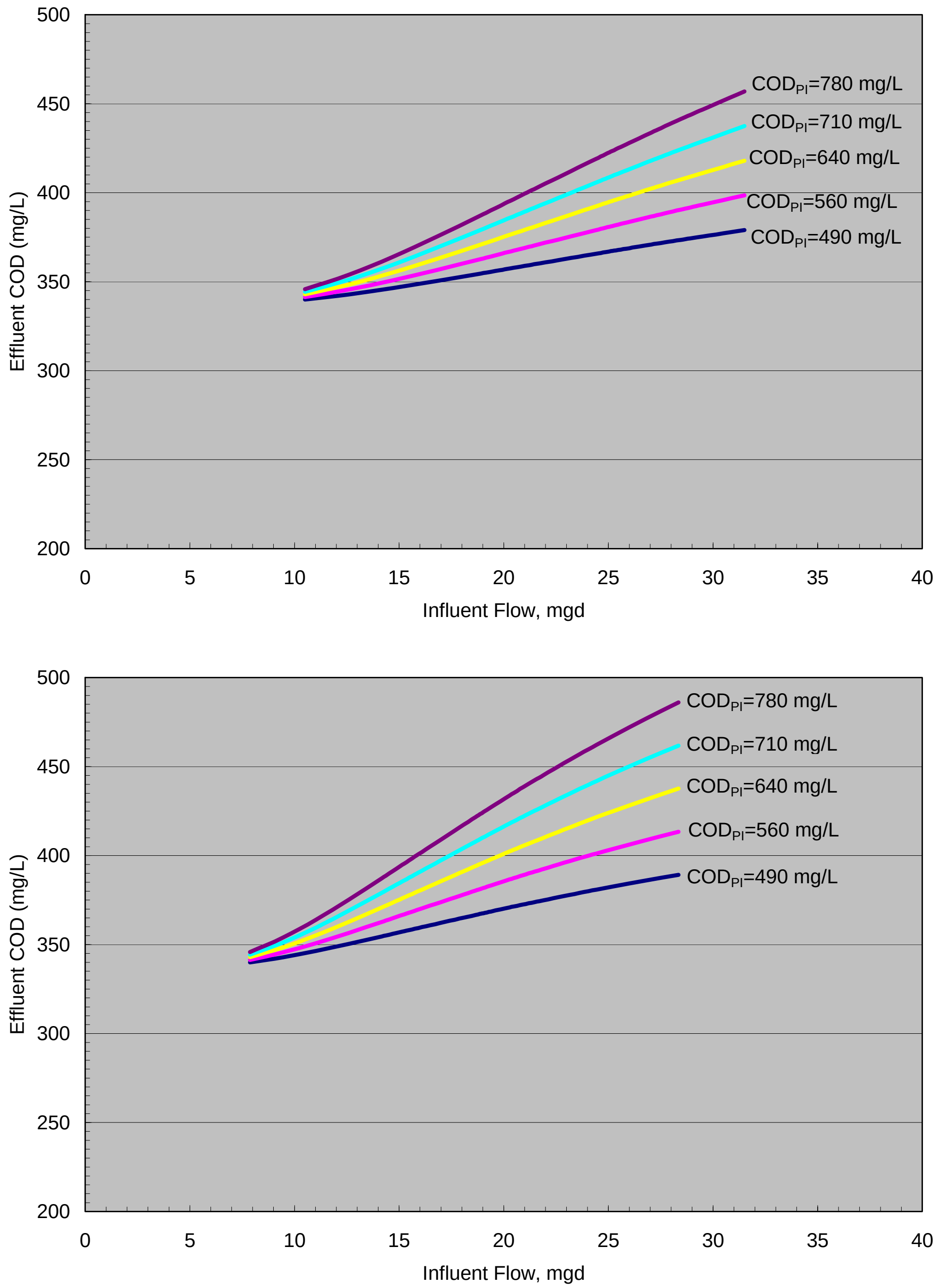


Figure 8. Performance curves of COD removal for 4 on-line primary clarifiers (top) and 3 on-line primary clarifiers (bottom).

APPENDIX D

BIOLOGICAL PROCESS EVALUATION TM

TECHNICAL MEMORANDUM –FINAL

DATE: AUGUST 10, 2006

TO: ANGELA MORROW, CITY OF ESCONDIDO

FROM: VICTOR OCCIANO, BROWN AND CALDWELL

PREPARED BY: RION MERLO, BROWN AND CALDWELL
RON APPLETON, BROWN AND CALDWELL
ERIC WAHLBERG, BROWN AND CALDWELL
SEVAL SEN, BROWN AND CALDWELL

SUBJECT: CITY OF ESCONDIDO
HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – BIOLOGICAL PROCESS EVALUATION

SUMMARY

The treatment capacity of the activated sludge process at the Hale Avenue Resource Recovery Facility (HARRF) was determined using the activated sludge simulator, BioWin™ 2.2. The hydraulic capacity of the activated sludge system was found to be sufficient for the existing rated capacity of 18.0 mgd (see *Plant Hydraulic Profile Analysis Technical Memorandum)*. The results of BioWin™ 2.2 model were combined with the state point analysis results used to determine secondary clarifier performance (see *Secondary Clarifier Settling Testing and Capacity Assessment Technical Memorandum)* to determine the existing process capacity of HARRF. The existing process capacity represents the capacity of the plant with the existing unit processes, and under the existing operating strategy. The capacity of the existing aeration system also was determined using results from off-gas aeration testing of the aeration basins.

A two-week wastewater characterization study was performed to determine model inputs for calibration. The characterization study results showed that there is unequal flow distribution between aeration basins attributed to uneven return activated sludge (RAS) flows and/or primary effluent flows to individual basins, and that the reported RAS flows are incorrect. In addition, the plant is not completely nitrifying, possibly due to one or a combination of the following: elevated RAS chlorination, high ammonia loads in solids recycle streams, or low dissolved oxygen (DO) concentrations in the aeration basins.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

The activated sludge model was calibrated using results from the wastewater characterization and then validated against historic data with good agreement. The calibrated model and the results of the secondary clarifier analysis were used to determine the process capacity of the plant under existing operating conditions. Process capacity was determined for dry weather and wet weather operating conditions using the 90th percentile SVI value (203 mL/g) and the average operating solids residence time (SRT) (2.75 d) determined from the historic data (2000 to 2005). These values represent the existing operating conditions of the plant and actual plant capacity may change with a change in operating conditions. For instance, if the SRT was increased, it is expected that the capacity would be reduced. Likewise, if the SRT were decreased, the capacity would be expected to increase.

The wet weather capacity of the plant was found to be governing plant capacity. Wet weather capacity, expressed in terms of average dry weather flow (ADWF) and assuming adequate air supply, was determined to be 14.8 mgd with all units in service. This conclusion is not in agreement with the plant capacity of 18 mgd determined in a Letter Report prepared by MWH. The disagreement arises from the design values used for the secondary clarifier solids loading rates assumed in the MWH Letter Report. The design solids loading rate used in the MWH Letter Report was 2.0 lb/sf-hr (48.0 lb/sf-hr). The solids loading rate used in this study, based on historic settling characteristics, was 1.25 lb/sf-hr (30.0 lb/sf-hr). High wet weather flows in January 2005 demonstrated that HARRF can treat flows greater than 14.8 mgd (expressed in terms of ADWF). However, the SVI during this high flow period ranged from 82 to 177 mL/g with an average value of 127 mL/g. This is significantly lower than the 90th percentile value (203 mL/g), determined from the 2000-2005 historic data, used to evaluate process capacity. Using the 90th percentile SVI value for 2005 (135 mL/g), the plant is capable of treating 18.4 mgd (expressed in terms of ADWF). As can be appreciated from this discussion, plant capacity (and, therefore plant performance) depends on a myriad of factors in addition to the size of the individual process units. The chance of the maximum flow occurring just after the maximum load occurred so the MLSS concentration is at its maximum during the worst sludge settling conditions with one secondary clarifier out of service is likely quite remote, but a possibility nonetheless.

As mentioned above, the dry weather and wet weather capacities were determined assuming the existing aeration system could deliver the oxygen required to accommodate the corresponding influent loadings. To determine the aeration system capacity, off-gas testing experiments were performed. The off-gas testing showed that the current α_F factor (ratio of oxygen transfer in dirty water to oxygen transfer in clean water) is 0.32. Based on the measured α_F, the existing blowers have an estimated capacity of 15.0 mgd. Additional aeration capacity could be realized if nitrification was eliminated by reducing the SRT. Operation at a 2-d SRT with an anaerobic selector could increase blower capacity to accommodate approximately 21.6 mgd ADWF capacity. The current fine-bubble aeration system (Parkson panels) were determined to not be sufficient to meet current or future demands and it is recommended that the diffusers be replaced with diffusers capable of operating at higher airflow rates.

For the Phase 1 improvements that have been identified for HARRF, an additional aeration basin and an additional primary clarifier will be constructed, the fine-bubble aerators will be replaced, the aeration basins will be modified to include anaerobic selector zones and the operating SRT will be reduced to 2 days (with the addition of mixed liquor wasting). With these

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

improvements, the capacity of HARRF is estimated to be approximately 18.9 mgd. The table below summarizes the results of the capacity analysis for HARRF.

Summary of Biological Capacity Assessment for HARRF

Operating Condition	Aeration Basin/Secondary Clarifier Capacity, mgd	Aeration System Capacity, mgd[a]	SVI, mL/g[b]	SRT, d
Existing Operating Conditions based on 2000-2005 historic data	14.8 (ADWF) 26.6 (PWWF)	15.0 (ADWF) 30.0 (PWWF)	203	2.75
Existing Operating Conditions based on 2005 historic data	18.4 (ADWF) 36.8 (PWWF)	15.0 (ADWF) 30.0 (PWWF)	135	2.75
Plant Capacity with Proposed Improvements	18.9 (ADWF) 37.8 (PWWF)	21.6 (ADWF) 43.2 (PWWF)	125[c]	2.00

a. Assuming existing fine-bubble aeration diffusers are replaced with diffusers capable of higher airflow rates
b. 90^{th} percentile value
c. Predicted Value with anaerobic selector

INTRODUCTION

The City of Escondido has engaged Brown and Caldwell to determine the capacity of the HARRF. One of the tasks of the Capacity Study is to evaluate the existing process capacity of the activated sludge system at HARRF using the activated sludge simulator, BioWin™ 2.2. As a part of this task, a two-week sampling program was conducted to collect input parameters for the simulator. The model was calibrated using the collected data and then validated using historic plant data. The purpose of this memorandum is to: (1) present the results of the characterization sampling, (2) present results of model calibration and validation, and (3) determine the process capacity of the activated sludge system.

MODEL CALIBRATION

Influent Flow

The HARRF currently receives an average dry weather flow (ADWF) of approximately 14.5 mgd. Most of the flow is domestic wastewater. There is approximately 1 mgd of industrial flow with a majority of this flow originating from a Sony facility (approximately 0.8 mgd). The flow from the Sony facility is treated for metals removal before discharge to the collection system. There is not a significant chemical oxygen demand (COD) or total suspended solids (TSS) contribution to HARRF as a result of the industrial loads; the main component of the industrial flows is dissolved solids. In addition to the industrial loads, the Escondido Water Treatment Plant (WTP) discharges ferric chloride sludge intermittently to the collection system.

Wastewater Characterization

A 2-week characterization study was conducted from December 8-21, 2005 to determine the characteristics of the HARRF influent. In BioWin™, the influent is divided into particulate and soluble fractions. Each fraction is further divided into biodegradable and non-biodegradable

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

portions. When performing a simulation, the values for COD, phosphorus, total Kjeldahl nitrogen (TKN), and inert suspended solids (ISS) are inputted. Ammonia and 5-day biochemical oxygen demand (BOD_5) are derived from the TKN and COD, respectively. The defined fractions divide the influent into the appropriate fractions. The characterization study was performed to determine the following fractions:

fbs	=	fraction of COD that is readily biodegradable
fac	=	fraction of COD that is acetate
fus	=	fraction of COD that is unbiodegradable soluble
fna	=	fraction of TKN that is ammonia
fnox	=	fraction of organic nitrogen that is particulate
fpo4	=	fraction of phosphorus that is phosphate

Equations 1 through 6 show how each fraction is calculated.

$$fbs = (ffCOD_{inf}-ffCOD_{eff})/COD_{inf} \quad (1)$$
$$fac = VFA/(fbs*COD_{inf}) \quad (2)$$
$$fus = SCOD_{eff}/COD_{inf} \quad (3)$$
$$fna = NH4\text{-}N_{inf}/TKN_{inf} \quad (4)$$
$$fnox = (TKN_{inf}-STKN_{inf})/(TKN_{inf}-NH4\text{-}N_{inf}) \quad (5)$$
$$fpo4 = PO4\text{-}P_{inf}/P_{inf} \quad (6)$$

where

$ffCOD_{inf}$	=	influent flocculated filtered COD, mgCOD/L
$ffCOD_{eff}$	=	effluent flocculated filtered COD, mgCOD/L
VFA	=	primary effluent volatile fatty acid, mgCOD/L
COD_{inf}	=	influent COD, mgCOD/L
$SCOD_{eff}$	=	effluent soluble COD, mgCOD/L
$NH4\text{-}N_{inf}$	=	influent ammonia, mg-N/L
TKN_{inf}	=	influent TKN, mg/L
$STKN_{inf}$	=	influent soluble TKN, mg/L
$PO4\text{-}P_{inf}$	=	influent orthophosphate, mg-P/L
P_{inf}	=	influent total phosphorus, mg/L

Samples also were collected from the primary effluent, secondary effluent, mixed liquor, dewatering centrate, and DAFT subnatant and analyzed for various constituents.

Total Suspended Solids (TSS)

Table 1 shows the TSS concentration of the influent and primary effluent. The influent TSS ranged from 104 to 328 mg/L and primary effluent ranged from 58 to 128 mg/L. A primary clarifier TSS removal efficiency of 63 percent was used for the model based on the results in Table 1. The ISS values ranged from 32 to 84 mg/L with a median value of 52 mg/L.

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 1. Suspended solids content from wastewater characterization.

Day	Influent TSS, mg/L	Influent VSS, mg/L	Influent ISS, mg/L	Primary Effluent TSS, mg/L	Primary Clarifier TSS Removal, percent
12/8/2005	232	214	18	88	62
12/9/2005	308	250	58	124	60
12/10/2005	284	200	84	112	61
12/11/2005	248	222	26	90	64
12/12/2005	324	276	48	108	67
12/13/2005	292	256	36	---	---
12/14/2005	324	264	60	100	69
12/15/2005	260	196	64	100	62
12/16/2005	308	248	60	98	68
12/17/2005	216	164	52	66	69
12/20/2005	104	72	32	58	44
Median Value	**284**	**222**	**52**	**99**	**63**

As indicated above, the WTP discharges sludge to the collection system. Table 2 summarizes the HARRF influent TSS loading and the estimated levels of solids originating from the WTP during the characterization study period. Sludge loading from the WTP was estimated using equation 7*. The results in Table 2 show that the WTP sludge contributed about 6 percent of the influent TSS loading.

* Adapted from MWH (2005) *Water Treatment: Principles and Design, Second Edition*. John Wiley & Sons, Inc., Hoboken, NJ.

Table 2. Estimated TSS loading to HARRF from WTP sludge.

Day	HARRF Influent TSS, mg/L	HARRF Influent Flow, mgd	WTP Influent Flow, mgd	WTP Influent Turbidity, NTU	WTP Ferric Chloride Dose, mg/L	WTP Sludge Flow, mgd	Estimated WTP Sludge, lb/d	Influent TSS Loading, lb/d	Fraction WTP sludge
12/8/2005	232	13.70	34.87	1.05	8.42	0.07	1557	26508	0.059
12/9/2005	308	13.60	38.62	1.09	8.49	0.08	1751	34935	0.050
12/10/2005	284	13.90	32.74	1.03	8.22	0.09	1429	32923	0.043
12/11/2005	248	14.50	32.80	1.11	8.21	0.09	1458	29991	0.049
12/12/2005	324	14.10	38.17	1.07	8.59	0.08	1738	38100	0.046
12/13/2005	292	13.90	37.54	1.09	8.47	0.08	1699	33850	0.050
12/14/2005	324	13.90	40.73	2.48	13.1	0.10	3189	37560	0.085
12/15/2005	260	13.80	33.82	3.55	16.5	0.09	3486	29924	0.116
12/16/2005	308	13.20	37.14	3.18	17.53	0.09	3838	33907	0.113
12/17/2005	216	13.60	33.32	3.1	16.9	0.09	3331	24500	0.136
12/18/2005	---	13.90	28.85	3.18	18.66	0.09	3112	---	---
12/19/2005	---	13.90	38.70	3.08	16.82	0.10	3848	---	---
12/20/2005	104	14.00	40.10	3.08	19.77	0.09	4461	12143	0.367
12/21/2005	---	13.80	37.90	3.1	20.53	0.09	4340	---	---
Median Value	**284**	**13.90**	**37.34**	**2.78**	**14.80**	**0.09**	**3150**	**32923**	**0.059**

$$TSS_{Loading}=Q_{WTP}*8.34*(C_{NTU}*CF_{NTU}+C_{ferric}*CF_{ferric}) \qquad (7)$$

where

$TSS_{Loading}$	=	Loading due to WTP sludge, lb/d
Q_{WTP}	=	WTP influent flow, mgd
C_{NTU}	=	WTP influent turbidity, NTU
C_{ferric}	=	Ferric chloride dose, mg/L
CF_{NTU}	=	Conversion factor, 1.25 gTSS/NTU
CF_{ferric}	=	Conversion factor, 0.48 g sludge/g ferric dose

Chemical Oxygen Demand (COD)

The COD content measured during the wastewater characterization is shown in Table 3. Using the results in Table 3, the fractions fbs, fus and fac were determined to be 0.179, 0.085 and 0.417, respectively. These values are within the range typical of domestic wastewater.

Table 3. COD values from wastewater characterization.

Day	Influent COD, mg/L	Influent SCOD, mg/L	Influent ffCOD, mg/L	Prim. Eff. VFA, mgCOD/L	Sec. Eff. COD, mg/L	Sec. Eff. SCOD, mg/L	Sec. Eff. ffCOD, mg/L	fbs	fus	fac
12/8/2005	585	191	130	44.5	48.5	42.4	21.2	0.186	0.072	0.409
12/9/2005	555	188	124	51.7	42.4	24.2	24.2	0.180	0.044	0.518
12/10/2005	542	188	118	43.4	36.4	33.3	18.2	0.184	0.061	0.435
12/11/2005	618	224	158	---	66.7	57.6	36.4	0.197	0.093	0.000
12/12/2005	642	227	155	42.5	55	61	42.4	0.175	0.095	0.378
12/13/2005	321	239	161	---	---	---	---	---	---	---
12/14/2005	618	230	161	45.6	75.8	57.6	39.4	0.197	0.093	0.375
12/15/2005	545	194	121	39.0	64	36.4	24.2	0.178	0.067	0.403
12/16/2005	636	206	136	44.9	60.6	48.5	30.3	0.166	0.076	0.424
12/17/2005	612	185	130	37.1	124	109	100.0	0.049	0.178	1.236
12/20/2005	576	179	136	55.8	45.5	54.5	33.3	0.178	0.095	0.543
Median Value	**585**	**194**	**136**	**45**	**58**	**52**	**32**	**0.179**	**0.085**	**0.417**

Nitrogen and Phosphorus

The nitrogen and phosphorus levels from the characterization study are shown in Table 4. The fna, fnox and fpo4 fractions were calculated to be 0.692, 0.636 and 0.291, respectively. The fna and fnox values are within the range typical of municipal wastewater. The fpo4 value, 0.291, is lower than typical (approximately 0.5). The reduced fpo4 level is attributed to the ferric chloride in the WTP sludge, which will remove soluble phosphate by chemical precipitation. A lowered level of phosphate relative to the total phosphorus will result in a reduced fpo4 value.

Table 4. Influent nitrogen and phosphorus values from wastewater characterization.

Day	Influent TKN, mg/L	Influent Soluble TKN, mg/L	Influent Ammonia, mg-N/L	Influent Total P, mg/L	Influent Orhtho-Phosphate, mg-P/L	fna	fnox	fpo4
12/8/2005	47.5	37.7	35.0	5.80	2.00	0.737	0.784	0.345
12/9/2005	39.1	37.6	33.8	8.00	2.00	0.864	0.283	0.250
12/10/2005	43.4	39.3	32.8	7.50	---	---	0.387	---
12/11/2005	50.3	40.7	33.6	7.40	2.20	0.668	0.575	0.297
12/12/2005	46.5	35.8	30.7	5.60	2.40	0.660	0.677	0.429
12/13/2005	92.3	15.2	27.6	3.50	1.90	0.299	---	0.543
12/14/2005	54.9	39.9	32.4	7.40	2.00	0.590	0.667	0.270
12/15/2005	50.9	41.7	35.7	7.80	1.80	0.701	0.605	0.231
12/16/2005	53.3	42.1	47.6	7.40	1.80	0.893	---	0.243
12/17/2005	52.3	39.5	36.2	8.10	---	0.692	0.795	---
12/20/2005	44.7	35.6	---	5.50	1.60	---	---	0.291
Median Value	**50.3**	**39.3**	**33.7**	**7.4**	**2.0**	**0.692**	**0.636**	**0.291**

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Diurnal Flow and Concentration Variations

As a part of the wastewater characterization, influent water quality data were collected every two hours for one week day and one weekend day. The diurnal profile is necessary to estimate the oxygen requirements in the aeration basins and to determine the diurnal flow variations in the secondary clarifiers. Table 5 summarizes the variation in influent flows and constituents throughout the day. The highest flows were observed around 11:45 for both days. The COD concentration peaked at 13:45 and 15:45 for the weekend day and week day, respectively. The highest TKN concentration peak occurred between 7:45 and 9:45 on both the week day and weekend day. The TSS peak concentration occurred at 1:45 for both the week day and weekend day.

Table 5. Influent diurnal flow and concentration variations.

	Weekend Day Peaking Factor				Week Day Peaking Factor			
Time	Flow	COD Conc.	TKN Conc.	TSS Conc.	Flow	COD Conc.	TKN Conc.	TSS Conc.
9:45	1.33	1.04	1.05	0.59	1.26	0.92	1.22	0.51
11:45	1.44	1.00	0.63	1.09	1.32	1.24	0.99	1.34
13:45	1.30	1.98	0.77	0.43	1.21	1.00	0.80	0.45
15:45	1.22	0.96	0.71	0.43	1.14	2.21	0.79	0.77
17:45	1.17	1.19	1.01	1.28	1.23	0.72	1.06	0.54
19:45	1.14	0.92	0.49	0.99	1.27	1.09	1.10	1.28
21:45	1.06	0.89	1.97	0.90	1.14	0.83	1.08	1.09
23:45	0.81	0.85	1.19	0.33	0.82	0.76	0.86	0.16
1:45	0.55	0.92	1.24	2.13	0.56	0.85	1.02	2.11
3:45	0.43	0.68	1.08	1.51	0.42	0.97	0.98	1.66
5:45	0.53	0.68	0.68	0.47	0.57	0.65	0.84	0.83
7:45	1.02	0.89	1.18	1.85	1.05	0.77	1.27	1.28

Impact of Recycle Streams

Table 6 summarizes the measured COD, TSS and TKN values measured for the influent, DAFT subnatant and dewatering centrate during the wastewater characterization study (subnatant and centrate samples were collected as grab samples). The influent sample includes the impacts of the recycle streams. The TSS and COD loading resulting from the recycle streams are significantly less than the overall influent loading. The centrate contributes approximately 740 lb/d of TKN, but it is still much less than the overall 5,900 lb/d in the influent. However, as noted above, the subnatant and centrate samples were grab samples and the influent was a composite sample. As a result, there may be some difference in recycle stream loading.

Table 6. Impact of recycle streams on influent loading.

Location	Flow, mgd	TSS, mg/L	COD, mg/L	TKN, mg/L	TSS Loading, lb/d	COD Loading, lb/d	TKN Loading, lb/d
Influent	13.8	264	568	52	30,380	65,370	5,990
DAFT Subnatant	0.44	45	133	15	170	490	55
Centrate	0.11	311	786	803	290	720	740

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Flow Distribution through Aeration Basins

The mixed liquor suspended solids (MLSS) concentration in each aeration basin was measured to determine if any significant flow imbalances exist caused by the RAS flow or primary effluent flow. The measured values showed that there are significant differences in MLSS concentrations in each of the 5 basins (Figure 1). Basin 1 had the highest MLSS levels and Basins 3 and 5 had the lowest. The RAS flow to each basin is controlled by individual valves and may be the cause of the differences in MLSS concentration. Improper flow distribution will cause an imbalance in loading to each aeration basin. Differences in loading between basins will result in varying degrees of treatment occurring in each basin and may reduce effluent water quality and capacity.

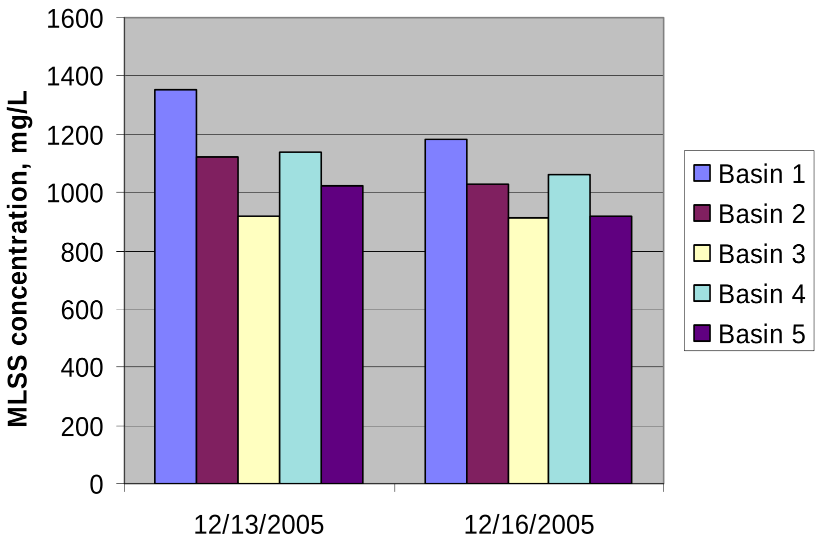


Figure 1. MLSS concentration in each aeration basin.

Model Calibration

Figure 2 shows the BioWin™ configuration for HARRF. The aeration basins were modeled as three tanks in series representing the three zones in each as defined by the aeration system. The solids processing portion of the plant also was modeled including the DAFTs, digesters, and centrifuges with corresponding recycle streams. Recycle streams were assumed to be returned consistently to the influent (in actuality, thickening and dewatering are intermittent operations). The model does not account for the tertiary filter backwash. The model was calibrated using data collected during the wastewater characterization sampling.

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

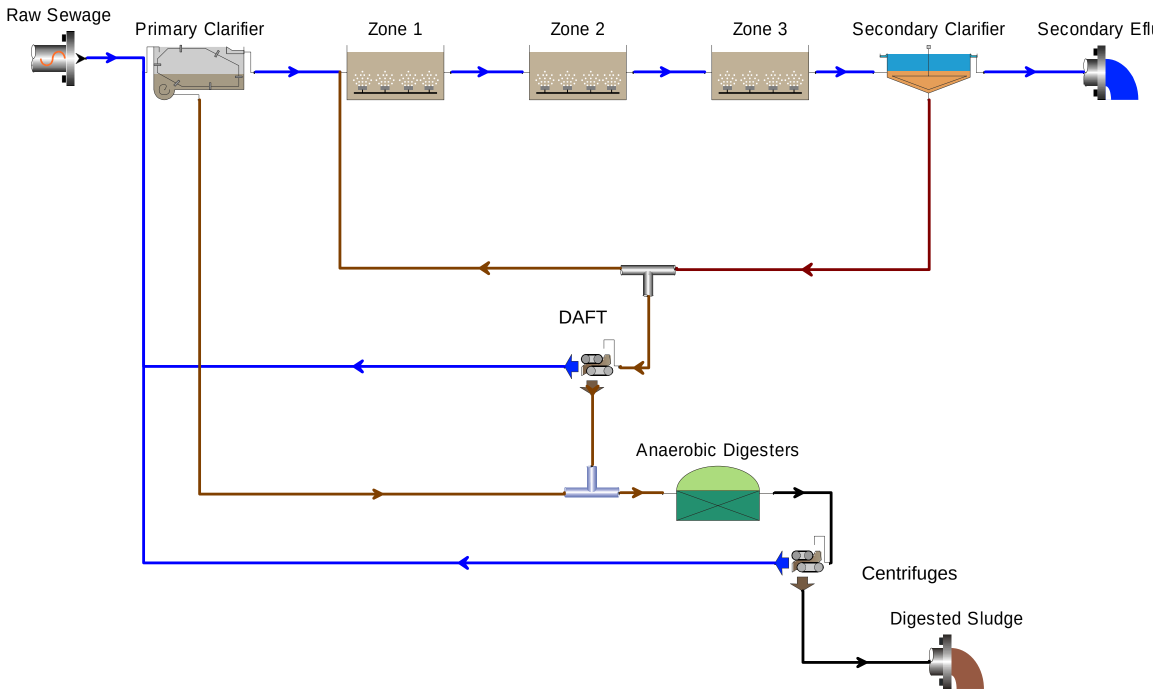


Figure 2. Screenshot of BioWin™ model for HARRF.

Composite Data Calibration

The fractions determined from the wastewater characterization were entered into the model (Tables 3 and 4). A two-week influent itinerary was generated representing the influent water quality of the composite samples collected during the wastewater characterization. The solids capture efficiencies for the DAFT and centrifuges were determined from historic data and were entered into the model. Daily flow data for the influent, primary sludge, waste activated sludge (WAS), RAS, thickened sludge, and dewatered sludge were entered into the model. The RAS flows reported by the plant flow meter did not agree with the mass-balance determined flow based on the measured RAS and MLSS suspended solids concentrations. In general, the measured RAS flows were less than the calculated flows. The RAS flows were adjusted to reflect the measured RAS suspended solids concentration.

The DO concentration data for the aeration basins were not available for the calibration, so the DO was set at 2 mg/L. It should be noted that, according to plant staff, the DO in the aeration basins is not always 2 mg/L and is usually lower. For the model calibration, 2-mg/L DO was used for model stability.

The primary clarifier removal efficiency was set at 63 percent (from data in Table 1). Model calibration was verified by comparing predicted values to measured values for MLSS, effluent water quality, dewatering centrate, DAFT subnatant, and primary effluent water quality. Figure 3 shows the model-predicted and measured COD, TSS and VSS content of the MLSS. The model was in good agreement with the measured values.

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

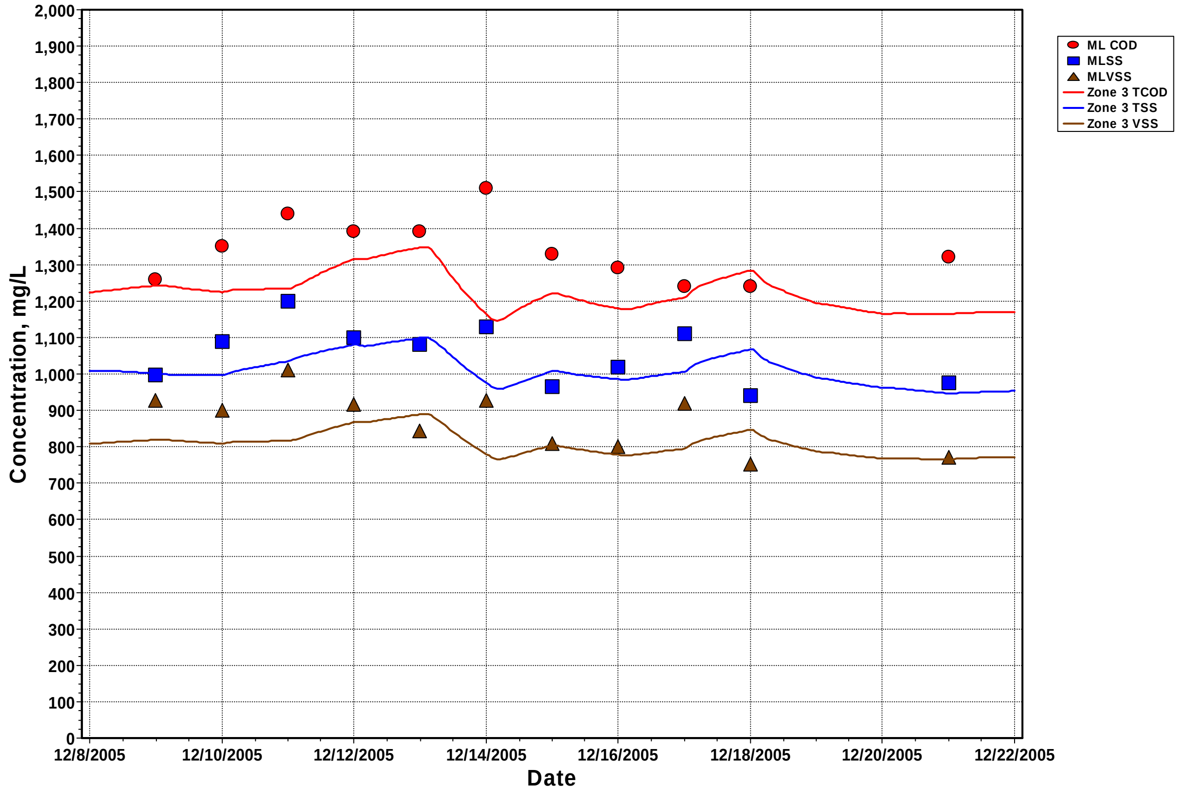


Figure 3. Model-predicted and observed values for COD, TSS and VSS of mixed liquor during model calibration.

Figure 4 shows the model-predicted and plant-reported solids retention time (SRT). There was a discrepancy between the two values, attributed to a difference in calculation. The plant reported SRT includes the volume of the aeration basins plus the secondary clarifiers to determine solids inventory, whereas the model-predicted values includes only the volume of the aeration basins for the inventory calculation.

The SRT (either model-predicted or plant reported) and wastewater temperature (22 degrees C) are sufficient to promote nitrification which will result in reduced (less than 2 mg-N/L) levels of ammonia in the effluent, as was predicted by the model. However, the measured ammonia levels ranged from 0.2 to 8.8 mg-N/L, indicating partial nitrification. The incomplete nitrification is attributed to one or all of the following: reduced DO levels in the aeration basins causing suppression of nitrifying bacteria, inhibition of nitrifying bacteria due to RAS chlorination, or elevated peak ammonia loading resulting from recycle streams.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

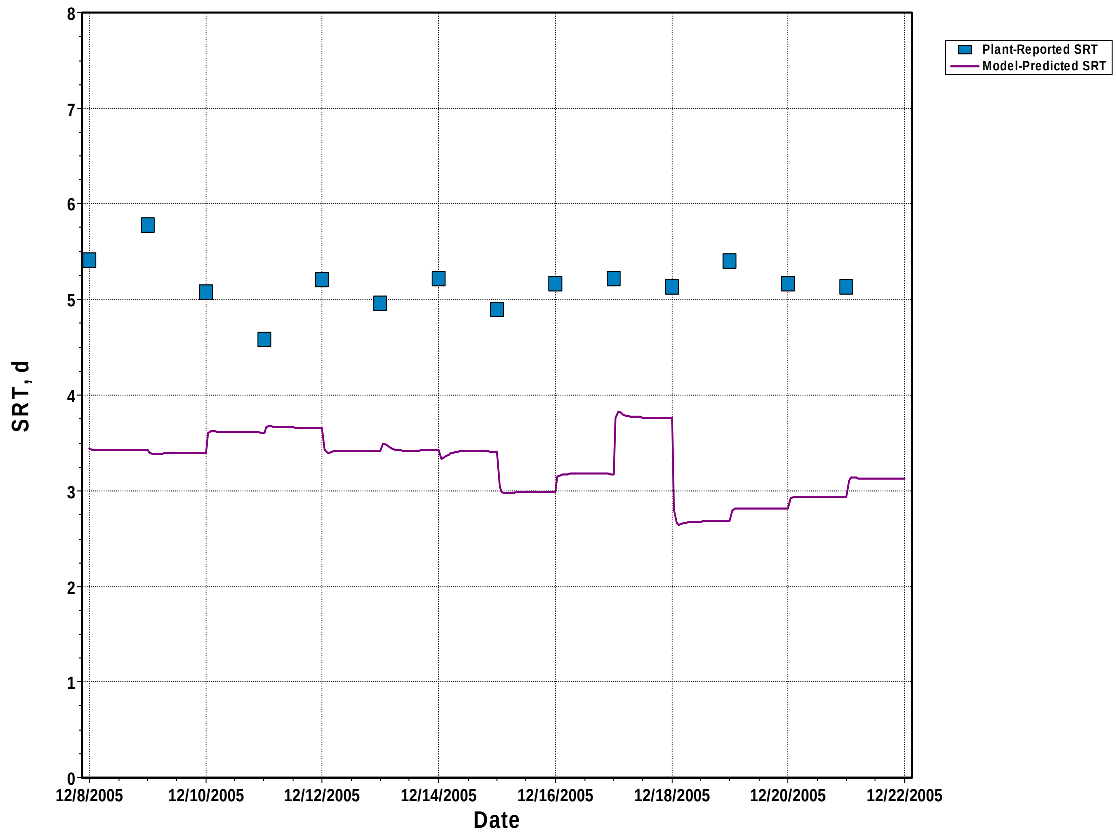


Figure 4. Model-predicted and plant-reported SRT values during model calibration.

Aeration Requirements

Typically, the activated sludge simulator is calibrated using airflow data and measured DO concentrations in the aeration basins to predict aeration requirements. For HARRF, the airflows sent to each of the three aeration basin zones are not monitored, which made model calibration infeasible. For the modeling, the activated sludge was assumed to be a nitrifying system with an alpha value of 0.65 and a standard oxygen transfer efficiency (SOTE) of 35.2 percent. The actual alpha value may be lower than 0.65 and will depend on age of the diffuser and/or degree of fouling. The model-predicted oxygen uptake rates (OUR) in each aeration zone were used to estimate aeration requirements using these assumed alpha and SOTE values.

Model Validation

After model calibration, the model accuracy was verified with a validation step. Plant data for July to September 2005 were used as inputs to the calibrated model, and a dynamic simulation was performed. Because the plant does not measure COD, the influent $cBOD_5$ values were used to estimate COD values using the ratio determined from the wastewater characterization data (3.32 g COD/g $cBOD_5$). The COD-to-$cBOD_5$ ratio is higher than for typical wastewater (approximately 2.1 gCOD/g $cBOD_5$). This is attributed to recycle streams at HARRF increasing the amount of non-

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

biodegradable material. In addition, $cBOD_5$ measurements can oftentimes under predict organic content due to the addition of a nitrification inhibitor in influent $cBOD_5$ tests, because the nitrification inhibitor also inhibits carbon-oxidizing microorganisms. Influent TKN and ISS values also were based on determined fractions. The plant-reported flow rates were entered into the model, and as before, the RAS flows were adjusted based on the mass-balance determined values.

The primary clarifier TSS removal efficiency during July to September was determined to be 77 percent, significantly higher than removals observed during model calibration (63 percent). The improved removal efficiency is attributed to higher levels of WTP sludge. The WTP sludge is expected to settle out in the primary clarifiers and the increased loading will result in elevated removal efficiency. Table 7 shows that during the validation, the WTP sludge accounted for a significantly higher amount of the influent TSS loading than during the validation - a median value of 0.23 for validation versus 0.06 for calibration.

Table 7. Summary of influence of WTP sludge on HARRF influent TSS loading.

	Median Value	Minimum Value	Maximum Value
WTP Influent Flow, mgd	52.8	23.5	62
WTP Influent Turbidity, NTU	3.49	0.72	6.85
WTP Ferric Chloride Dose, mg/L	33.4	11.0	45.0
WTP Sludge Flow, mgd	0.12	0.1	0.23
Estimated WTP Sludge, lb/d	9,070	2,350	11,873
HARRF Influent TSS Loading, lb/d	37,622	13,881	57,049
Fraction WTP sludge	0.23	0.08	0.7

Figure 5 shows the model-predicted and plant-reported values for the MLSS and mixed liquor volatile suspended solids (MLVSS). The model-predicted MLSS and MLVSS concentrations were in good agreement with the measured values, and indicated a calibrated model.

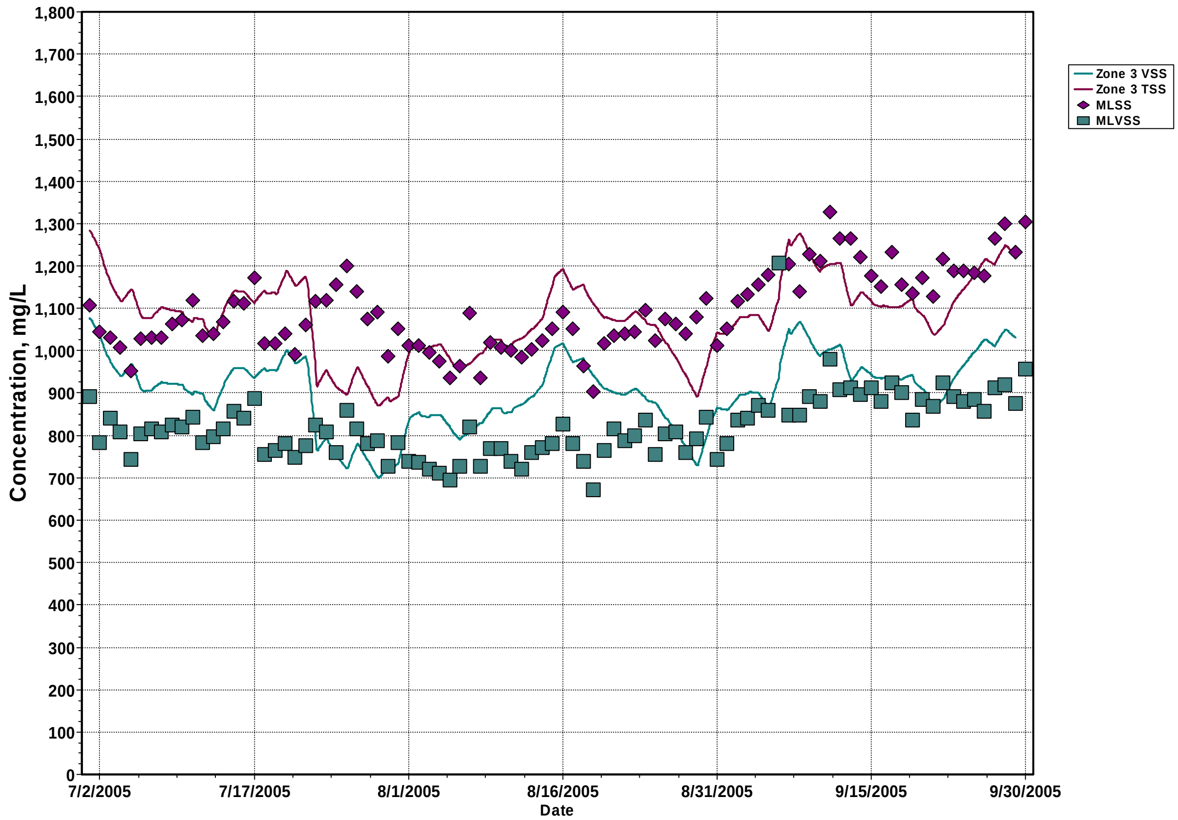


Figure 5. Model-predicted and plant-reported VSS and TSS values for mixed liquor.

PROCESS CAPACITY ASSESSMENT

Process Capacity Assessment Assumptions

The calibrated and validated model was used to determine the process capacity of the biological treatment facilities of HARRF. Hydraulic capacity is the other component that defines overall plant capacity and was presented in the *Plant Hydraulic Profile Analysis Technical Memorandum.* The process capacity assessment assumes that there is an equal distribution of primary effluent and RAS among the aeration basins so that operation and performance in each basin is similar. Figure 1 showed an imbalance in MLSS concentration among the aeration tanks that indicates an unequal flow split of RAS and/or primary effluent. This assessment assumes balanced operation of the five aeration tanks for an optimistic process capacity estimate.

Table 8 summarizes the dry weather influent wastewater concentrations used for the capacity assessment. The influent TSS and $cBOD_5$ values were based on historic data. The COD, TKN, ISS and total phosphorus values were determined from results of the wastewater characterization. The SRT value represents an average value based on historic data, using only the volume of the aeration basins to determine solids inventory (secondary clarifier volume was neglected). The process capacity of the plant represents current mode of operation.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Table 8. Assumptions for process capacity assessment.

Parameter	Value
TSS, mg/L	300
$cBOD_5$, mg/L	225
COD, mg/L	621
TKN, mg/L	45
Total Phosphorus, mg/L	5.5
ISS, mg/L	57
SRT, d	2.75

The process capacity is determined for both dry weather and wet weather conditions with the lower capacity value controlling. The dry weather process capacity is determined at average loading conditions with either one aeration tank and one primary clarifier out of service, or one larger (i.e., 110-ft diameter) secondary clarifier out of service. The limiting suspended solids concentration that the secondary clarifiers can handle is determined using the diurnal peaking factor determined from the wastewater characterization study as the maximum clarifier hydraulic loading and the 90^{th} percentile historic sludge volume index (SVI) value (i.e., 203 mL/g) to determine the limiting clarifier solids flux. The 90^{th} percentile SVI value implies that additional means of controlling SVI (e.g., RAS chlorination, polymer addition) must be used approximately 30 days per year to maintain minimum acceptable clarifier performance.

Evaluation of process capacity with primary and secondary units out of service represents a worst-case situation during dry weather when inspection and routine maintenance are performed to ensure that all process units are available during wet weather periods. Wet weather process capacity is determined using the peak week loading determined from historic data to estimate activated sludge MLSS concentration and peak daily wet weather flow as the maximum clarifier hydraulic loading, and the 90^{th} percentile SVI value to determine the limiting mixed liquor concentration. Because the peak daily wet weather flow is used instead of the peak hourly flow, the sludge blanket depth in the secondary clarifier will vary during a storm event. However, the existing secondary clarifiers are considered to be deep enough to handle variations in blanket depth over a storm event. Table 9 summarizes the peaking factors, relative to dry weather conditions, used for the capacity assessment. The peak wet weather flow peaking factor is conservative assumption because of the Rancho Bernardo flows. The Rancho Bernardo flows are expected to be 9 mgd because this is limitation of the pump station.

Table 9. Peaking factors used for process capacity assessment.

Parameter	Value
Flow	
Diurnal Peaking Factor	1.30[a]
Wet Weather Daily Peaking Factor	2.00[b]
Loading	
Peak Week $cBOD_5$ Loading Factor	1.58[b]

[a] determined from wastewater characterization
[b] determined from Wastewater Collection System Master Plan Update, November, 2005

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

The removal efficiency as a function of influent flow for the primary clarifiers was calculated based on the dry weather or wet weather surface overflow rate (SOR) using a lambda value of 2,347 gpd/ft^2 and a non-settleable solids concentration of 92.2 mg/L as described in the *Primary Clarifier Stress Testing and Capacity Assessment Technical Memorandum*. DO in the aeration basin was set at 2 mg/L for both the dry weather and wet weather evaluations to allow for complete nitrification.

Process Capacity Assessment Assuming No Aeration Limitations

Figure 6 shows the simulated aeration tank MLSS concentration as a function of ADWF with one primary clarifier out of service and one aeration basin out of service and with all aeration tanks in service. In addition, the calculated limiting suspended solids concentration for the secondary clarifiers as a function of ADWF is also shown with all secondary clarifiers in service and with one 110-ft secondary clarifier out of service. The intersection of the MLSS concentration and the limiting suspended solids concentration represents the process capacity of the plant assuming no aeration limitations. The impact of aeration system performance is discussed separately in the following section.

Figure 6 shows the HARRF dry weather process capacity to be 20.0 mgd with one aeration basin and one primary clarifier out of service and 19.4 mgd with all aeration basins and primary clarifiers in service and one 110-ft secondary clarifier out of service.

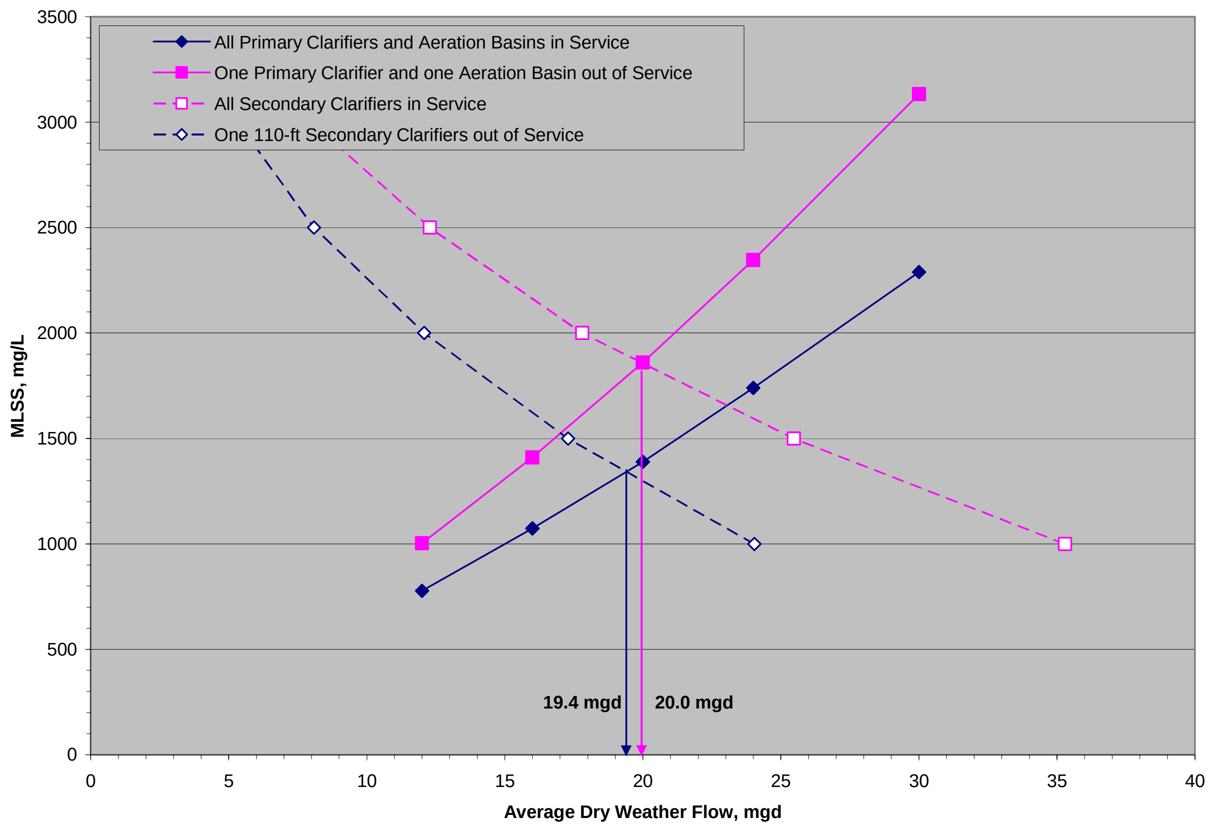


Figure 6. Process capacity assessment for dry weather conditions.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Figure 7 shows the corresponding wet weather process capacity assuming no aeration limitations. It is assumed that all units will be in service during wet weather operation. As shown on Figure 7, the wet weather process capacity of HARRF is 14.8 mgd on an ADWF basis with the existing 90th percentile SVI of 203 mL/g. However, if the sludge settleability were improved (decreased SVI), additional capacity could be realized. Figure 7 also shows that if the 90th percentile SVI value were decreased to 125 mL/g, the capacity of the plant could be increased to 19.0 mgd. A decrease in SVI can be achieved with the addition of a biological selector and operation at 2.0-d SRT. In Figure 7, the MLSS concentration as a function of influent loading for the 2.75-d SRT and 2.0-d SRT conditions are similar even though they are operated at different SRT. This is because using an anaerobic selector would displace approximately 25 percent of the total aeration basin volume. This volume used for the anaerobic selector is not included in the SRT calculation and accounts for the difference.

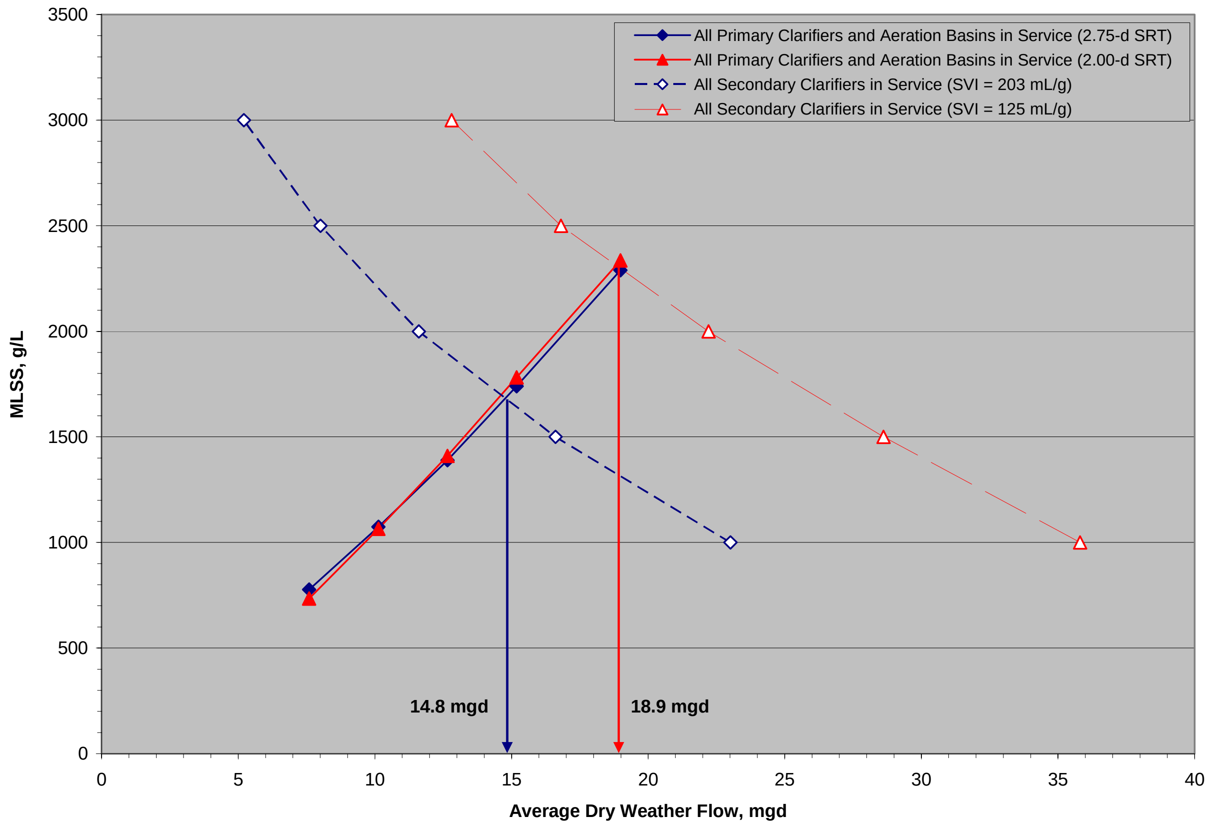


Figure 7. Process capacity assessment for wet weather conditions for existing operation (90th percentile SVI = 203 mL/g and SRT = 2.75 d) and for operation with an anaerobic selector (90th percentile SVI = 125 mL/g and SRT = 2.0 d).

Capacity Assessment of Aeration System

The results presented above for the wet weather and dry weather capacity assume that the existing aeration system can meet the oxygen requirements of the influent loading. The HARRF currently has Parkson panels in the aeration basins and three Turblex Blowers. Each blower has a capacity of 10,300 scfm.

Off-gas aeration testing was performed on April 26, 2006 to determine the oxygen transfer efficiency of the existing aeration system. Off-gas testing is performed by collecting gas from the aeration basin by floating a hood on the surface of the aeration basin. The gas collected by the hood is measured for gas content and results of the testing can be used to estimate an α_F value. An α value is the ratio of oxygen transfer in "dirty" water to clean water for a clean aeration diffuser. An α_F value accounts for the degree of fouling of the aeration diffuser. When determining the α_F value, an SOTE value is assumed based on clean water testing performed by the manufacturer. For the Parkson panels that are in service, the SOTE value was assumed to be 36.0 percent. The measured α_F value is estimated to be 0.32. Using the results of the off-gas testing, the capacity of the aeration system was estimated.

The ADWF capacity was determined from the peak hour oxygen requirements and assuming a DO of 1 mg/L (representing a temporary reduction from the average value of 2 mg/L). Peak air requirements were determined using the OUR values predicted by the calibrated BioWin™ model. The peak airflow requirement occurs during the peak COD and/or ammonia loading as described by the diurnal variations and it was assumed that the plant was nitrifying. Currently, the plant is not completely nitrifying as discussed previously. Figure 8 shows the resulting capacity of the existing aeration blower system. With all three blowers in service at peak loading conditions, the current capacity is estimated to be 15.0 mgd. The α_F value that was measured is considered relatively low and is attributed to the low SRT (2.75 d) operation condition of HARRF. A high-rate operation, or low SRT, has been shown to result in a reduced α_F value. The capacity of the aeration system can be increased by suppressing nitrification. Nitrification can be suppressed by operating at a reduced SRT. Figure 8 also shows that the estimated blower capacity is 21.6 mgd at a 2.0-d SRT and with an anaerobic selector (it is recommended that a selector be used for a 2-d SRT to control sludge settleability).

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

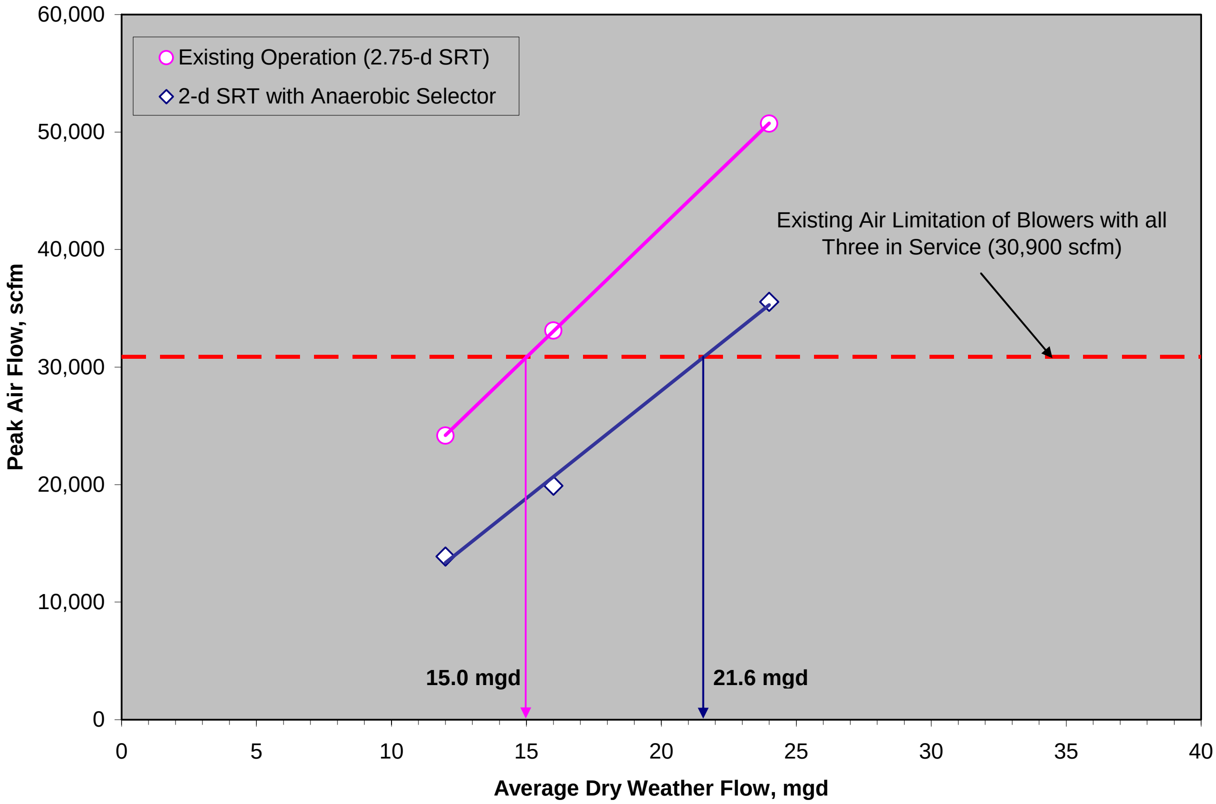


Figure 8. Aeration requirements for peak loading conditions for existing operation (2.75-d SRT) and with an anaerobic selector (2.0-d SRT).

In addition to the blower system, the aeration system also consists of the fine-bubble aeration equipment. Currently, the plant has Parkson panels. The Parkson panels are intended to operate at a maximum airflow of 1 scfm/ft^2 and as such are not able to meet the current aeration demands. It is recommended that an aeration system be installed that is capable of operating at higher air rates so that additional capacity is possible.

<u>Summary of Results</u>

Table 10 summarizes the results of the process capacity assessment for the secondary system at the current operating condition (2.75-d SRT, and 90th percentile SVI of 203 mL/g). The capacity of the plant was determined to be 14.8 mgd driven by the wet weather condition. If the 90th percentile SVI value were to be reduced to 125 mL/g which is considered a 90th percentile value for an activated sludge system using an anaerobic selector, the plant capacity could be increased to 18.9 mgd. However the existing aeration system is estimated to only be sufficient for 15.0 mgd. As mentioned above, these values represent the ADWF corresponding to the wet weather flow, and not the actual wet weather flows.

Table 10. Results of process capacity assessment under average conditions for 2000 to 2005 for HARRF (90th percentile SVI = 203 mL/g, SRT = 2.75 d).

	Capacity Assuming Sufficient Aeration	Capacity with Existing Aeration System*
Dry Weather Operation		
One 110-ft secondary clarifier out of service	19.4 mgd	---
One Aeration Basin and One Primary Clarifier Out of Service	20.0 mgd	---
Wet Weather Operation		
All Units in Service	14.8 mgd	15.0 mgd

* Assuming new fine-bubble diffusers are installed capable of operating at higher air flux than existing system

DISCUSSION OF RESULTS

The current rating of HARRF is 18 mgd as a result of a letter report produced by MWH titled, "Final Letter Report for Capacity Rerating of the Hale Avenue Resource Recovery Facility (HARRF)" (Letter Report) submitted in 2004. Previously, the plant was rated at 16.5 mgd after the 1981 project to increase capacity. The capacity analysis performed by MWH was a desktop study using published design guidelines. The MWH capacity study found that the aeration basins and secondary clarifiers were the limiting processes at HARRF and were both rated for an average flow of 18 mgd (assuming a peak wet weather factor of 2.0). For the process capacity study that was presented in this technical memorandum, the secondary system (including the aeration basins and secondary clarifiers) were also determined to limit the capacity of HARRF. It should be noted that the existing solids processing equipment is not sufficient to meet the 18 mgd flow (see *Solids Handling Processes Evaluation Technical Memorandum*) in contrast to the 2004 Letter Report.

The differences in the capacity conclusions between the Letter Report and this analysis lie in the assumptions. For this analysis, the limiting condition (assuming sufficient aeration capacity) was the wet weather condition which assumes peak week loading at peak flow with all units in service. The capacity was determined in two steps. The first step was to determine the MLSS concentration using the calibrated BioWin™ model. At 18 mgd, the MLSS concentration was estimated to be 2,150 mg/L at peak week loading and 1,220 mg/L at average loading with all aeration basins in service. The Letter Report assumed a MLSS concentration of 2,500 mg/L which is higher than both values determined from the model. For the secondary clarifier analysis, the Letter Report assumed a peak solids loading rate (SLR) or 2.0 lb/sf-hr, which was referenced from Metcalf & Eddy, "Wastewater Engineering Treatment, Disposal and Reuse", Third Edition, 1991. This value has since been reduced to 1.6 lb/sf-hr in the recent Fourth Edition of the textbook which will reduce the secondary clarifier capacity. For either of these SLR design values, the SVI of the sludge is not considered. A sludge with a higher SVI will reduce the potential SLR and reduce capacity, while a sludge with a lower SVI will increase the SLR and increase the capacity. For this analysis, the historic SVI data was analyzed to determine settling characteristics of the sludge which were applied to state point analysis to determine secondary clarifier capacity. The 90th percentile SVI value was determined to be 203 mL/g, which when used with state point analysis, predicts a critical SLR of 1.25 lb/sf-hr at MLSS concentration of 2,500 mg/L. This is significantly lower than the value assumed in the Letter Report and accounts for the difference in process capacity between the Letter Report and this analysis.

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Flows higher than the capacity rating of 14.8 mgd have been observed at HARRF in the past without permit violation. The recent storm events in January 2005 resulted in flows of 30.9 mgd (corresponding to an ADWF capacity of 15.5 mgd assuming a peaking factor of 2.0). This observation is in disagreement with the results of the process capacity assessment, and is attributed to differences in sludge settleability. Figure 9 shows the historic SVI values from 2000 to 2005. The values in 2005 were significantly lower in 2005 than for previous years. The 90^{th} percentile SVI value for 2005 was determined to be 135 mL/g which equates to a critical SLR of 1.71 lb/sf-hr (MLSS=2,500 mg/L). This value is much lower than the value from the 2000 to 2005 data.

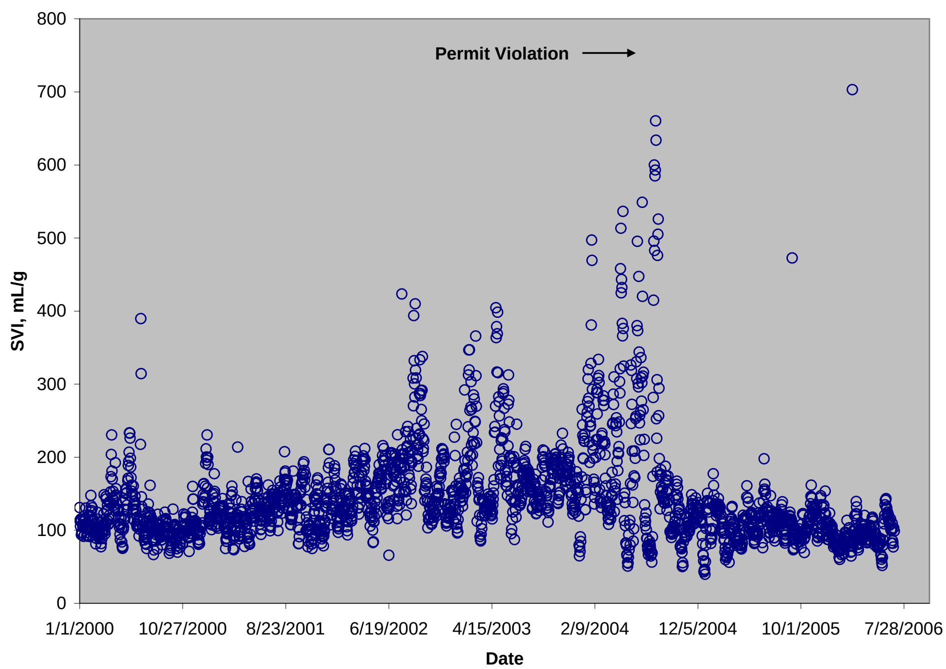


Figure 9. Historic SVI values for HARRF (the time period corresponding to the permit violation due to toxic shock is highlighted in red).

Performing the process capacity analysis based on the 2005 condition, the plant is capable of treating 18.4 mgd as shown in Figure 10. However, it should be noted that the plant is still limited by the capacity of the existing fine-bubble diffusers, which will require replacement to improve oxygen transfer.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

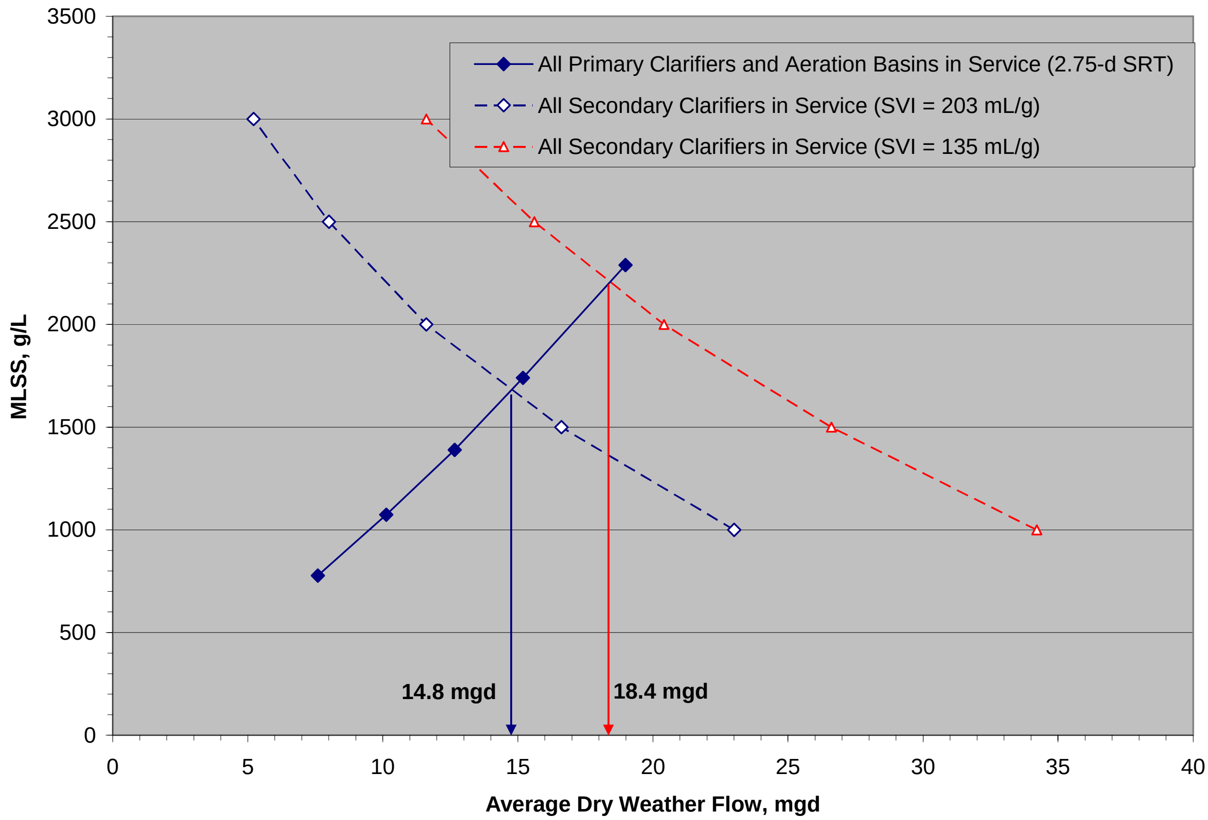


Figure 9. Process capacity assessment for wet weather conditions assuming 90th percentile SVI (203 mL/g) for 2000 to 2005 and 90th percentile (SVI = 135 mL/g) for 2005.

CONCLUSION

This process capacity assessment represents HARRF secondary process capacity under current operations and does not represent the absolute capacity of the plant. Additional capacity may be possible with some modifications to eliminate identified "bottlenecks". For example, additional capacity is possible through the following actions.

- Chemically enhanced primary treatment (CEPT) can be used to improve suspended solids removal and reduce secondary treatment system organic load.
- Addition of an effective biological selector[†] will improve sludge settleability and will increase the secondary clarifier limiting suspended solids concentration.
- Flow equalization can be used to reduce peak wet weather flows and to reduce secondary clarifier hydraulic loading.

† Biological selectors work by selecting against filamentous microorganisms by creating an environment that is not conducive to their growth, therefore lowering SVI values. For HARRF, either an anoxic or anaerobic selector is recommended depending on the operating SRT; anoxic selector for a nitrifying system (higher SRT) and an anaerobic selector for a non-nitrifying condition (lower SRT). The selector is implemented by converting the beginning of each aeration basin to a non-aerated section. The selector process can be optimized by including baffling to improve plug-flow conditions.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

FUTURE IMPROVEMENTS

In order to improve the plant performance and restore process capacity to 18.0 mgd, the following improvements have been identified :

1. Construct one additional primary clarifier
2. Construct one additional aeration basin
3. Simultaneously, replace the fine-bubble aeration system and install a selector zone.
4. Modify air control system to improve process control.
5. Modify the RAS flow distribution so that there is equal split to each aeration basin and calibrate the RAS flow meters.
6. Reduce operating SRT to 2 days and install new WAS pumps and pumping station to enable mixed liquor wasting.

By constructing an additional aeration basin equipped with a selector zone and new aeration equipment will allow the plant to stay in operation without reducing capacity. Once the new aeration basin is constructed and put into operation, the existing basins can be taken out of service one at time for modifications. The installation of a selector zone would involve installing submerged mixers and some baffles. The addition of an anaerobic selector would improve the sludge settleability, and a 90^{th} percentile SVI value of 125 mL/g would be expected. The future fine-bubble aerators will be capable of higher air flow rates than the existing equipment. Currently, the plant is limited by the airflow that is possible through the existing aerators; the blowers are adequately sized. In addition to replacing the aerators, it is also recommended that the air control system be modified to improve process control and performance. Because the plant will be operated at a reduced SRT (2 days), it is recommended that mixed liquor wasting be installed to simplify plant operations and provide better process control. The existing settled sludge wasting system located in the RAS/WAS pump station will be retained.

The final recommendation is to improve the existing RAS flow distribution. Improving the RAS distribution will equally distribute solids between aeration basins and provide a more balanced operation. In addition, the RAS flow meters should be recalibrated for improved process control.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

APPENDIX E

SECONDARY CLARIFIER EVALUATION TM

TECHNICAL MEMORANDUM - FINAL

DATE: JULY 13, 2006

TO: ANGELA MORROW, CITY OF ESCONDIDO

FROM: VICTOR OCCIANO, BROWN AND CALDWELL

PREPARED BY: RION MERLO, BROWN AND CALDWELL
ERIC WAHLBERG, BROWN AND CALDWELL
JOSE JIMENEZ, BROWN AND CALDWELL
SEVAL SEN, BROWN AND CALDWELL

SUBJECT: CITY OF ESCONDIDO
HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – SECONDARY CLARIFIER SETTLING TESTING AND CAPACITY ASSESSMENT

SUMMARY

The City of Escondido has engaged Brown and Caldwell to determine the capacity of the Hale Avenue Resource Recovery facility (HARRF). One of the tasks is to evaluate the capacity of the secondary clarifiers. The purpose of this memorandum is to: (1) present the results of data analysis of historical secondary clarifier data at the HARRF, (2) present results of the on-site settling tests used to determine settling characteristics, and (3) determine the capacity of the secondary clarifiers based on the mixed liquor suspended solids (MLSS) concentration in the aeration basins.

Six years of historic data (2000 to 2005) were analyzed to determine secondary clarifier performance and sludge settling characteristics. With the exception of an apparent process upset from April to August, 2004, most of the effluent suspended solids (ESS) concentrations were less than 30 mg/L. The ESS values were observed to have a cyclic pattern where higher levels were observed during the winter months. Historic sludge volume index (SVI) data were analyzed to assess sludge settleability. The 90th percentile SVI value was determined to be 203 mL/g. On-site settling tests were then performed to further characterize sludge settleability so that secondary clarifier capacity could be determined at the 90th percentile SVI condition. Using the results of the sludge settling tests, a state point analysis (SPA) was performed to determine secondary clarifier capacity over a range of MLSS concentrations. Two scenarios were analyzed: (1) all clarifiers in service and (2) one

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

110-ft clarifier out of service. The resulting secondary clarifier capacity curve was used with the results from the BioWin modeling to determine ultimate capacity.

INTRODUCTION

The HARRF is rated to treat an average daily flow of 18.0 million gallons per day (mgd). Table 1 summarizes the design values of the existing secondary clarifiers. Secondary clarifiers 1 and 2 have a diameter of 80 ft and a side water depth of 15 ft. Two additional secondary clarifiers (3 and 4) were added with the 2002 expansion. The newer secondary clarifiers have a 110-ft diameter and a side water depth of 15 ft. With the addition of the 110-foot diameter clarifiers, the three, original square clarifiers were removed from service. In 2005, the square clarifiers were converted to a chlorine contact chamber. Each of the four circular clarifiers has a dedicated return activated sludge (RAS) pump. Clarifiers 1 and 2 have a common standby pump as do clarifiers 3 and 4.

Table 1. Existing secondary clarifiers at HARRF.

Process Unit	Design Criteria
Clarifiers 1 & 2	
Diameter	80 ft
Surface Area/tank	5,000 sq ft/clarifier
Sidewater Depth	15 ft
RAS Pumps	1/clarifier (2 total)
RAS Pump Maximum Capacity	4.32 mgd/pump
Clarifiers 3 & 4	
Diameter	110 ft
Surface Area	9,500 sq ft/clarifier
Sidewater Depth	15 ft
RAS Pumps	1/clarifier (2 total)
RAS Pump Maximum Capacity	8.64 mgd/pump

The capacity of secondary clarifiers is a function of the four factors: (1) available surface area, (2) RAS pumping capacity, (3) MLSS concentration, and (4) sludge settleability. Hindered settling velocity has been shown to be related to MLSS concentrations as shown in Equation 1. Hindered settling refers to the condition where all solids settle at the same velocity, forming a distinct interface and a sludge "blanket":

$$V_s = V_o e^{-k*MLSS} \quad (1)$$

where,

V_s = hindered settling velocity, m/h
V_o = empirical settling constant, m/h
k = empirical settling constant, L/mg
MLSS = mixed liquor suspended solids concentration, mg/L

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

The constants, V_o and k, can be determined from settling tests where the hindered settling velocities are measured for a range of MLSS concentrations. Performing the necessary settling tests to determine settling velocity on a routine basis can be tedious. As a result, the SVI is typically used to measure sludge settleability. SVI is defined as the volume occupied by 1 g of MLSS after 30 minutes of settling using a graduated cylinder or settleometer. Wahlberg et al. (1995) showed that the SVI can be related to the V_o and k as described by Equations 2 and 3.

$$V_o=\gamma e^{-\delta * SVI} \tag{2}$$

$$k=\alpha+\beta * SVI \tag{3}$$

where,

α	=	empirical constant, L/mg
β	=	empirical constant, -
γ	=	empirical constant, m/h
δ	=	empirical constant, g/mL
SVI	=	sludge volume index, mL/g

By determining the sludge settling characteristics, the capacity of a secondary clarifier at steady-state operation can be determined by performing a SPA. The SPA is a graphical method for determining clarifier capacity by plotting sludge settling characteristics, secondary clarifier loading, and sludge removal due to the underflow, or RAS. Figure 1 shows a typical SPA showing: (a) an underloaded clarifier, (b) an overloaded clarifier, and (c) clarifier failure due to clarification failure.

To determine the capacity of the HARRF secondary clarifiers, the 90^{th} percentile SVI value, determined from historic data (2000 to 2005), was used to represent design settling conditions. In addition, settling tests were performed on-site to determine V_o and k values, which were compared against values predicted by the relationship developed by Wahlberg et al. (1995). The α, β, γ and δ values given by Wahlberg et al. (1995) were used to determine V_o and k values that are representative of the 90^{th} percentile SVI. Using these values, a settling curve was generated to perform a SPA to determine secondary clarifier capacity.

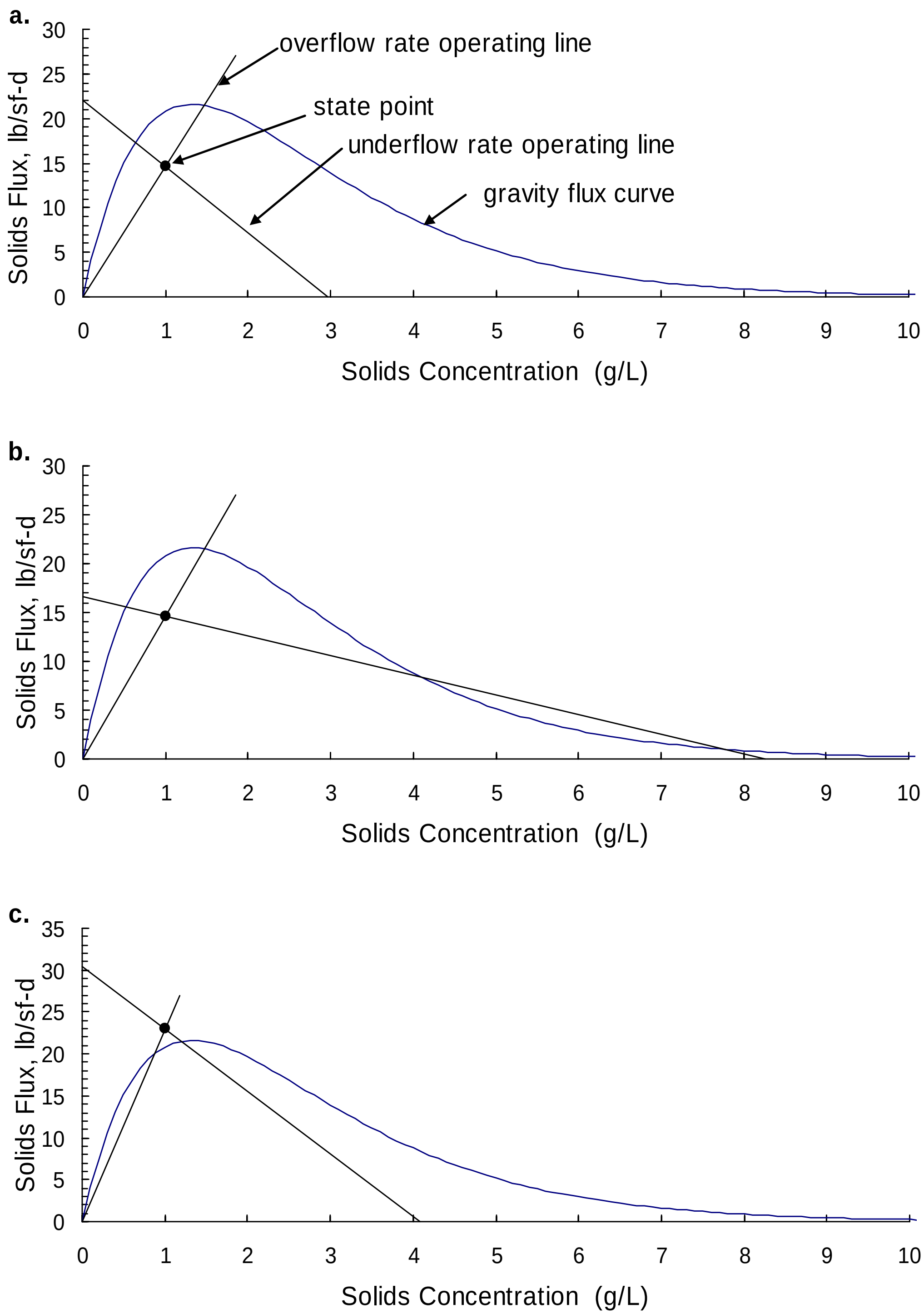


Figure 1. State point analysis (SPA) for (a) an underloaded clarifier, (b) an overloaded clarifier, and (c) clarifier failure due to clarification failure.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

RESULTS AND DISCUSSION

Historic Data

The historic ESS values from HARRF's secondary clarifiers are shown in Figure 2. In general, most of the values are less than 30 mg/L with the exception of a known process upset starting at the end of April 2004 and lasting through to the beginning of August 2004. Also shown is the influent wastewater temperature. The ESS concentration appears to have a cyclic pattern where higher levels are observed during the winter months.

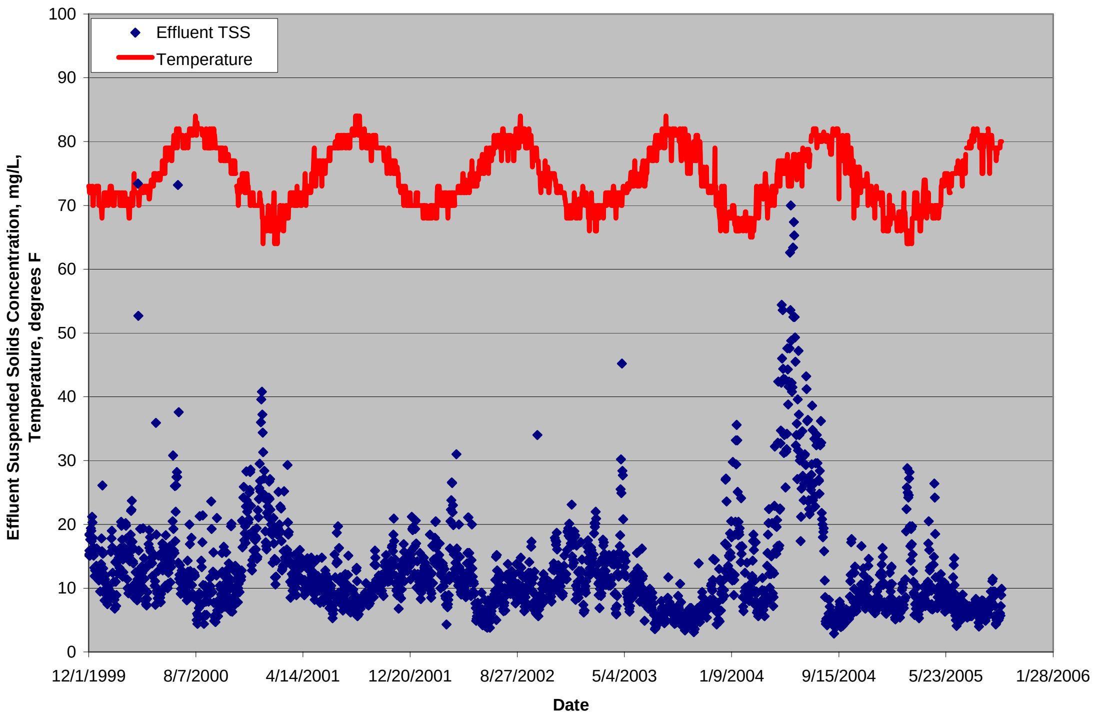


Figure 2. Historic ESS and influent wastewater temperature.

SVI values are determined by plant staff using a 2-L settleometer. The historic values are shown in Figure 3. At the time the SPA was performed, data for 2000 to 2005 were available and were used for analysis. Data for January 2006 to June 2006 were added after the analysis was performed and are shown in Figure 3. The plant was in violation from May 3, 2004, to August 17, 2004, due to a toxic substance in the influent wastewater. The 90th percentile SVI value (2000 to 2005) was determined to be 203 mL/g (SVI values occurring at the time of permit violation were not included in the analysis). The elevated ESS values that occurred in April 2004 through the beginning of August 2004 correspond to elevated SVI values. However, there is not a clear trend in the historic SVI data related with temperature as was the case with the ESS concentrations.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

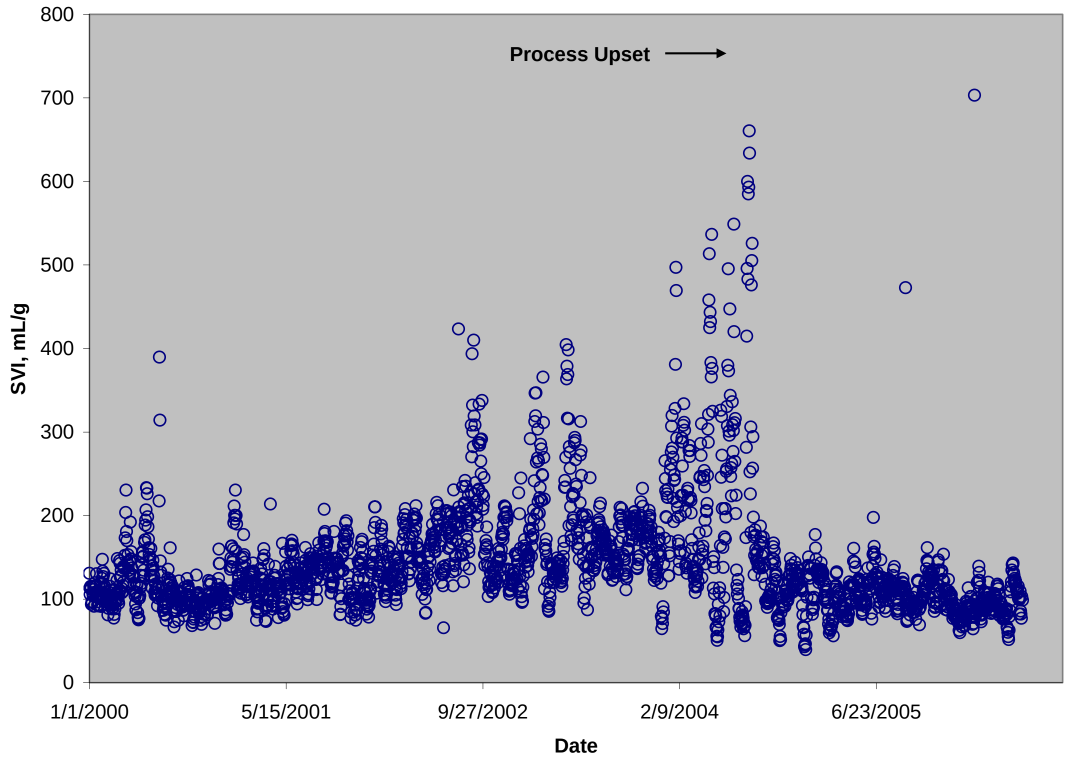


Figure 3. Historic SVI values for HARRF (the time period corresponding to the process upset due to toxic shock is highlighted in red).

<u>On-Site Settling Tests</u>

Settling tests were performed on December 6, 2005. The field-determined and model-predicted settling velocities based on Equation 1 are shown on Figure 4. The V_o and k values were determined to be 12.1 m/h and 0.0006 L/mg, respectively. The SVI measured on the test day was 134 mL/g.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

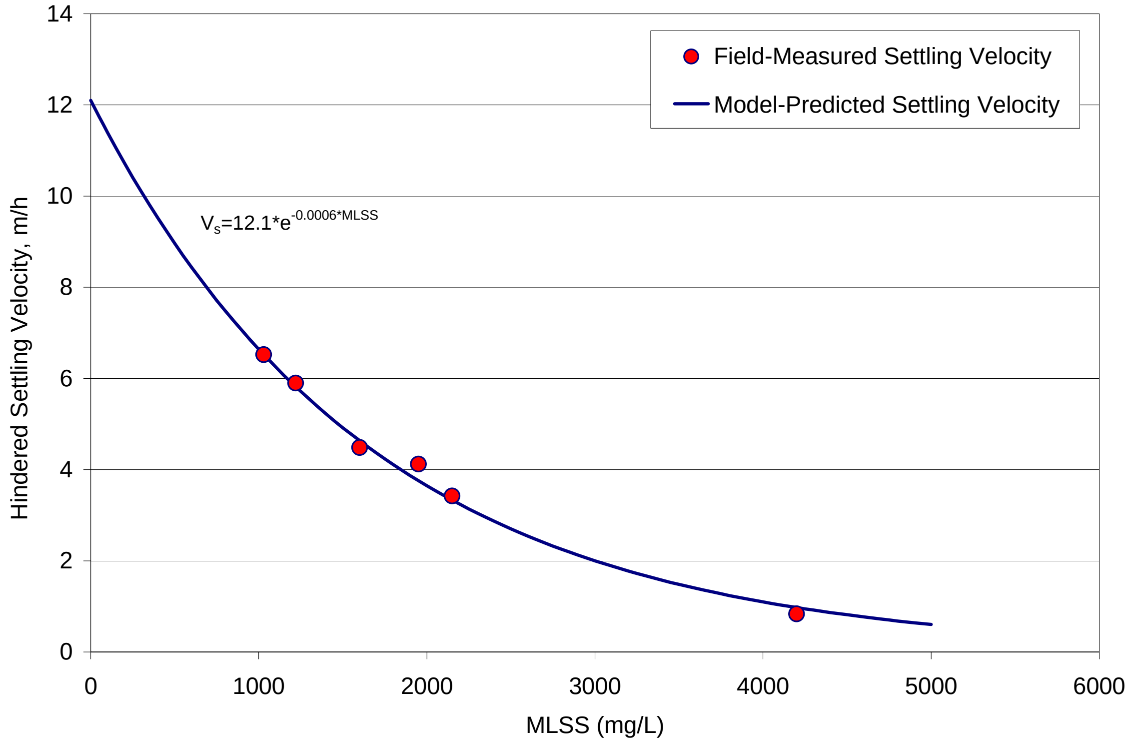


Figure 4. Results of on-site settling tests.

<u>Capacity Assessment</u>

The values determined from the stress testing were compared against values given in Wahlberg et al. (1995). Figure 5 shows that the field-measured values were in good agreement. The α, β, γ and δ values from Wahlberg et al. (1995) were then used to determine V_o and k values for the 90th percentile SVI (203 mL/g). Values were determined to be 164 m/d and 0.610 L/g for V_o and k, respectively and were used to construct a representative settling curve for the SPA.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

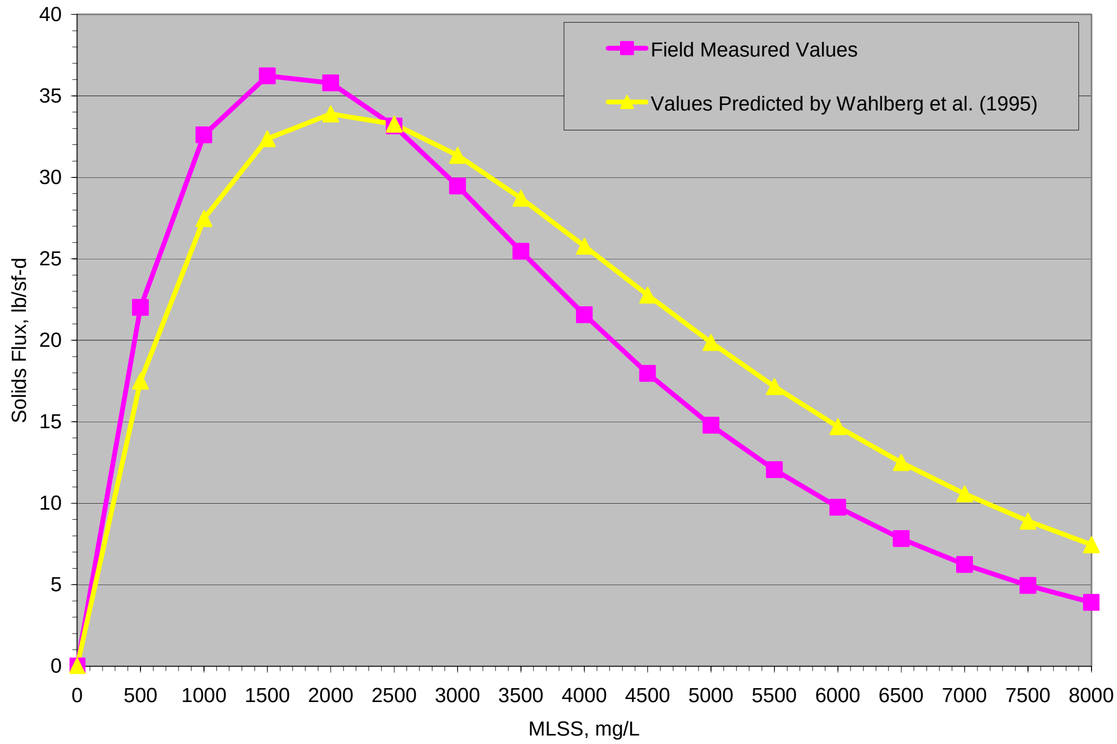


Figure 5. Comparison of Field-Measured V and k values with

For the SPA, two scenarios were analyzed: (1) all clarifiers in service and (2) one 110-ft clarifier out of service. The RAS rate for the two scenarios was set at 25.9 and 17.3 mgd, respectively, which is the maximum RAS capacity available with all the clarifiers on line and with one of the larger clarifiers off line, respectively. The maximum solids loading rate determined by SPA for each scenario was derated by 20 percent to account for differences in flow distribution and non-idealities in the secondary clarifier performance.

The capacity of the secondary clarifiers was determined using the SPA over a range of MLSS concentrations. As an example of this analysis, Figure 6 is given. For a MLSS concentration of 2,000 mg/L, all clarifiers in service, and a RAS flow of 25.9 mgd, the influent flow (i.e., the overflow rate operating line) was increased until the clarifier was just overloaded. The resulting solids loading rate was then decreased by 20 percent and the influent flow corresponding to this lower solids loading rate was calculated.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

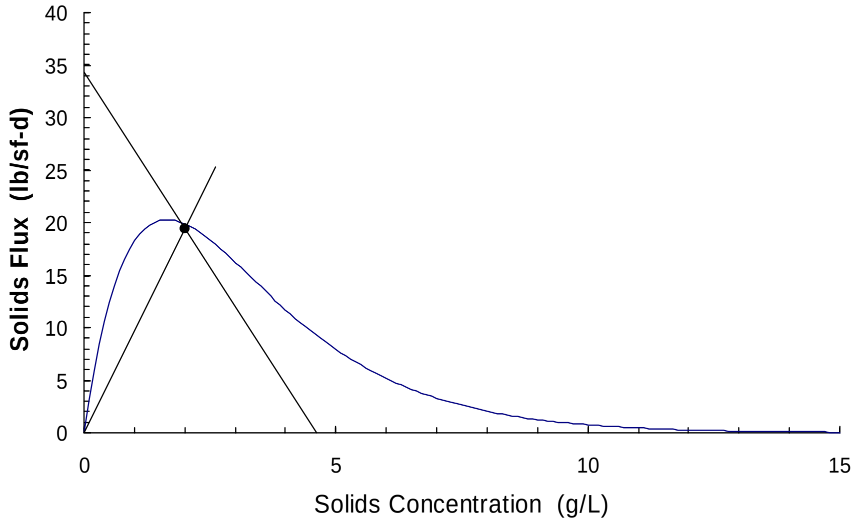


Figure 6. Results of SPA with all secondary clarifiers in service (MLSS=2,000 mg/L and RAS=25.9 mgd)

Results summarizing the two scenarios are given in Figure 7. As illustrated in the figure, a higher MLSS concentration lowers the capacity of the clarifiers because of elevated solids loading.

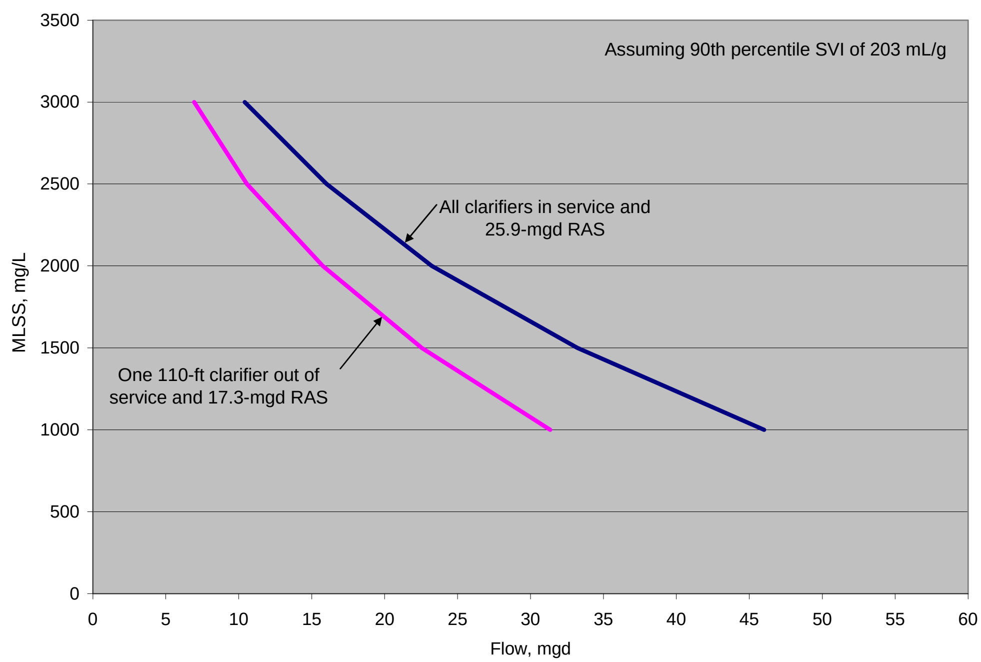


Figure 7. HARRF secondary clarifier capacity assuming a 90th percentile SVI value of 203 mL/g (2000-2005).

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

CONCLUSIONS

For the secondary clarifier capacity analysis, historic SVI data (2000 to 2005) were evaluated to determine a 90th percentile representing sludge settleability. On-site settling tests were performed to characterize the sludge settleability, and results were used to estimate settling characteristics for the 90th percentile condition. A SPA was used to produce a secondary clarifier capacity curve relating capacity to MLSS concentration for two conditions: one with all clarifiers in service, the other with one of the large clarifiers out of service. The capacity of the secondary clarifiers will ultimately depend on the MLSS concentration of the aeration basins. The final capacity of the secondary clarifiers was determined using the MLSS concentration determined from the BioWin analysis of the secondary system.

REFERENCES

Wahlberg, E.J. and Keinath, T.M. (1995) Development of settling flux curves using SVI: An addendum, Water Env. Res. **67** 872-874.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

APPENDIX F

SOLIDS MASS BALANCE TM

TECHNICAL MEMORANDUM –FINAL

DATE: JULY 5, 2006

TO: ANGELA MORROW, CITY OF ESCONDIDO

FROM: VICTOR OCCIANO, BROWN AND CALDWELL

PREPARED BY: SEVAL SEN, BROWN AND CALDWELL
RON APPLETON, BROWN AND CALDWELL

SUBJECT: CITY OF ESCONDIDO
HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – MASS BALANCE EVALUATION

SUMMARY

A mass balance model, MABLE, was used to validate the accuracy of performance data from the HARRF by tracking 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), and volatile suspended solids (VSS) across the liquid treatment and solids handling processes. The good agreement between reported and modeled mass balance demonstrates the accuracy of the historical performance data, so that they may be used for other analyses as part of the treatment plant capacity study.

INTRODUCTION

Historical performance data from the HARRF were used to prepare a mass balance model of the plant. A spreadsheet-based program that was developed by Brown and Caldwell, MABLE, was used for this analysis. The goal of this evaluation was to validate the accuracy of the historical performance data by tracking BOD_5, TSS and VSS through primary treatment, secondary treatment, sludge thickening, anaerobic digestion, and sludge dewatering.

Performance data from July and August 2005 were used as input to the model. This period was considered representative of average plant operations and performance based on discussion with plant staff. A copy of the model output is attached. Model input is shown as shaded cells with pink text.

Average primary influent flow and BOD and TSS concentrations were used as model input. Average primary influent VSS concentration was estimated based on primary sludge volatile

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

solids (VS) concentration. The first sample point in the HARRF liquid treatment train is the primary influent, which includes Escondido and Rancho Bernardo raw sewage, solids handling returns, and tertiary filter backwash. Separate raw sewage data were not available.

Average primary effluent BOD and TSS concentrations were specified to describe primary clarifier performance. Average primary effluent VSS concentration was estimated based on primary sludge VS concentration. Average primary sludge concentration was specified also.

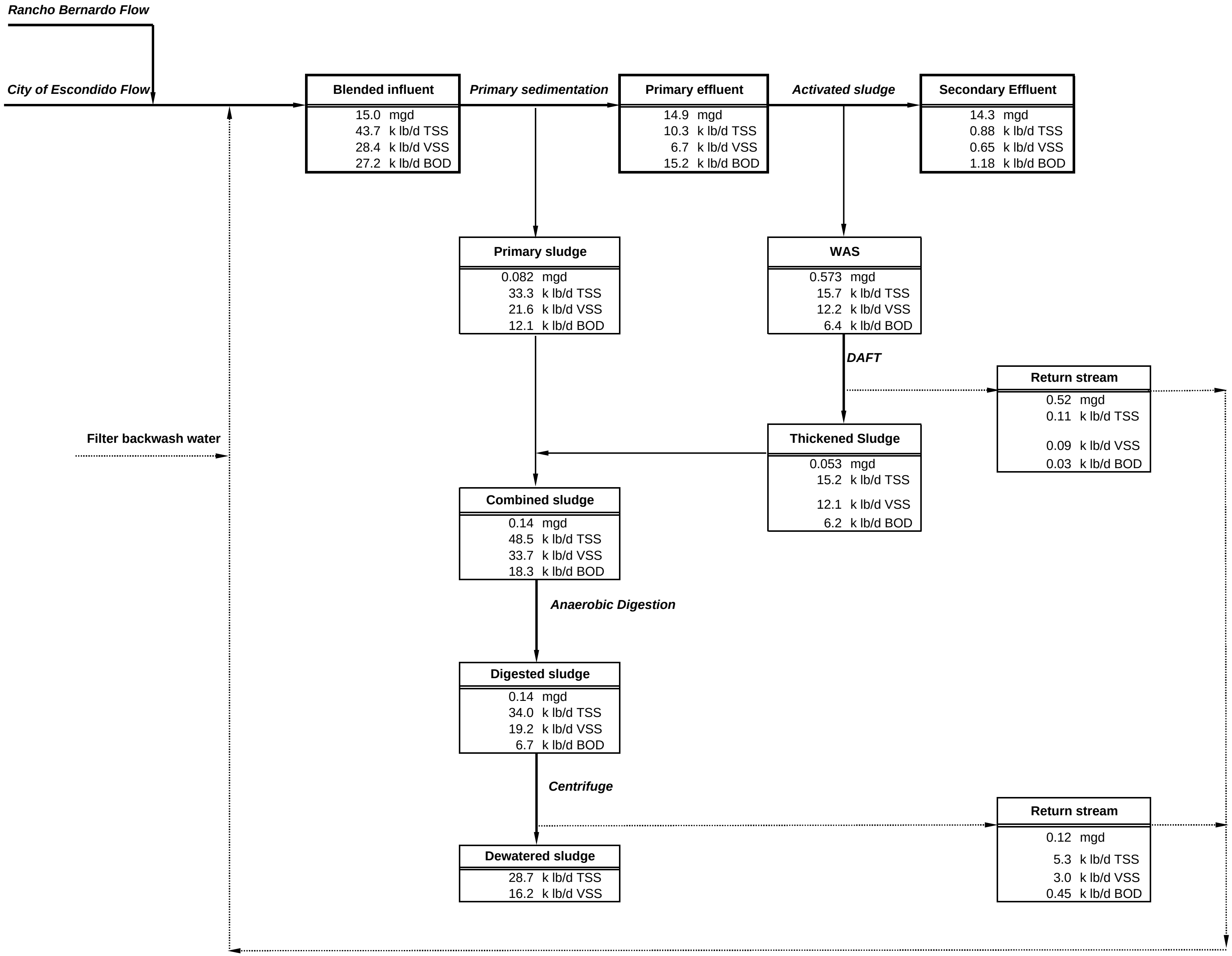
Average secondary effluent BOD, TSS, and VSS concentrations were specified to describe secondary treatment performance. Average waste activated sludge (WAS) flow and sludge production were specified also.

Average thickened sludge flow, TS concentration, and solids capture efficiency were specified to describe dissolved air flotation thickening (DAFT) treatment performance. Average volatile solids destruction was specified to describe anaerobic digestion treatment performance. Average dewatered sludge TS concentration and solids capture efficiency were specified to describe centrifuge performance.

Raw sewage flow and BOD, TSS, and VSS concentrations are not measured directly at the HARRF, so these values were calculated from the specified primary influent characteristics using MABLE. Calculated solids handling return streams based on specified thickening and dewatering treatment performance were subtracted from the specified primary influent loadings. In addition, the specified filter backwash stream was subtracted from the specified primary influent loadings. The filter backwash stream flow was assumed to be 15 percent of the filter influent flow of 4.0 mgd, or 0.60 mgd. Filter backwash stream BOD, TSS, and VSS loads were assumed based on complete removal of secondary effluent BOD, TSS, and VSS concentrations. The filter backwash stream TSS load also included estimated chemical sludge from filter influent coagulant addition.

Figure 1 summarizes the HARRF mass balance calculations using average performance data from July and August 2005. The figure shows that the specified input and performance conditions result in a difference between influent solids and effluent solids (secondary effluent and dewatered sludge) of only 2.4 percent. The reported average dewatered sludge production for this period was 28.7 k lb/d, which compares well with the MABLE-calculated value of 30.5 k lb/d. In addition, the MABLE-calculated raw sewage BOD, TSS, and VSS concentrations are typical of municipal wastewater.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833



Solids Figure.xls Figure1

Figure 1. Simulated Solids Production at Existing Loadings

HARRF-Escondido
STREAM SUMMARY

Mass Balance Data Period: July-August 2005

Stream	Flow	BOD		TSS		VSS	
	mgd	k-lb/day	mg/L	k-lb/day	mg/L	k-lb/day	mg/L
Sewer Influent	*13.76*	26.32	*229*	30.49	*266*	24.79	*216*
Septage	*0.00000*	0.00	*#DIV/0!*	0.00	*#DIV/0!*	0.00	*#DIV/0!*
Other Sludge	*0.60000*	0.33	*66*	7.37	*1473*	0.18	*36*
Primary Influent	15.00	27.24	218	43.66	349	28.36	227
Primary Sludge	0.082	12.06	17553	33.33	48510	21.64	31495
Primary Scum	*0.00000*	0.00	*#DIV/0!*	0.00	#DIV/0!	0.00	#DIV/0!
Secondary Influent	14.92	15.18	122	10.33	83	6.72	54
Secondary Effluent	14.34	1.18	9.90	0.77	6.40	0.65	5.40
Waste Activated Sludge	*0.573*	6.35	1328	*15.65*	3275	12.16	2545
Secondary Scum	*0.00000*	0.00	#DIV/0!	0.000	#DIV/0!	0.000	#DIV/0!
Flotation Thickener Influent	0.57	6.35	1328	15.65	3275	12.16	2545
Flotation Thickener Overflow	0.52	0.03	8	0.11	25	0.09	20
Flotation Thickener Underflow	0.00	0.11	13871	0.39	51000	0.30	39632
Flotation Thickened Sludge	0.053	6.21	14048	15.15	34272	12.08	27321
Digester Influent	0.14	18.27	16181	48.48	42936	33.72	29861
Digester Effluent	0.14	6.73	5957	33.98	30096	19.22	17021
Sludge Storage Effluent	0.14	6.73	5957	33.98	30096	19.22	17021
Dewatering Centrate	0.12	0.45	432	5.30	5093	3.00	2880
Dewatered Solids	0.01	6.28	74682	28.68	341250	16.22	192992

Overall Mass Balance Check:

Flow Balance (mgd) = 5.04E-04
Inert Solids Balance (k-lb/d) = 3.04E-01

Note: Input data (given and assumed) are in italics. The TSS concentration values for primary sludge, thickened sludge and dewatered solids are calculated from given (or assumed) % solids and assumed specific gravity values.

MABLE 1.0							
FACILITY NAME:	*HARRF-Escondido*						
DATA PERIOD:	*July-August 2005*						
TOTAL PLANT INFLUENT							
Sewer Flow, mgd	13.76	Septage Flow, mgd	*0*	Other Sludge Flow, mgd	*0.60*	A	
VSS/TSS ratio	0.813	VSS/TSS ratio	#DIV/0!	VSS/TSS ratio	0.02		
Concentrations, mg/L		Concentrations, mg/L		Concentrations, mg/L			
BOD	229	BOD	*0*	BOD	*66.0*	A	
TSS	266	TSS	*0*	TSS	*1473.0*	A	
VSS	216	VSS	*0*	VSS	*36.0*	A	
Mass loadings, k-lb/d		Mass loadings, k-lb/d		Mass loadings, k-lb/d			
BOD	26.32	BOD	0.00	BOD	0.33		
TSS	30.49	TSS	0.00	TSS	7.37		
VSS	24.79	VSS	0.00	VSS	0.18		
Assumptions:	0.60 mgd filter backwash flow (15% of 4 mgd filter influent); Complete removal of SE BOD, TSS, VSS; backwash TSS load adjusted so that RS VSS:TSS = 0.813,						
	typical municipal wastewater; verify polyaluminum chloride Al content and dose						
INFLUENT PRELIMINARY TREATMENT							
☑ Recycle flows entering process unit, incl.:		*DAFT subnatant, centrate*					
Influent Flow, mgd	15.00	Effluent Flow, mgd	15.00	Screenings and Grit Flow, mgd	*0*		
Mass loadings, k-lb/d		Mass loadings, k-lb/d		Mass loadings, k-lb/d			
BOD	27.24	BOD	27.24	BOD	*0*		
TSS	43.66	TSS	43.66	TSS	*0*		
VSS	28.36	VSS	28.36	VSS	*0*		
Screenings production rate, cu ft/Mgal	*0*						
Grit production rate, cu ft/Mgal	*0*						
Screenings production, cu ft/d	0.00						
Grit production, cu ft/d	0.00						
Note: Grits and screenings are, by default, assumed not to be included in the TSS measurement of the influent raw wastewater, so that the TSS mass flow remains the same.							
However, in the event that the mass balance does not converge, caution should be taken that no solids and BOD loss occurs due to screenings and grit removal. If that							
should be the case, a value for screenings and grit production should be specified.							
If screenings and grit removal does not affect flow and BOD, TSS and VSS loading, the values for screenings and grit should be set at zero.							
Assumptions:							

PRIMARY SEDIMENTATION

☐ Recycle flows entering process unit, incl.:

Influent Flow, mgd	*15.00*		Effluent Flow, mgd	14.92		Sludge Flow, mgd	0.082		Scum Flow, mgd	*0*
Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass loadings, k-lb/d	
BOD	*27.24*		BOD	15.18		BOD	12.06		BOD	0
TSS	*43.66*		TSS	10.33		TSS	33.33		TS	*0.000*
VSS	*28.36*	A	VSS	6.72		VSS	21.64		VS	0.000
Solids removal eff. (infl-effl)/infl, %	76.35		Concentrations, mg/L							
Primary sludge solids, %	*4.62*		BOD	*122*					Scum Spray Water Flow, mgd	0.000
Specific gravity of PS	*1.05*	A	TSS	*83*					Mass loadings, k-lb/d	
Volatile content of PS, VSS/TSS	0.65		VSS	*54*	A				BOD	0.0000
BOD removal eff. (infl-effl)/infl, %	44.28								TSS	0.0000
Primary scum solids, %	#DIV/0!								VSS	0.0000
Specific gravity of primary scum	*0.95*								Concentrations, mg/L	
Volatile content of scum, VS/TS	*0.90*								BOD	*9.9*
BOD content of scum, BOD/VS	*0*								TSS	*6.4*
Scum spray water flow/nozzle, gpm	*0*								VSS	*5.4*
Total tankage surface area, sq ft	*21000*									
Spray area per nozzle, sq ft	*58*									
Total number of nozzles	362									

Note: Primary scum flow and solids loading are specified by the user. Primary scum flow and solids loading are added to the mass balance at primary sedimentation.

Specify scum spray water flow/nozzle to be zero if spray water is not used. Solids and BOD % removals include contribution from scum spray water.

Scum spray can be specified as part of process water recycled from secondary effluent (see Recycle Streams under MassBal menu).

If no data are available for scum and spray water, enter zero for scum flow and TSS loading and for scum spary water flow per nozzle.

Assumptions: Primary influent and effluent VSS based on primary sludge VSS:TSS ratio

SECONDARY TREATMENT

☐ Recycle flows entering process unit, incl.:

Influent Flow, mgd	14.92		Effluent Flow, mgd	14.34		Sludge Flow (WAS), mgd	0.573		Scum Flow, mgd	0	A
Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass loadings, k-lb/d		
BOD	15.18		BOD	1.184		BOD	6.35		BOD	0	
TSS	10.33		TSS	0.766		TSS	15.65		TSS	0.000	
VSS	6.72		VSS	0.646		VSS	12.16		VSS	0.000	
Net Biomass yield, VSS/BOD_{in}	0.84		Concentrations, mg/L			Sludge mass that contains PolyP, k-lb/d					
WAS BOD/VSS ratio	0.522	BW	BOD	9.9		ISS	0		Scum Spray Water Flow, mgd	0.000	
WAS VSS/TSS ratio	0.78		TSS	6.4					Mass loadings, k-lb/d		
WAS solids, %	0.32		VSS	5.4					BOD	0.0000	
Specific gravity of WAS	1.02	A							TSS	0.0000	
Secondary scum solids, %	0.5								VSS	0.0000	
Specific gravity of secondary scum	0.95								Concentrations, mg/L		
Volatile content of scum, VS/TS	0.90								BOD	9.9	
BOD content of scum, BOD/VS	0								TSS	6.4	
Scum spray water flow/nozzle, gpm	0								VSS	5.4	A
Total tankage surface area, sq ft	28997										
Spray area per nozzle, sq ft	58										
Total number of nozzles	500										

Note: Secondary scum flow and solids loading are specified by the user. Secondary scum flow and solids loading are added to the mass balance at secondary treatment.

Specify scum spray water flow/nozzle to be zero if spray water is not used. Spray water can be specified as part of process water recycled from secondary effluent.

If no data are available for scum and spray water, enter zero for scum flow and scum % solids and for scum spary water flow per nozzle.

Entry of "Sludge mass that contains PolyP" is applicable only for systems with excess biological phosphorus removal. Enter zero if not applicable.

Assumptions:

FINAL EFFLUENT

Process water flow, mgd	0.00		Final effluent flow, mgd	14.34
Mass loadings, k-lb/d			Mass loadings, k-lb/d	
BOD	0.00		BOD	1.18
TSS	0.00		TSS	0.77
VSS	0.00		VSS	0.65
Concentrations, mg/L			Concentrations, mg/L	
BOD	9.90		BOD	9.90
TSS	6.40		TSS	6.40
VSS	5.40		VSS	5.40

Note: Process water is recycled secondary effluent to be used for scum spray, centrifuge/belt press wash water and/or chemical make-up water.

SLUDGE THICKENING										
Recycle flows entering process unit, incl.:										
Flotation Thickening										
Influent Flow, mgd	0.57		Thickener Overflow, mgd	0.52		Thickened Sludge Flow, mgd	*0.053*		Thickener Underflow, mgd	0.00092
Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass loadings, k-lb/d	
BOD	6.35		BOD	0.0325		BOD	6.21		BOD	0.1065
TSS	15.65		TSS	0.1096		TSS	15.15		TSS	0.3915
VSS	12.16		VSS	0.0851		VSS	12.08		VSS	0.3043
Thickened solids, %	*3.36*		Concentrations, mg/L							
Solids capture (loss to overflow), %	*99.30*		BOD	7.51					Polymer Solution, mgd	0
Underflow solids, %	*5.00*	A	TSS	25.31					Mass loadings, k-lb/d	
Underflow TSS load, % of ext. inf. load	2.50		VSS	19.66					TSS	0
Overflow solids, %	0.0025									
Specific gravity of thickened sludge	*1.02*									
Specific gravity of underflow sludge	*1.02*									
Volatile content of thickened solids	0.80									
Volatile content of underflow solids	0.78									
Soluble BOD fraction in influent	*0.015*									
Underflow solids BOD/VS ratio	*0.35*									
Ratio of TSS to TS in overflow	*1.00*									
Ratio of VSS to TVS in overflow	*1.00*									
Assumptions:										
SLUDGE DIGESTION										
Influent Flow, mgd	0.1354		Effluent Flow, mgd	0.1354		Accumulation/loss, mgd	*0*			
Mass loadings, k-lb/d			Mass loadings, k-lb/d			Mass Loading, k-lb/d				
BOD	18.27		BOD	6.73		ISS	*0.00*			
TSS	48.48		TSS	33.98						
VSS	33.72		VSS	19.22						
Concentrations, mg/L			Concentrations, mg/L							
BOD	16181		BOD	5957						
TSS	42936		TSS	30096						
VSS	29861		VSS	17021						
VS reduction, %	*43.00*									
Digested solids BOD/VS ratio	*0.35*									
Assumptions:										

SECONDARY SLUDGE DIGESTION / SLUDGE STORAGE							
Influent Flow, mgd	0.1354	Effluent Flow, mgd	0.1354	Accumulation/loss, mgd	0		
Mass loadings, k-lb/d		Mass loadings, k-lb/d		Mass Loading, k-lb/d			
BOD	6.73	BOD	6.73	ISS	0.00		
TSS	33.98	TSS	33.98				
VSS	19.22	VSS	19.22				
Concentrations, mg/L		Concentrations, mg/L					
BOD	5957	BOD	5957				
TSS	30096	TSS	30096				
VSS	17021	VSS	17021				
VS reduction, %	0.00						
Digested solids BOD/VS ratio	0.35						
Notes: No supernatant is assumed to be withdrawn from the digesters and sludge storage tanks.							
Assumptions:							

SLUDGE DEWATERING							
Influent Flow, mgd	0.1354	Centrate Flow, mgd	0.1248	Dewatered Sludge Flow, mgd	0.0101	Wash water/chemical, mgd	0
Mass loadings, k-lb/d		Mass loadings, k-lb/d		Mass loadings, k-lb/d		Mass loadings, k-lb/d	
BOD	6.73	BOD	0.45	BOD	6.28	BOD	0
TSS	33.98	TSS	5.30	TSS	28.68	TSS	0
VSS	19.22	VSS	3.00	VSS	16.22	VSS	0
water	1095.19	water	1035.62	water	59.57	water	0
Influent TSS, %	3.01	Concentrations, mg/L		Solids loss due to release of PolyP, k-lb/d		Concentrations, mg/L	
Dewatered sludge solids, %	32.50	BOD	432.05	ISS	0	BOD	0
Specific gravity of dewatered sludge	1.05	TSS	5093.07			TSS	0
Solids capture (TSS and VSS), %	84.40	VSS	2880.37			VSS	0
Centrate BOD/VS ratio	0.15						
Ratio of TSS to TS in centrate	1.00						
Ratio of VSS to TVS in centrate	1.00						
Note: Solids capture is based on influent loadings only (does not include loadings due to wash water or chemical make-up water).							
Assumptions:							

Overall Plant Mass Balance Checks		
Flow Balance (mgd)	5.04E-04	(Sewer + Septage + Other Sludge + Polymer + Spray & Wash Water (if not recycled)) - Final Effl. - Dewatered Sludge - Screenings & Grit - Dig. Vol Change
Inert Solids Balance (k-lb/d)	3.04E-01	(Sewer + Septage + Other Sludge + Polymer + PolyP uptake - PolyP release) - Final Effluent - Dewatered Sludge - Screenings & Grit - Dig. Inventory Change
		Inert Solids (ISS) = TSS - VSS
		ISS balance should approach zero for mass balance check (except in cases where TSS/TS and VSS/TS are less than 1).
Percentage of Influent		
Flow Balance	0.00%	Flow Balance/(Sewer Flow + Septage Flow + Other Sludge Flow + Primary Scum Flow + Secondary Scum Flow)*100
Inert Solids Balance	2.36%	Inert Solids Balance/(Sewer ISS + Septage ISS + Other Sludge ISS + Primary Scum ISS + Secondary Scum ISS)*100

Note: Input Data are in pink and *italicized*. Calculated values are in black.

A denotes assumed value.

BW denotes value derived from BioWin simulation results.

* denotes that calculated value does not match given plant data.

Users are advised to list all assumptions associated with mass balance calculations!

Last modified: 5/3/99 PT

APPENDIX G

SOLIDS SYSTEM EVALUATION TM

BROWN AND CALDWELL

TECHNICAL MEMORANDUM – FINAL

DATE:	OCTOBER 20, 2006
TO:	ANGELA MORROW, CITY OF ESCONDIDO
FROM:	VICTOR OCCIANO, BROWN AND CALDWELL
PREPARED BY:	KEN FONDA, BROWN AND CALDWELL
SUBJECT:	CITY OF ESCONDIDO HALE AVENUE RESOURCE RECOVERY FACILITY - SOLIDS HANDLING PROCESSES EVALUATION

SUMMARY

Results of the assessment performed on the solids handling facilities at the Hale Avenue Resource Recovery Facility (HARRF) are summarized in this technical memorandum. This assessment is part of the overall assessment of the treatment capacity of the HARRF. Tasks completed for this assessment include the following:

- Evaluation of the thickener performance and polymer consumption
- Desktop capacity analysis of thickeners
- Evaluation of the digester performance of mixing and heating systems
- Evaluation of the capacity of the dewatering centrifuges
- Review of results with City staff
- Preparation of a technical memorandum

Evaluation of the capacity of unit processes and components is based on available information provided by the City in the form of plans, O&M manuals, previous reports and studies, and information observed during a site visit conducted on March 2, 2006. Criteria used to evaluate the capacity of unit processes and components are based on Brown and Caldwell design guidelines and regulatory requirements as they apply to the pertinent process. Table 1 summarizes the design criteria used for this evaluation. Since Mass Balance projections for current flow have been based on limited data, solids projections

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

developed for near term improvements to handle 18mgd have been used to calculate equivalent process capacity.

Equivalent influent capacity for each process is determined by dividing the available capacity (solids loading, flow, solids retention time (SRT), etc.) by the required capacity multiplied by the 18 mgd average daily plant influent flow. If the available capacity is greater than the required capacity the resulting equivalent influent flow capacity will be higher than the 18 mgd average influent flow. Evaluation of process performance is based on historical and laboratory data provided by the City. Where interim process modifications could provide improved performance, the improved process performance has been used for comparison to existing performance.

Table 1. Process Capacity Evaluation and Design Criteria

Process Unit	Item	Units	Value	Source
DAFT	Solids loading rates[1] WAS Only Thickening • Average day, one unit out of service • Peak day, all units in service Co-Thickening • Average day, one unit out of service • Peak day, all units in service	Lb/sf-d Lb/sf-d	15 18 30 45	Brown and Caldwell Design Guideline and Plant Experience, without polymer
	Air to Solids Ratio	---	0.03	
	Minimum liquid retention time	min.	0.75	
	Saturation constant	mg/L	100.7	Henry's Law, air at 75 ^oF
Anaerobic Digestion	Vector Attraction Reduction	minimum % volatile solids reduction (VSR)	38	EPA 40 CFR 503 Part B
	Process to Significantly Reduce Pathogens (PSRP) for Class B biosolids Solids Retention Time SRT at 35 to 55° C • Average day, largest unit out of service • Peak 2-week, all units in service	Days	20 15	EPA 40 CFR 503 Part B Brown and Caldwell Design guideline for average conditions to provide better VSR and ensure regulatory compliance when one unit is out of service
	Active digester volume is based on number of digesters that are heated and mixed	---	---	EPA 40 CFR 503 Part B
Centrifuge Dewatering	Hydraulic Loading	gpm	75 to 150	Vendor information
	Solids Loading	lb/hr	550 to 2800	Vendor information

(1) Previous capacity assessment prepared by MWH used a solids loading rate of 45 lb/sf. BC believes this loading rate could not be achieved without adding a significant amount of polymer.

Environmental Engineering And Consulting

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Findings

Based on the process design criteria shown in Table 1 for each process, the following findings/conclusions can be made:

1) DAF Thickening – The process capacity is currently below the rated plant capacity of 18 million gallons per day (mgd) on the basis of solids loading when only one of two existing units is in service.

2) Anaerobic Digestion – Current digester volume does not meet SRT required by EPA 40 CFR 503 regulations when the largest unit is out of service.

EPA 40 CFR 503 Part B regulations define three alternatives for meeting Class B pathogen requirements:

1. Monitoring for indicator organisms – coliforms <2 million MPN per gram or <2 million CPUs per gram
2. Biosolids treated in a Process to Significantly Reduce Pathogens (PSRP)
3. Biosolids treated in a Process Equivalent to a PSRP

One of the methods defined for Alternative 2 in Table 5-7 of the 503 regulations is anaerobic digestion with a minimum of 15 day SRT at 35 to 55 deg C listed. Unlike Class A time and temperature requirements, there is no flexibility to reduce the time by increasing the temperature.

The BC criteria recommends 20 day SRT at Average flow conditions with one unit out of service to ensure proper pathogen reduction and better volatile solids destruction. BC recommends 20 days under these conditions because there is a possibility of having a peak event when one unit is out of service. Escondido currently operates all the digesters now so that they meet the minimum 15 day requirement under all flow conditions. Based on this criteria the HARF does not have enough digester capacity to allow taking one digester out of service for cleaning or process upset. Cleaning can take longer than 2 weeks which is the duration of our peak flow criteria. Typically process upsets can be cured in a shorter period of time by using sludge from a healthy digester to seed a sick one to bring it back on line more quickly. Taking a digester out of service for cleaning is typically something that is planned well in advance and can be scheduled during a period of the year when influent flows are low, like the summer months.

Base on solids projections prepared for the 18 mgd interim solution if the SRT criteria is reduced to 18 days, the digesters would have an equivalent capacity of nearly 18 mgd with one unit out of service assuming co thickening to 6%. The ratio of peak 2 week flow to average flow for co thickening is 1.5/1. Therefore, unless the City wants to use one of the other alternative methods listed above like measuring coliforms, reducing the SRT could put them in risk of not meeting Class B requirements. Preliminary results of ongoing research by the Water Environment Research Federation (WERF) examining fecal coliform regrowth in centrifuge dewater cake appears to indicate there is a strong possibility that there could be an order of magnitude increase in coliforms if the cake is stored for more than two days before it is applied to the field. Therefore, the combination of marginal retention time and centrifuge dewatering could increase the probability that the dewatered cake would not meet the fecal coliform requirements for Class B. This would essentially render the product unusable reducing Escondido's disposal options which would invariably lead to higher disposal costs. If the City was forced to meet the fecal coliform requirement because of the insufficient SRT a possible way of reducing the potential for regrowth would be to operate the digesters in a series mesophilic mode. BC knows of several plants that are doing this and have significantly reduced coliform regrowth to meet acceptable levels.

Improved thickener performance would have a direct impact on available digester capacity. Increased thickened solids concentration will reduce hydraulic loading to the digesters and increase the available capacity. To meet the required SRT at average daily flow with one unit out of service for current plant capacity, the combined solids concentration of TWAS and primary sludge would need to be above 6.5% If the thickening system was modified to co-thicken both primary and secondary sludge to a concentration of 6.0%, the equivalent Digester Capacity would be 16 mgd with one unit out of service and 24.3 mgd with all units in service.. The digester currently meets vector attraction requirements by providing greater than 38% volatile solids reduction, which is the minimum VSS reduction required by EPA 503 Regulation.

3) Centrifuge Dewatering – Centrifuge capacity is adequate for the solids generated at the projected average future plant influent flows of 27.5 mgd. Additional capacity may be provided by running the centrifuges more than 12 hours per day. Emergency capacity may be provided by operating the third centrifuge and increasing the pressure capacity of the digested sludge transfer pumps or pigging the line between the secondary digester and the dewatering feed tank.

Recommendations

Recommendations to improve the performance of the thickening, digestion, dewatering processes are summarized in Table 2. These recommended improvements would not take the place of new facilities required to increase the solids processing capacity to an equivalent plant capacity of 18.0 or 27.5 mgd. Instead, they are intended to bring the existing facilities to the performance level of new facilities. Detailed discussions are provided in the main body of this technical memorandum (TM).

Table 2. Recommended Solids System Process Performance Improvements

Process	Item	Recommended Improvement	Purpose
DAF Thickening	Polymer feed	Move polymer feed closer to discharge point of pressurized flow	Improves mixing efficiency by using turbulence of rising bubbles
		Modify center feed piping to accommodate new polymer discharge point	Necessary to implement change noted above
	Thickener effluent	Replace thickener overflow weir with submerged launder pipe	Provides cleaner water for recycle to pressurized flow system

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 2. Recommended Solids System Process Performance Improvements			
		Provide control valve on thickener effluent line for level control in the DAFT	Controls liquid level to maximize drainage through float
	Saturation System	Replace pressurized flow pumps to meet necessary recycle flow for solids loading	Provides sufficient flow for air saturation
		Add second pressurization tank or increase operating level	Provides sufficient residence time for air to dissolve; reduces possibility of vortexing
		Add continuous vent to purge excess nitrogen	Increases gas absorption and improves stability
		Modify inlet and outlet piping to prevent vortexing and inlet pipe flooding	Vortexing can bring in undissolved air to DAFT discharge point, disturbing small bubbles being released and break up flock as it forms with rising bubbles
	General	Consider modifying DAFTs to co-thicken primary and secondary solids	Provides homogeneous feed to digesters; reduces soluble BOD return to head of the plant; can allow higher primary clarifier surface overflow rate due to continuous sludge withdrawal
Anaerobic Digesters	Digester Feed Sequencing	Feed primary and secondary solids simultaneously to all digesters minimum of 24 feed cycles per day	Stabilizes operation through more consistent solids feed; prevents gas production spikes
	Digester mixing	Verify lances and draft tubes are clear	Ensures system is operating as designed
		Verify draft tube mixing capacity provides 16 to 24 turnovers per day	Verifies mixing capacity is sufficient to prevent solids deposition, surface matting, dead zones, and hot spots; provides efficient contact of existing biomass with new food
		Provide dedicated compressors for Digesters Nos. 1 and 2	Needed to provided balanced operation to draft tube gas mixing systems
		Perform dye study	Confirms mixing efficiency in digesters, particularly for Digester 1
Centrifuge Dewatering	Polymer application	Provide sludge samples to centrifuge and polymer suppliers	Verifies that the sludge character has not changed since centrifuges have been placed into service
		Perform polymer trials	Establishes whether a new polymer should be used
	Scale Control	Perform periodic acid cleaning of centrate pipes and/or use polyphosphate scale inhibitors	Maintains centrate system hydraulic capacity to prevent backups from occurring

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

EXISTING FACILITIES AND CONDITIONS

The HARRF is rated to provide 18 MGD of secondary treatment and currently treats approximately 15 MGD of predominantly domestic sewage. The projected average daily influent flow to the HARRF at build out flow is approximately 27.5 MGD. Existing solids handling processes are briefly described below. A detailed listing of process equipment is included in Table 3.

- Dissolved air flotation thickening (DAFT) of waste activated sludge (WAS)
  - Two DAFTs, each 35-foot diameter, are available
  - One is operated under average conditions and two are operated at peak
  - The DAFT in service is selected based on seasonal changes
- Mesophilic anaerobic digestion
  - Three digesters: one 80-ft diameter (Digester 1 - 0.94 million gallon) and two 85-ft diameter (Digesters 2 and 3 - 1.06 million gallon each)
  - All three digesters operate continuously; all are heated and mixed with three confined gas draft tube mixing systems
  - One 55-ft diameter (0.409 million gallon) secondary digester used for digested sludge storage prior to dewatering; this digester is unheated and unmixed and cannot be included in digester capacity for solids residence time (SRT) calculations
- Centrifuge dewatering
  - Three (two duty, one standby) 150 gpm Andritz high solids units
  - Dewatering operation performed during swing and graveyard shifts, seven days a week
  - Dewatered cake solids discharged into a single truck loading bin with four bottom gates for even distribution in the trucks; truck loading bin has capacity for 6 hours storage under current conditions

Table 3. Existing Solids Handling Facilities

Item	Value	
Dissolved Air Flotation Thickeners		
Thickener ID Number (Location)	1 (East)	2 (West)
Tank Diameter, ft	35	35
Side water depth, ft	8.5	10.5
Surface Area, sq ft	962	962
Cover	None	None
Number of float box	1	1
Overflow weir location, length, ft	External, 109	External, 109
Number of surface skimmers, speed control	6, Constant speed	6, Constant speed

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 3. Existing Solids Handling Facilities		
Item	**Value**	
Bottom Scrappers	2	2
Thickened sludge pumps – Manuf.	Seepex	Seepex
Pump type	Progressive cavity	Progressive cavity
Pump capacity, gpm	260	260
Pump rated pressure, psi	50	36
Pressurization System		
Pump Manuf.	Peerless	Peerless
Pump type	Centrifugal	Centrifugal
Pump capacity, gpm	450	500
Pump pressure, ft	162	175
Compressor Manuf.	Comp Air	Comp Air
Compressor Type	Piston	Piston
Compressor capacity, scfm	15.0	17.2
Compressor capacity, lb/hr	67.4	77.29
Compressor pressure, psi	100	100
Pressurization Tank size	4'-6" x 5'-4"	4'-6" x 5'-4"
Pressurization Tank liquid depth	2'-6"	2'-6"
Pressurization Tank liquid volume, gal (including bottom knuckle)	328	328
Pressurization Tank Level control	Yes	Yes
Vent	Manual	Manual
Polymer System		
Feed pump type	Diaphragm metering	Diaphragm metering
Feed pump capacity, gpm		
Polymer type	Clarifloc C-331, Mannich, cationic	Clarifloc C-331, Mannich, cationic
Polymer dosage, avg lb/ton dry solids	7.5	7.5
Polymer dosage, peak day lb/ton dry solids	10	10

Anaerobic Digesters			
Digester ID Number	1	2	3
Digester type	Primary	Primary	Primary
Tank Diameter, ft	80	85	85
Side water depth, ft	25	25	25
Unit volume, 1000 gal	940	1,061	1,061
Number of top access opening	1	1	1
Top access opening diameter, ft	8	8	8
Number of side access opening,	2	2	2
Side access opening dimension, ft x ft	3 x 3	3 x 3	3 x 3
Cover type	Concrete, Fixed		

Environmental Engineering And Consulting

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 3. Existing Solids Handling Facilities			
Item	**Value**		
Mixing	Gas, Draft Tube		
Number of draft tubes	1	3	3
Number of lances	4	6	6
Lance diameter, in.	3	3	3
Gas Compressor type	Rotary Lobe		
Gas Compressor Manuf	Roots		
Number of Gas Compressor(s)		3 (2+1)	4 (3+1)
Gas Compressor capacity, scfm	1,000		
Gas Compressor pressure, psi	6.5	7-8	
Recirculation			
Number of pumps	1	3	
Pump type	Centrifugal		
Pump Manuf	Vaughan		
Capacity gpm	220-250	220-250	
Operating head, ft	24	24	
Horse Power	5	5	
Speed, rpm	1170	1170	
Heating			
Number of Heat Exchangers	1	1	1
Type	Spiral	Spiral	Spiral
Manufacturer	Alfa Laval	Alfa Laval	Alfa Laval
Size, Million BTU	1	1	1
Dewatering System			
Storage Tank (Secondary Digester)			
Diameter, ft	55		
Sidewater Depth, ft	23		
Volume, 1000 gal	409		
Dewatering Equipment			
Manufacturer, Model	Andritz, D5L		
Number of Centrifuge(s)	2 Duty, 1 Standby		
Capacity each, gpm	150		
Operating Mode	Auto/torque		
Operating schedule, hrs per day	12		

Based on the data collected, the average performance of each solids handling process is as follows:

- DAF Thickening – 99 percent capture, 3.5 percent solids concentration
- Anaerobic Digestion – 43 percent Volatile solids reduction in primary digesters
- Centrifuge Dewatering – 84 percent capture, 32 percent solids concentration

A site visit was conducted on March 2, 2006 to obtain additional equipment information and to interview operations staff to gain operator insight on process concerns. The following information was obtained during the site visit:

- Primary Sludge Pumping
  - Seven diaphragm pumps exist made by two manufacturers; Gorman Rupp and Dorr Oliver
  - Capacity of the Gorman Rupp pumps are 4.5 gallons per stroke and the Dorr Oliver pumps are 3.8 gallons per stroke. The strokes on each pump are limited to a maximum of five strokes per minute.
  - Pumps controlled by blanket level to maintain 12 inches at hopper end
  - Progressive cavity pumps replaced due to plugging problems prior to installation of new headworks screens - no plugging problems noted since change was made
  - Digester feed to primary digesters rotates every two hours: one digester primary sludge only, one digester TWAS only, and one digester holding with no feed.
  - Unfiltered, undried plant air used to supply the diaphragm pumps - causes additional maintenance on diaphragm operators due to oil and moisture in the supply air (per Operations staff); industry standard is to use instrument air (filtered and dried air)
- Dissolved Air Flotation Thickening
  - DAFT equipment recently rehabilitated within the last 5 years - replaced all major mechanical equipment including support systems (i.e. compressors, pressurization pumps, and pressurization tanks)
  - Polymer (Polydyne Clarifloc C-331, Mannich, cationic) added upstream of DAFT in the WAS feed line
  - One DAFT used always, the other remains empty for redundancy
  - DAFT No. 2 (west) used during winter months
  - DAFT No. 1 (east) used for the remaining months for ease of access and maintenance - lower elevation
  - Thickened WAS pumped to digesters similar to primary sludge, i.e., one digester rotated every 2 hours
- Anaerobic Digestion
  - Digester heating upgrades being designed by HDR to replace Cleaver Brook Steam boiler system (100% design completed, startup expected by spring of 2007)
  - Existing steam heating system goes through two steps to heat the sludge: through a steam-to-water heat exchanger first then through a water-to-sludge heat exchanger

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

- Heating controlled by deactivating the steam boiler system when the sludge temperature reaches a setpoint; heat is exchanged from the sludge to the hot water side during cooling cycle
- Digesters are gas mixed.
- Operations staff indicated a preference towards replacement of the existing gas mixing system
- One Roots rotary lobe compressor feeds gas to mixing systems of the two oldest digesters (Digester Nos. 1 and 2)
- Balancing the gas flow to Digester Nos. 1 and 2 is difficult because of unequal number of lances; gas flow meters on each line feeding the draft tubes used for measurement only and not control
- Digester No. 3, built in 1999, has its own dedicated gas compressor to feed the mixing system
- Digesters Nos. 1 and 2 cleaned after startup of Digester No. 3
- Digesters had not been cleaned for about 18 years - found considerable amount of floating debris and grit accumulated in the bottom when the last cleaning was performed
- HARRF staff plans to clean the digesters on a 5 to 7 years cycle; all digesters are now due for cleaning

▪ Centrifuge Dewatering

- Three Andritz D5L high solids, (3,300 RPM/3,164 G) centrifuges installed in 2002 to replace plate and frame presses
- Centrifuge dewatering performed 12 hours per day, 7-days per week during the swing and graveyard shifts
- Dewatered cake loaded directly into trailers through the cake hopper
- A shaftless screw conveyor moves the cake to four loadout gates to facilitate even loading into trailers
- Full trailers picked up in the morning and transported off site
- Polymer (Polydyne Clarifloc WE-122, emulsion, cationic) added to aid in dewatering
- Ferric chloride added at times during hot summer months to reduce odors generation
- Digested sludge transferred from the secondary digester using progressive cavity pump
- Transfer pipelines are insufficiently sized, restricting the amount of flow that can be dewatered (per HARRF staff)
- Calcium scale build up in the centrate lines requires periodic rodding and high pressure jetting to maintain capacity; acid added in the past, but mechanical cleaning still necessary

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

CAPACITY EVALUATION

The current capacity of the thickening, digestion and dewatering processes was evaluated based on sludge projections (using BioWin) for near term process improvements to treat 18 mgd. The results of the modeling are included in the System's Integration Technical Memorandum. The average solids production rates used for the evaluation are follows:

- Primary sludge
  - 42,378 lb/day, 4.48% solids concentration, 0.115 mgd (for separate thickening)
  - 42,378 lb/day, 2.0% solids concentration, 0.254 mgd (for co-thickening)
- WAS
  - 15,200 lb/day, 0.33% solids concentration, 0.697 mgd

Equivalent plant influent capacity for the existing solids handling facilities using the current operating mode was calculated by multiplying the 18 mgd plant influent raw wastewater flow used for the BioWin model by the ratio of required process unit size to available process unit size. For instance, the allowable solids loading for the existing DAFT surface area with one unit out is 14,400 lb and the WAS solids loading for projected flow of 18 mgd is 17,280 lb/day. The ratio of the allowable loading to the projected loading equals 0.88. By multiplying the current plant raw wastewater influent flow of 18 times this ratio of 0.88 the equivalent influent capacity equals 15 mgd.

Brown and Caldwell design guidelines, manufacturer's recommended operating conditions, and regulatory requirements were used to establish the design criteria used for the evaluation of the existing solids processing units and ancillary equipment. The criteria are summarized in Table 4.

Table 4. Process Capacity Evaluation and Design Criteria

Process Unit	Item	Units	Value	Source
DAFT	Solids loading rates[1] WAS Only Thickening Average day, one unit out of service Peak day, all units in service Co-Thickening Average day, one unit out of service Peak day, all units in service	 Lb/sf-d Lb/sf-d 	 15 18 30 45	Brown and Caldwell Design Guideline and Plant Experience, without polymer
	Air to Solids Ratio	---	0.03	
	Minimum liquid retention time	min.	0.75	
	Saturation constant	mg/L	100.7	Henry's Law, air at 75 ^{o}F
Anaerobic Digestion	Vector Attraction Reduction	minimum % volatile solids reduction (VSR)	38	EPA 40 CFR 503 Part B

Environmental Engineering And Consulting

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 4. Process Capacity Evaluation and Design Criteria				
	Process to Significantly Reduce Pathogens (PSRP) for Class B biosolids Solids Retention Time SRT at 35 to 55° C Average day, largest unit out of service Peak 2-week, all units in service	Days	 20 15	EPA 40 CFR 503 Part B Brown and Caldwell Design guideline for average conditions to provide better VSR and ensure regulatory compliance when one unit is out of service
	Active digester volume is based on number of digesters that are heated and mixed	---	---	EPA 40 CFR 503 Part B
Centrifuge Dewatering	Hydraulic Loading	gpm	75 to 150	Vendor information
	Solids Loading	lb/hr	550 to 2800	Vendor information
(1) Previous capacity assessment prepared by MWH used a solids loading rate of 45 lb/sf. BC believes this loading rate could not be achieved without adding a significant amount of polymer.				

The estimated capacity of each solids processing unit, assuming continued separate thickening in terms of equivalent raw wastewater plant influent flow, are reported in Table 5.1. Equivalent capacity of each solids processing unit, assuming co-thickening in the DAFTs to 6.0% in terms of plant influent flow, are reported in Table 5.2.

Table 5.1. Summary of Process Capacity Evaluation WAS Only Thickening[1]	
Evaluation Criteria	Equivalent Plant Influent Capacity (MGD)
DAF Thickening	
Average Solids Loading - one unit out of service	15
Average Solids Loading – all units in service	30
Average Saturation System Capacity	30
Anaerobic Digestion[1]	
Average Solids Retention Time - one unit out of service (20-day minimum)	11
Average Solids Retention Time – all units in service (20-day minimum)	17
Vector Attraction Reduction (38% VSR)	Meets Requirements
Centrifuge Dewatering	
Hydraulic Loading – one unit out of service, 7 day/24 hr per day operation (150 gpm each)	27.5
[1]Assume thickened WAS concentration of 5.0%. Actual performance would need to be determined through pilot testing.	

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 5.2 Summary of Process Capacity Evaluation (Co-thickening)	
Evaluation Criteria	Equivalent Plant Influent Capacity (MGD)
DAF Thickening	
Average Solids Loading - one unit out of service	9
Average Solids Loading – all units in service	18
Average Saturation System Capacity	30
Anaerobic Digestion	
Average Solids Retention Time - one unit out of service (20-day minimum)	16
Average Solids Retention Time – all units in service (20-day minimum)	24
Vector Attraction Reduction (38% VSR)	Meets Requirements
Centrifuge Dewatering	
Hydraulic Loading – one unit out of service, 7 day/24 hr per day operation (150 gpm each)	34.4

As shown in Table 5.1 and 5.2, if the solids loading rate on the DAFTs for WAS only could be increased to 2415/sf, the DAFTs would have an equivalent influent capacity greater than 18 mgd. This could be achieved with additional polymer dosing. The amount needed would be determined in the field. The digester equivalent influent capacity could be met with all digesters in service if the average SRT was reduced to 19 days. The equivalent raw wastewater influent capacity related to the thickening and digestion units is less than the rated capacity of the plant of 18 mgd. Process modifications and equipment additions will be necessary to meet current rated plant capacity. Evaluation of future capacity requirements will be provided in a separate technical memorandum after all the process optimization recommendations have been evaluated.

PROCESS PERFORMANCE AND OPTIMIZATION

DAFT Thickening

Review of process configuration and sizing of DAFT equipment has identified several improvements that could be made to increase solids concentration. For instance, if DAFT solids loading could be increased by controlling the liquid level, then the equivalent plant influent capacity could be increased. Similarly, increasing the thickened sludge concentration will increase the digester SRT. Pilot testing would be needed to determine actual process performance for both WAS only and co-thickening systems.

Increased thickened solids concentration has a direct impact on hydraulic loading and solids retention time in the anaerobic digestion system. Although DAFT performance is somewhat site-specific, at the loading rates being applied, thickened solids concentrations of

Environmental Engineering And Consulting

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

at least 4% (without polymer addition) are to be expected. Even higher thickened solids concentration is to be expected at the current polymer dosage rates. Since these DAFTs have polymer addition and the thickened solids concentration is only 3.5 percent, it may be concluded that polymer is being wasted due to inefficient polymer application. However, since the secondary solids average SVI is over 200, thickened solids concentrations could be as low as 3.5 percent - consistent with observed process performance. Implementation of secondary treatment improvements recommended in a separate TM that addresses the biological process, would reduce the SVI and improve thickener performance and possibly reduce polymer usage.

Other factors effecting polymer usage include percent active solids versus total solids delivered, dilute polymer storage, and polymer type (e.g., emulsion versus Mannich). Since total solids delivered include both active and inert solids, periodic spot checks of polymer loads delivered would verify the supplier is providing the specified product in their contract. The City of San Diego uses this procedure as a part of their quality control program and has identified loads that were several percentage points below the contracted active solids. Dilute polymer should be used within an eight hour period according to the manufacturer's technical data sheet. The product can loose some of its effectiveness if used after this period. The method used for mixing batches of polymer can also affect the character of the polymer once a batch has been made. Mixing the polymer too heavily can break down the polymer molecular chains and reduce the effectiveness of the polymer. At other locations, DAFT performance has also been improved by switching from a Mannich polymer to an emulsion. Furthermore, the switch to emulsion polymer may be more prudent since there is a limited number of Mannich polymer suppliers.

Recommended modifications to the DAFTs are discussed in detail below:

Polymer Injection Improvements

Polymer is currently being injected into the WAS feed line upstream of the DAFTs. Polymer is more effective when it is introduced at a point of high mixing energy and where precipitated bubbles and WAS solids are already blended. Based on vendor drawings, the WAS flow and PF enter DAFT 1 from the side rather than down the center of the tank (as indicated on the DAFT 2 as-built drawings). The DAFT 1 pressurized flow (PF) line should be re-routed to enter from the top at the center column. In addition, a new polymer line should be installed in the center column, outside the PF line, and terminate just above the PF discharge point. This arrangement provides polymer feed at the point of high turbulence. The polymer can also adhere readily to the float being formed as the dissolved air is released in the center column. The polymer feed point to DAFT 2 should also be relocated similar to DAFT 1.

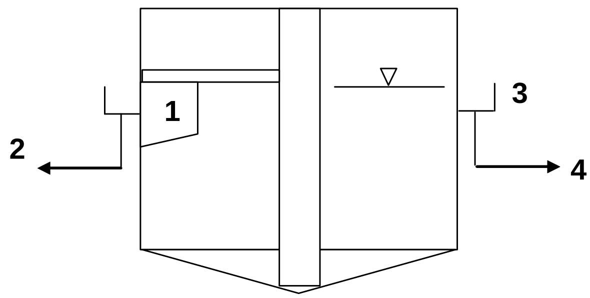
Thickener Overflow Weir Improvements

As noted in Table 2 and shown on Figure 1, the existing DAFTs have an outboard launder to collect thickener overflow. Because this weir is fixed, liquid level in the DAFT cannot be controlled. Although there is a baffle plate in front of the weir, the effluent typically has a higher solids content than the subnatant below the internatant zone (the layer between the float and the subnatant where a mixture of float and subnatant exist). Since the subnatant is recycled through the saturation system, having a lower solids concentration would improve the efficiency of the saturation system. We recommend replacing the fixed effluent weir with a submerged launder pipe with orifices just above the bottom scraper. A schematic of a typical BC DAFT design with a submerged effluent launder pipe is shown on Figure 2.

The BC design includes a single pipe that tees off from the submerged launder through the outer wall into a two compartment thickener overflow (TO) box. A pipe with a backpressure control valve connects the two TO box compartments. The first compartment provides level control for the DAFT and the second compartment provides a gravity drain and emergency overflow back to the headworks. By adjusting the backpressure on this line, the level in the DAFT can be raised or lowered to control the float level being removed from the DAFT. Controlling the level in the DAFT provides optimum drainage through the float to improve float concentration.

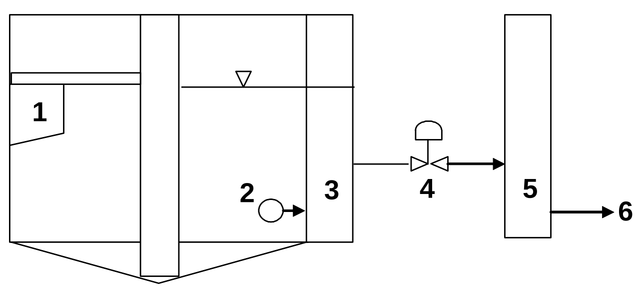
Environmental Engineering And Consulting

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833



Key Notes:
1. Float Box
2. Recycle flow to saturation system
3. Thickener Overflow (TO) outboard launder
4. Gravity Drain to Headworks

Figure 1 - Existing DAFT Schematic



Key Notes:
1. Float Box
2. Submerged Launder Pipe
3. Thickener Overflow (TO) Box
4. Level Control Valve
5. Gravity Drain/Emergency Overflow Box
6. Gravity Drain to Headworks

Figure 2 - Typical BC Design DAFT Schematic

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Saturation System Improvements

Review of the existing saturation system revealed several shortcomings. Recommended improvements are described below. A schematic of a typical BC-designed saturation tank is shown on Figure 3. A copy of the existing saturation tank vendor drawing is provided in Attachment A.

<u>Recycle Flow Pumps</u>

Based on the projected solids quantities for 18 mgd, the recycle flows for average daily and peak day flow conditions are 773 and 1046 gpm, respectively. As shown in Table 1, the capacity of the DAFT recycle pumps are 500 gpm each. It is assumed that both DAFTs would be in service at peak day conditions. However, the recycle flow pumps limit the capacity of the DAFTs. The capacity could be increased by enabling both recycle pumps to operate with one DAFT modifying the existing pump impeller or speed to increase the capacity, or installing an additional redundant pump. An additional redundant pump would need to be located outside of the saturation building. There is insufficient space for this extra pump in the existing building. Interconnecting the two DAFT pressurization pumps requires further evaluation that is outside of the scope of this study. The ability to change the impeller size or motor speed would require further investigation that is also outside the scope of this study.

<u>Liquid Level Control</u>

The required liquid volume for average daily flow is 694 gallons to provide the minimum 0.75 minutes of detention time. The current level in the saturation tank only provides approximately 327 gallons. Therefore, the operating level needs to be increased by about 0.5 feet (assumes both DAFTs are in operation at both average and peak daily flow.) To provide additional volume for peak conditions with (one tank in service) a second pressurization tank could be installed. This tank would also need to be installed outside the building.

<u>Pressurization Tank Inlet And Outlet Piping</u>

The existing pressurization tank has an inlet pipe that discharges at the top of the tank onto a target baffle; the outlet pipe draws from the bottom of the tank. A baffle does not exist on the outlet side of the tank to prevent vortexing, which can introduce larger bubbles in the pressurized flow stream. These undissolved bubbles can break up the floc forming within the DAFT.

As shown on Figure 3, the typical BC design places the inlet pipe through the side of the tank, discharging against the target baffle. Subnatant is withdrawn from the bottom of the tank which has an anti-vortex baffle to prevent a free vortex from forming. Vortices can also be prevented by raising the liquid operating level.

Nitrogen Purge

A frequently overlooked factor is the effect of nitrogen and oxygen solubility on gas bubble formation. The solubility of nitrogen is roughly half that of oxygen at a given temperature and pressure. Since air is composed of 78% nitrogen and 21% oxygen (by volume), a significant fraction of the air remains undissolved. Without venting the headspace, the quantity of gas that dissolves is reduced, lowering the overall capacity of the air saturation system.

BC recommends continuously venting a portion (approximately 10%) of the air in the headspace to improve saturation efficiency. The existing saturation tanks have vent lines and pressure relief lines that could be retrofitted to allow continuous venting to take place. Since most of the energy consumed in the DAFT process comes from the pressurized flow system, this can result in significant energy savings as well.

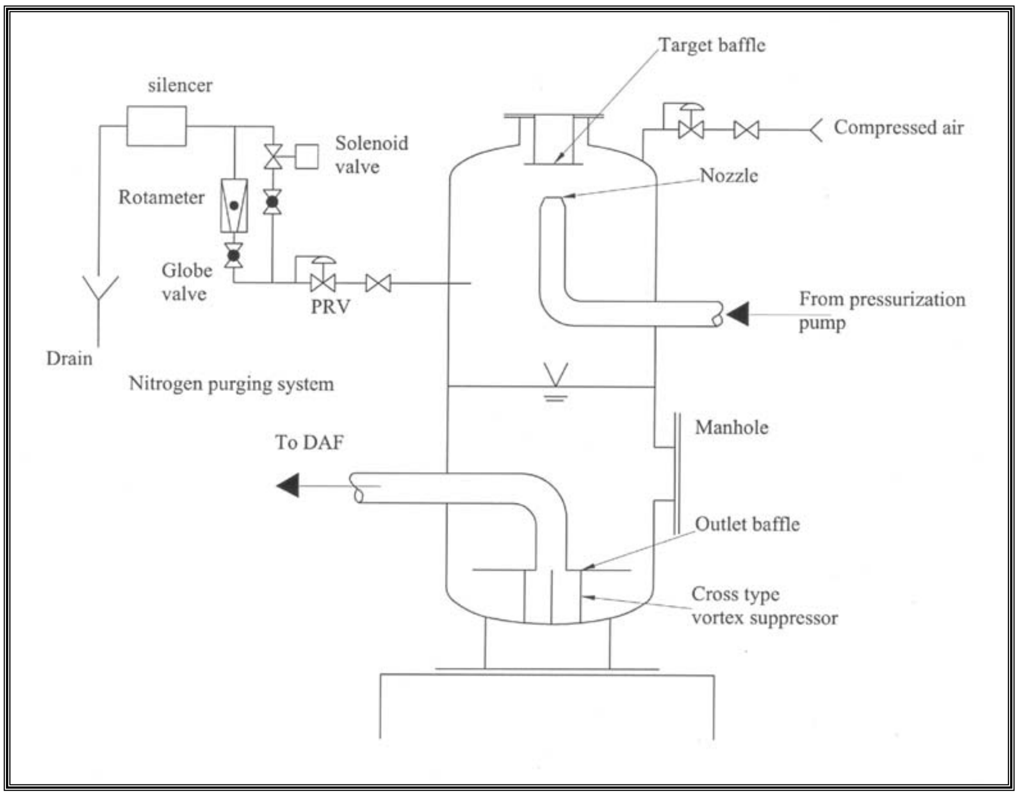


Figure 3 – Air Saturation Tank Schematic

(Source: "State-of-Practice of DAFT Technology – Is There Still a Place for It?", Bratby, J, et al; WEFTEC '04 77TH Annual Technical Exhibition and Conference, New Orleans, LA October 2 – 6, 2004)

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Other Process Improvements

Other improvements are described below.

Co-thickening Primary Sludge and WAS

Co-thickening the primary sludge and WAS together in the DAFTs is another process modification that could be implemented to improve combined thickened sludge concentration as well as digester performance and capacity. Primary solid particles are larger than WAS floc. Thus, the air bubbles adsorbs to the primary solids more readily, causing them to float. Consequently, in co-thickening, the WAS particles adhere to the large, buoyant primary solids thereby improving WAS solids removal efficiency.

Although performance is site-specific, BC's experience at other facilities has shown that a combined solids concentration of 6 to 6.5 percent is achieved through co-thickening as compared to the current combined solids concentration of approximately 4 percent. Increasing the combined solids concentration to 6 percent would increase the digester SRT during average flow (with one unit out of service) to over 19 days and increase the equivalent plant influent flow digester capacity to 17.5 mgd. Therefore, co-thickening could eliminate the need to construct an additional digester to meet digester requirements for an equivalent plant flow of 18.0 mgd if the average SRT criterion were reduced to 19 days. Additional digester capacity would be needed for plant average daily flow greater than 18 mgd or the plant would need to operate with all digesters on line. This would leave no room to take a digester out of service due to process upset or cleaning. Co-thickening would also yield a more homogeneous solids feed to the digesters, resulting in a more stable digester operation and better gas production (digestion of primary sludge produce more gas than digestion of WAS alone or primary sludge/WAS combined).

Noting that the primary to secondary sludge ratio for HARRF is approximately 2:1, the combined solids loading to the DAFTs would increase by threefold. To enable the existing DAFTs to be used to co-thicken primary and secondary sludge, the saturation system and polymer feed systems must be expanded along with the other recommendations noted above.

Anaerobic Digestion

Suggested improvements to the anaerobic digestions are presented below.

Digester Heating

As noted earlier, the existing digester heating system using steam boilers has been a difficult system to control and operate effectively. For this reason, the City of Escondido has hired a consultant to design modifications to the digester heating system. Therefore, evaluation of the capacity of the existing digester heating system would not be useful at this time and is not included in this technical memorandum.

Digester Mixing

Proper digester mixing is essential to effective volatile solids reduction. Gas mixing can be an efficient and economical means of mixing when proper gas flow is provided to the lances

Environmental Engineering And Consulting

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

and draft tubes and lances are kept clean. The low volatile solids removal rate (43 percent) observed from the data provided by the City indicates that the mixing system is not functioning as designed. Typically, digesters that are properly heated and mixed should achieve 50 percent or more VSR, depending on the primary to secondary solids ratio. As noted in Table 1, the 80-foot diameter digester has only one draft tube with four lances while the two 85-foot diameter digesters have three draft tubes with six lances. Generally, an effective mixing rate results in at least sixteen to twenty four turnovers per day. Since information on the pumping rates for these draft tube gas mixers was not available, we cannot tell whether there is sufficient pumping capacity to provide a minimum of sixteen turnovers per day. It is likely that the content of the 80-foot diameter digester, which has only one draft tube mixer versus three for the other similarly-sized digesters, is not thoroughly mixed. Additional draft tube mixers are needed in this digester. A lithium tracer study can be performed to confirm the mixing efficiency for this and the other digesters. Since the digesters have not been cleaned for more than eight years and may have a mat of floating material at the top which can block the draft tube discharge. A physical inspection is needed to verify the impact of debris accumulation.

HARRF operators identified a problem with the ability of the current gas compressor system for Digesters 1 and 2 to provide balanced gas flow to the two digesters because of the varying number of lances in the two digesters. There is no means of automatically controlling gas flow to each digester mixing system; gas meters on each of the draft tube feed lines are not used to balance or control the system. A dedicated compressor for each of these two digesters is recommended to provide proper gas flow to each of the mixing systems.

Several alternative mixing technologies exist if the City intends to replace the existing gas mixing system with one that minimizes the safety hazards related to gas handling and provides a more efficient mixing system. A copy of a recent evaluation of mixing technologies being presented at the CWEA 2006 State conference is included for reference in Attachment B.

Digester Feed Sequencing

Unstable digester operation and poor volatile solids reduction can also be caused by improper influent feed. Digesters operate most efficiently when steadily fed with a homogenous mixture of feed solids. The existing digesters are fed primary and secondary solids separately every two hours. Primary sludge is fed to one digester while another is fed secondary solids; the third is idle. The feed sequence is rotated every two hours such that the digester previously fed primary solids is the idle digester, the digester previously fed secondary solids is fed primary solids, and the previously idle digester is fed secondary solids. BC recommends that each digester be fed both primary and secondary solids at least once per hour. Valve sequencing for each digester must be reprogrammed to direct sludge (primary and secondary) to the digester 24 times or more per day. The valve opening duration can be set by the operator such that the total daily sludge volume typically fed to the digester is divided into equal volumes fed at one hour or less intervals throughout the day.

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Centrifuge Dewatering

Recommended improvements to the centrifuge dewatering system at HARRF are discussed below.

Dewatered Cake Production and Solids Recovery

Review of process performance data for the dewatering system indicates the capture rate is lower than expected. Typical capture rates of 95% or better are expected from high solids centrifuges like those employed at HARRF. Centrate quality and solids recovery may be improved by optimizing the polymer dosage, strategically locating the polymer injection point, and possibly selecting a different type of polymer. It is possible that the sludge quality has changed since the centrifuges were first installed. The City should consider sending sludge samples to both the polymer supplier and Andritz to see if adjustments to the centrifuges or changes to the polymer formula or feed system are needed. Polymer trials may be useful in identifying a better polymer to increase solids capture.

The dewatered cake concentration is between 30 and 32 percent, which is well within acceptable performance for high solids centrifuges.

Centrate Quality And Scaling

HARRF operators noted that calcium scale is forming in the centrate collection system, requiring periodic cleaning. While aggressive mechanical cleaning and jetting can be an effective means of cleaning centrate lines, they can damage the inside of the pipe over time and form rough spots where scale can accumulate. Acid soaking have shown to be an effective method of removing scale as long as it is held in the pipe for an appropriate length of time.

Other commercial scale inhibitors can be added to the sludge being fed to the centrifuges to bind the calcium and magnesium or to prevent nucleation or agglomeration of the precipitates. Polyphosphate solutions (a popular scale inhibitor) have been known to cause foaming at turbulent points. The City of San Diego has tried several products with little success. However, they plan on continuing to test scale inhibitors from Protreat Technologies Corp, Shaners, and Polydyne (Flowsperse HT).

SUMMARY OF PROCESS OPTIMIZATION RECOMMENDATIONS

Process optimization recommendations have been described in detail above. Table 6 summarizes recommendations for the thickening, digestion and dewatering processes.

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 6. Recommended Solids System Process Performance Improvements			
Process	Item	Recommended Improvement	Purpose
DAF Thickening	Polymer feed	Move polymer feed closer to discharge point of pressurized flow	Improves mixing efficiency by using turbulence of rising bubbles
		Modify center feed piping to accommodate new polymer discharge point	Necessary to implement change noted above
	Thickener effluent	Replace thickener overflow weir with submerged launder pipe	Provides cleaner water for recycle to pressurized flow system
		Provide control valve on thickener effluent line for level control in the DAFT	Controls liquid level to maximize drainage through float
	Saturation System	Modify pressurized flow pumps, add common redundant pumps, or replace pumps to meet necessary recycle flow for solids loading	Provides sufficient flow for air saturation
		Add second pressurization tank and increase operating level	Provides sufficient residence time for air to dissolve
		Add continuous vent to purge excess nitrogen	Increases gas absorption and improves stability
		Modify inlet and outlet piping to prevent vortexing and inlet pipe flooding	Vortexing can bring in undissolved air to DAFT discharge point, disturbing small bubbles being released and break up floc as it forms with rising bubbles
	General	Consider modifying DAFTs to co-thicken primary and secondary solids	Provides homogeneous feed to digesters; reduces soluble BOD return to head of the plant
Anaerobic Digesters	Digester Feed Sequencing	Feed primary and secondary solids simultaneously to all digesters on the same day	Stabilizes operation through more consistent solids feed; prevents gas production spikes
	Digester mixing	Verify lances and draft tubes are clear	Ensures system is operating as designed
		Verify draft tube mixing capacity provides 16 to 24 turnovers per day	Verifies mixing capacity is sufficient to prevent solids deposition, surface matting, dead zones, and hot spots; provides efficient contact of existing biomass with new food
		Provide dedicated compressors for Digesters Nos. 1 and 2	Needed to provided balanced operation to draft tube gas mixing systems

Environmental Engineering And Consulting

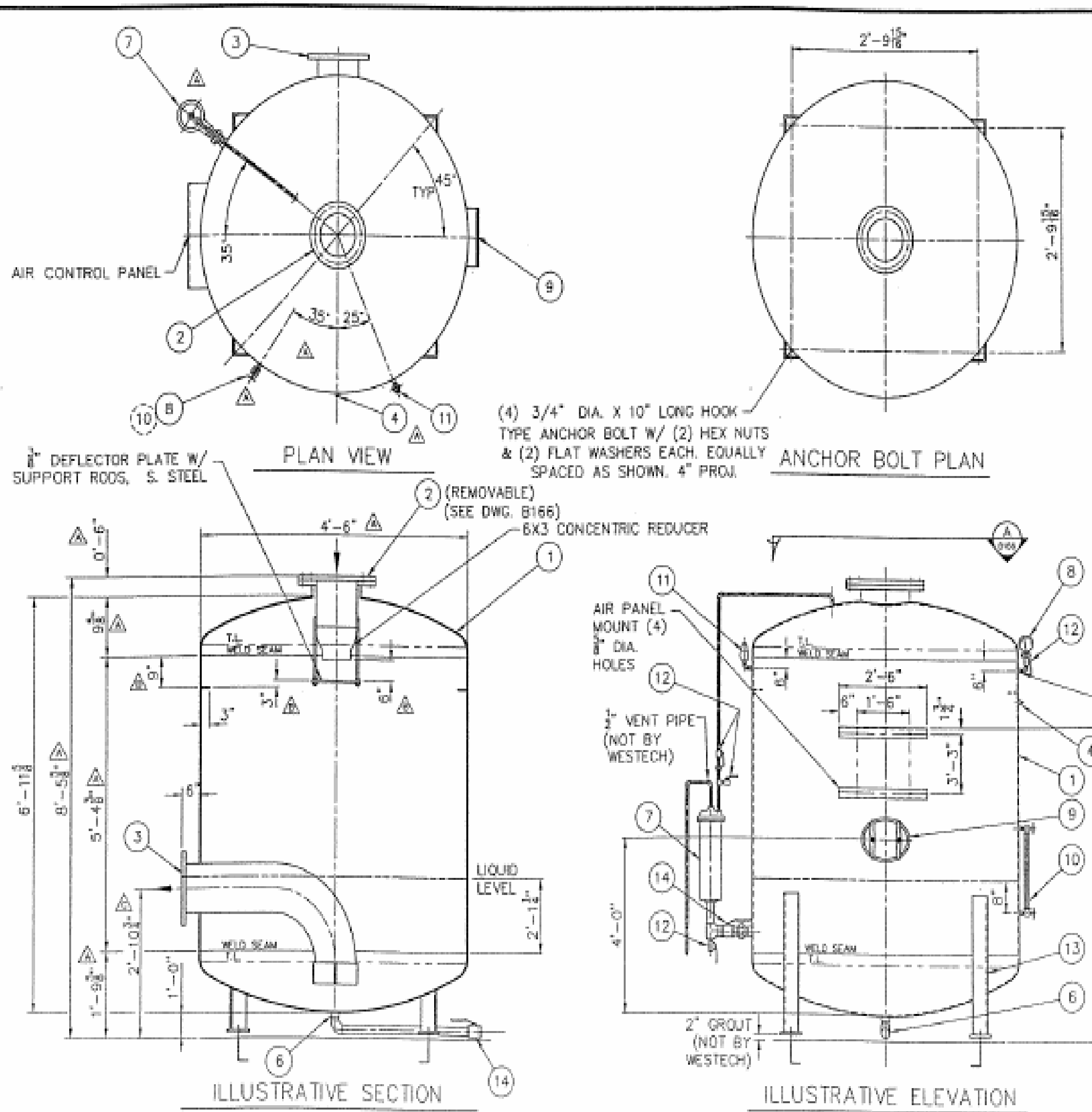
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 6. Recommended Solids System Process Performance Improvements			
		Perform dye study	Confirms mixing efficiency in digesters, particularly for Digester 1
Centrifuge Dewatering	Polymer application	Provide sludge samples to centrifuge and polymer suppliers	Verifies that the sludge character has not changed since centrifuges have been placed into service
		Perform polymer trials	Establishes whether a new polymer should be used
	Scale Control	Perform periodic acid cleaning of centrate pipes and/or use polyphosphate scale inhibitors	Maintains centrate system hydraulic capacity to prevent backups from occurring

Environmental Engineering And Consulting

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

ATTACHMENT A
VENDOR DRAWINGS



EQUIPMENT LIST

1. RETENTION TANK 4'-6" x 5'-4 3/8" SIDE SHELL ASME CODE, SECTION VIII, DIV. I, FOR 100 psig WORKING & HYDRO STATIC TEST PRESSURE @ 150 psig. MIN. 1/4" THK. WALL F&D HEAD w/1½" S.F.
2. 6" DIA. FLANGED INLET CONNECTION
3. 6" DIA. FLANGED OUTLET CONNECTION w/ VORTEX BREAKER.
4. 3/4" NPT AIR INLET CONNECTION
5. 1/2" NPT PRESSURE GAUGE CONNECTION
6. 2" NPT DRAIN LINE CONNECTION
7. LEVEL CONTROL W/AIR BLEED-OFF VALVE, ST. STEEL FLOAT & TRIM, C.I. CHAMBER (AIR INLET SOLENOID VALVE LOCATED IN AIR CONTROL PANEL)
8. PRESSURE GAUGE, 0-100 psig, 1/2" NPT, 4 1/2" DIA. FACE, ST. STEEL, SHATTERPROOF GLASS, LIQUID FILLED W/DIAPHRAGM SEAL
9. MANHOLE 12" X 16" CRAB TYPE
10. 24" SIGHT GLASS W/ISOLATION VALVES & DRAIN COCK (1/2" NPT)
11. PRESSURE RELIEF VALVE, 3/4" NPT, 80 psig SETTING ASME STAMPED
12. 1/2" BALL VALVE
13. SUPPORT LEGS
14. 2" BALL VALVE

NOTES:

1. ALL FLANGES TO BE 150 LB ASA S W/HOLES STRADDLING CENTER LINE
2. WORK THIS DWG. B100

PREPARED FOR: HALE AVE. W.W.R.P. ESCONDIDO, CA.
ENGINEER: MONTGOMERY WATSON
CONTRACTOR: MARGATE CONSTRUCTION, INC. 25007 SO. FIGUEROA ST. CARSON, CA.
PURCHASE ORDER NO. 113-178

RETENTION TANK GENERAL ARRANGEMENT
D.A.F. PRESSURIZATION SYSTEM-CLARIFIER 4'-6" X 5'-4 3/8"

WESTECH
D106
18156

ATTACHMENT B
EVALUATION OF MIXING TECHNOLOGIES
(paper being presented at CWEA 2006 State Conference)

Improving the Reliability of Goleta's Anaerobic Digestion System Through Evaluation of Digester Mixing Technologies

California Water Environment Association
2006 State Conference

Ken Fonda, PE
Brown and Caldwell
9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
858-571-6749
kfonda@brwncald.com

Jeff Salt, Plant Manager
Goleta Sanitary District
One William Moffett Place
Goleta, CA 93116

Abstract

The Goleta Sanitary District Wastewater Reclamation Plant (GSDWRP) anaerobic digestion facilities consists of three floating cover digesters equipped with an Envirex Pearth gas mixing system. Because of periodic interruption of the gas mixing system and scum accumulation at the top of the digesters, the GSDWRP plant management and operations staff became interested in evaluating alternative digester mixing systems to improve process reliability. Several conventional mechanical mixing technologies as well as an innovative vortex ring mixing technology were evaluated as alternatives to the gas mixing system. Evaluation of these technologies as well as studying the operation of the existing gas mixing system led to recommendations to improve reliability of the gas mixing system. Present worth comparison of these alternatives clearly showed the District the most cost effective solution to this problem.

Introduction

Proper digester mixing is essential to reliable anaerobic digester process operation. To improve digester operation, increase reliability and enhance the safety of the digester mixing system the Goleta Sanitary District retained Brown and Caldwell to evaluate its existing gas mixing system and recommend changes to the existing system or modifications to convert the digesters to mechanical or pump mixing. Effective mixing systems will provide the following benefits.

- Provide close contact between active biomass and incoming sludge
- Prevent stratification and temperature gradients
- Minimize formation of top scum layer

- Minimize bottom solids deposition
- Distribute food and buffering alkalinity to control pH

Objectives

The objectives of this project were to:

1. Evaluate converting the existing floating cover anaerobic digesters gas mixing system to a pump mixing system, and
2. Identify possible modifications to existing gas mixing system to improve reliability.

Existing Conditions

The existing anaerobic digestion facilities at the GSDWRP were constructed in the early 1960s and ‘70s and were upgraded in the late 1980’s. One of the upgrades to Digester No. 1 included pouring an inner wall up to the corbel level (19.25 ft high) to provide additional strength to the wall that was cracking. The existing anaerobic digestion system consists of three Brown and Caldwell design Downs floating cover digesters equipped with an Envirex Pearth gas mixing system. The covers are designed to operate submerged with the water surface up into the gas dome to reduce the surface area for scum formation. Currently, the three digesters are being operated in parallel with their liquid level at the maximum overflow height. The capacity of the digesters is adequate to provide a minimum of 20 days SRT at average daily flow with the largest digester, Digester No. 3, out of service; and 15 days solids retention time (SRT) at peak flow with all digester in service for projected future flows. Currently the digesters are less than fully loaded. Periodically bottom sludge is discharged to the downstream sludge lagoons, but normally the digesters just overflow to the lagoons as they are loaded. A description of current anaerobic digestion equipment is presented in Table 1.

Table 1. Existing Anaerobic Digestion Facilities

<table>
<tr><th>Item</th><th colspan="4">Value</th></tr>
<tr><td>Digester Number</td><td colspan="2">1[a]</td><td>2</td><td>3</td></tr>
<tr><td colspan="5">Anaerobic Digesters</td></tr>
<tr><td>Tank Diameter (ft)</td><td>43</td><td>45</td><td>45</td><td>55</td></tr>
<tr><td>Side water depth, (ft)[a, b]</td><td colspan="2">26.25[d]</td><td>26.25</td><td>31.25</td></tr>
<tr><td>Unit volume, (gallons)[c]</td><td colspan="2">311,362</td><td>331,266</td><td>592,935</td></tr>
<tr><td>Side access opening (L x W/Diam) (ft)</td><td colspan="2">2.5 x 2</td><td>2.5 x 2</td><td>3.5 Ø</td></tr>
<tr><td>Side access opening, (No.)</td><td colspan="2">1</td><td>1</td><td>2</td></tr>
<tr><td>Cover type</td><td colspan="4">Downs Floating</td></tr>
<tr><td>Mixing</td><td colspan="4">Envirex Pearth Gas Lance</td></tr>
<tr><td>Number of lances</td><td>6</td><td colspan="2">6</td><td>7</td></tr>
<tr><td>Gas Compressor type</td><td colspan="4">Rotary Lobe</td></tr>
</table>

Gas Compressor mfgr	Aertzen		
Gas Compressor capacity (scfm)	150	150	175
Gas Compressor pressure (psi)	9	9	9
Gas Compressor motor (hp)	15	15	15
Recirculation			
Number of pumps	1	1	1
Pump type	Recessed impeller centrifugal		
Pump Mfgr	Wemco		
Capacity (gpm)	500	500	500
Operating head (ft)	34	34	34

[a] Digester 1 – Smaller diameter shown for portion of tank where inner wall was added in 1987
[b] Sidewater depth based on maximum overflow level based on current operating mode
[c] Digester volume includes cone volume and tank volume to maximum overflow level
[d] Digesters sidewater depth 19.25 feet for the 43-foot diameter section, plus 7 feet for the 45-foot diameter section.

To identify operational issues a site visit was conducted with plant O&M staff. They indicated that the primary impetus for changing to a different type of mixing system was the dangers and operational difficulties associated with the use of digester gas including:

1. Safety issues related to gas handling;
2. The poor condition of the compressors due to their age;
3. Nuisance compressor auto shutdown due to low vacuum pressure. The compressor low suction pressure auto shut down occurs during normal startup on all digesters and periodically during normal operation of Digester 3. Sometimes it takes several start attempts before the compressors continue to operate;
4. Accumulation of hair and rags contained in the primary sludge. Operation’s feels that a hydraulic mixing system (using non-clog pumps with built in cutters) would minimize the build up of stringy material and other debris in the digesters; and
5. Inaccurate gas flow measurement relative to calculated digester gas production values.

Further discussion of operating issues with the existing gas mixing system is provided later in the Modify Existing Gas Mixing System alternative.

Evaluation of Mixing Alternatives

Overview Of Digester Mixing

Proper digester mixing benefits digester process operation by providing effective utilization of the digester volume. As new sludge is introduced into the digester, mixing brings the new food into contact with the active biomass. A healthy digester promotes higher levels of volatile solids destruction, reducing the volume of solids for disposal and increasing production of digester gas that can be burned as fuel to heat the digesters or produce electricity. A well mixed, uniformly heated digester provides an environment for the biomass to grow and thrive. Efficient digester mixing also keeps solids in suspension, preventing solids deposition that would reduce the working volume of the digester. Reducing solids deposition also lowers maintenance costs by increasing time between digester cleaning and decreasing the amount of solids to be removed. An added benefit is the reduced scum layer at the top of the digester.

There are several ways to determine whether adequate mixing is occurring within a digester. Sludge samples from different levels in the digester can be colleted to determine the uniformity of solids concentration and temperature. Typically, the temperature should not vary more than 2 °F and the concentration should not vary more than 10 percent. Tracer media can also be added to the digester to compare actual mixing with theoretical complete mix model outputs. Often tracer studies are a part of digester mixing performance specifications. BC typically specifies a pumping capacity that would provide a minimum of 16 physical turn-overs per day in pump mixing systems. Some emerging dynamic mixing technology systems have proposed designs based on kinetic energy gradients rather than physical turn-over rates. Initial operating data for these dynamic mixing systems appear to have equivalent levels of mixing as hydraulic pump mixing systems. Actual required kinetic energy gradients can vary from tank to tank depending on tank geometry and nozzle placement however, a typical value used by one manufacturer is 25 BHP/MG with an effective digester volume safety factor of 1.11 (see Attachment B for additional information).

Digester Mixing Alternatives

The purpose of this study was to evaluate alternatives to the current gas mixing system and possible modifications to the gas mixing system to improve its reliability, efficiency and safety. Because the digesters have floating covers that are still in good condition, the District has no intention on changing this type of cover in the near future. Therefore, mixing alternatives developed in this study must be compatible with floating covers. Mixing alternatives considered in this evaluation are as follows.

- External Pumped Circulation
- Dynamic Mixing
- Draft Tube Mixing
- Votex Ring Mixing
- Modify Existing Gas Mixing System

Vortex Ring Mixing is not technically considered a Pump Mixing system. However, it is included as an alternative method of mechanical mixing that warrants further consideration.

A brief description of these alternative systems and general advantages and disadvantages is presented in Table 2.

Table 2. Mixing Alternatives Considered for GSDWRP Digesters

Mixing System Description	Advantages	Disadvantages
External Pump Circulation Involves installation of large pumps and piping to provide physical turn-over rate of >16/day	• Simple, reliable, measurable pumping technology • Easily maintained, nothing inside digester to maintain other than piping • Low foaming potential	• Most applicable to smaller (<50 ft diameter) digesters • Moderate energy efficiency • Potential for dead spots (moderate mixing effectiveness) • Large pumps and piping require more space than gas system • Multiple wall penetrations • Must drain digester to install submerged mechanisms
Dynamic Mixing A variation of external pump circulation. Mixing energy is provided by specially designed and placed nozzles.	• Simple, reliable, pump mixing • May be adapted to larger (>50 ft) diameter digesters • Easily maintained, nothing inside digester to maintain other than piping • Low foaming potential • Rapid re-suspension of settle solids after shutdown • Smaller pumps and piping • Lower energy consumption than conventional pump circulation • Suitable for varying tank levels. • Natural vortex surface motion draws floating solids down to reduce matting.	• Mixing must be evaluated by tracer testing • Limited installations in the US. • External nozzle adjustment (Jet Mix™ system) • Must drain digester to install submerged mechanisms

Mixing System Description	Advantages	Disadvantages
Draft Tube Mixing Submerged impeller in a draft tube draws liquid in from the bottom or top and promotes a rolling action.	• High mixing effectiveness • High energy efficiency • Single manufacturer responsibility • Accessible equipment • Low foaming potential • Flexible operation (forward or reverse) • May qualify for energy conservation rebates from PG&E to offset construction costs	• Some models have experienced shaft seal and main bearing failures • Crane removal required • Careful vertical alignment required • Internal draft tube mixing with floating cover requires internal tube to have telescopic operation • Unable to see problems with mixing • Must drain digester to install submerged mechanisms
Vortex Ring Mixing An emerging technology that produces toroid waves by way of a vertically moving round plate. Vortex rings transfer mass through three mechanisms: carrying fluid from the generation site to the point of disintegration, by production of some mixing in the wake of the ring, and creating fluid convection within the tank	• External drive mechanism easy to maintain without taking digester out of service • Low foam potential • Single manufacturer responsibility	• Limited experience with applications in municipal market • Moderate energy usage • Unable to see problems with mixing • Added dynamic loading to roof covers from vertical thrusts • Un-synchronized operation of multiple vortex ring generators could unbalance the floating cover • Must drain digester to install submerged mechanisms • Large hatch(s) in roof needed to install and access submerged plates • Most applicable to fixed cover digesters • Potential for ragging

Modify Existing Gas Mixing Mixing energy is provided by the buoyant forces obtained from gas bubbles as they rise within the digester. Digester gas is recirculated and discharged through individual gas lances. The Pearth gas mixing system utilizes a rotor valve to select which lance is receiving gas at any one time.	• Lances can be pulled for maintenance • Lances can have individual purge systems • Flexibility to modify mixing pattern	• Potential for ragging • Mixing efficiency effected by depth of submergence • Unable to see problems • Potential for surface debris accumulation • Potential for foaming • Handling flammable gases

External Pumped Circulation
As stated above, the External Pump Circulation alternative would be sized to provide a minimum of 16 physical turn-overs per day. The pumping capacity required to meet this specified turn-over rate for each digester based on the digester volumes listed in Table 1 are as follows:

- Digester 1 = 3.500 gpm (7.7 cfs),
- Digester 2 = 3,700 gpm (8.2 cfs), and
- Digester 3 = 6,600 gpm (14.7 cfs).

The External Pump Circulation system would consist of piping, valves, and one or more chopper pumps to circulate the digested sludge through several suction and discharge points at a minimum of two locations around the digester perimeter. To avoid having to core drill through the existing walls, new suction and discharge piping could be brought through the existing side access hatches. All mixing piping and nozzles would be kept inside the digester. To keep the velocity in the pipe below 6 fps, the suction and discharge piping would need to be sized as follows:

- Digester 1 = 15 inches,
- Digester 2 = 15 inches, and
- Digester 3 = 24 inches.

As shown in Table 1, the access hatch size limits the size of the pipes and pipe reducer fittings needed to make it fit. Since Digester 3 has two access hatches

180° apart, two pumps could be used that would reduce the size of pumps and piping entering and exiting the digester.

Since the pumps would have suction and discharge points below the operating level of the digesters, there would be no static losses, only friction losses exist. Assuming a single mixing pump is used for each digester (two may be used for Digester 3), each pump would be sized as follows:

- Digester 1 & 2 – 3,700 gpm at 22 ft TDH, 30 hp
- Digester 3 (one-pump option) – 6,600 gpm at 22 ft TDH, 75 hp
- Digester 3 (two-pump option) - 3,300 gpm at 21 ft TDH, 30 hp

Dynamic Mixing
This alternative would be similar to the External Pump Circulation alternative, but the pumping capacities would be reduced to 1,500 gpm for Digesters 1 and 2 and 1,700 gpm for digester 3. Each digester would have two nozzle fittings located 180° apart mounted to the tank floor placed approximately 66 to 75% of the total diameter of the digester. Suction and discharge pipe diameters would be 10 inches. Each mixing system would have one dedicated chopper type pump with a 30 HP motor

Draft Tube Mixing
The Draft Tube Mixing alternative would involve installation of a single draft tube mixer in each of the existing digesters. As stated above in Table 2, draft tube mixers have primarily been used with fixed cover digesters, however, one of the vendors contacted indicated a telescoping draft tube could be fabricated to allow the draft tube to slide up and down with the floating cover. The new mixers would be mounted in the existing gas dome by replacing the top of the gas domes with new mixer mounting flanges. The existing pressure relief valve would be installed on the new mounting flanges. The sludge feed line currently discharging to the gas dome would be relocated through another part of the roof. Alternating the operating flow direction of the draft tube will prevent scum and other floating debris from accumulating at the top of the digester. Although the draft tube can be operated in an up or down mode, providing external grinding is considered beneficial to prevent clogging of the recirculation pumps and heat exchangers. The large impellers within the draft tube are designed to prevent ragging. Each draft tube mixer would be driven by a 5 HP motor. Since the draft tube mixers would use less energy than the existing gas mixing system, it would qualify for a rebate under the PG&E's Standard Performance Contract. The estimated rebate would be approximately $15,000.

Vortex Ring Mixing
The Vortex Ring Mixing alternative includes three motorized drive units mounted on the roof of the digester and connected to a vertical drive shafts with a specially designed plate attached to the end of the driving shaft. As the shaft cycles up and down, approximately 60 strokes per minute, toroid rings are

formed that move outward from the vortex ring generator. Several rings are formed on each stroke of the vortex ring generator. These rings create a rolling motion of the digester contents. The vendor is hesitant to predict what the actual average energy consumption would be for Goleta's digested sludge without more sludge data or possibly pilot testing. However, they believe that the initial energy draw would be higher than the average energy usage to get the mixing started. Once stable mixing is established, the kinematic viscosity of the digested sludge would be reduced resulting in lower mixing energy required. Pilot test data from a similar application indicated that the average energy consumption for three units mixing digester sludge with a concentration of 5% was 6 Hp.

Because this is still an emerging technology with limited applications in the wastewater industry, BC would not consider this for a full scale replacement of the existing gas mixing system without performing some pilot testing. The vendor has expressed an interest in providing pilot test equipment at no cost to the District to obtain more data. This would involve modifying the floating cover on one digester to receive the new vortex ring mixers. The vortex ring drive shafts could be installed through existing gas lance sleeves already in place. Additional hatches would need to be fabricated in the roof to install the vortex ring plates and provide access for future maintenance. Some modifications to the roof trusses may be needed to handle the additional dynamic loading from oscillation of the vortex ring generators. Redistribution of the roof ballast may also be necessary to keep the floating covers submerged.

Modify Existing Gas Mixing
Based on information obtained during the kickoff/brainstorming meeting further investigation of the existing gas mixing system was conducted. Preliminary findings indicated the following conditions may be causing the pressure fluctuation problems described by O&M staff:

1. The Perth Gas Lance mixing system uses a rotor valve for sequential lance operation. Each lance system rotor valve operator has at least one position (the small digesters had two positions) where no gas is being discharge through any of the lances.
2. The center lance is provided with a set of control valves that allow its flow to bypass the rotor valve. The center lance also has a piped port location on the rotor valve. Shop drawings show that these valves may be opened to allow some gas to flow to the center lance all the time.

This condition appears to cause the compressor's discharge pressure to spike and release high pressure relief timed discharge into the gas collection dome of the floating cover. This may subsequently cause the pressure relief valve on top of the gas dome to activate if it lasts very long. Purging digester gas from the pressure relief valve would cause odor problems and may be a reason for the digester gas meters showing less gas than should be generated by the digester loading. In addition, as the rotor valve moves to the next lance the gas suction pressure drops because digester gas isn't reaching the gas dome as quick as it is being withdrawn. This may be a contributing factor to the low suction pressure and the nuisance auto shutdown of the compressors.

A low cost, immediate solution to the pressure fluctuation problem may be to reposition the bypass valves on the center lance to allow some flow to always be fed to the center lance. This should eliminate the pressure fluctuations in the gas dome and the low pressure in the suction piping at the compressors should cease to cause compressor shut down.

The District could realize immediate improvement in the operation of the gas mixing system by making the following changes to pressure settings on the vacuum and pressure relief on top of the gas dome as well as the bypass system on the piping from the gas compressor:

- Set pressure relief at 15 inches water column (" W.C)
- Set vacuum relief at 0" W.C.
- Set compressor high pressure bypass to the gas dome at 12 pounds per square inch (psi)
- Set compressor low pressure bypass to the gas dome at 4" W.C.

With these settings, compressor flow will be bypassed back into the gas dome whenever the gas dome pressure at the rotor valve gets below 4" W.C. and whenever the compressor discharge pressure at the rotor valve goes above 12 psi. The low and high pressure switches at the gas compressor should be set at 0" W.C. pressure and 15 psi, respectively. These settings, along with the elimination of the compressor spikes should help make the entire gas mixing system more reliable and ensure gas system safety.

Quotes were obtained for both sliding vane and rotary lobe compressors. The sliding vane compressors were more than three times the cost of the rotary lobe compressors and the motor horse powers were the same. Because the sliding vane compressor would be operating at approximately the same speed as the rotary lobe compressor there would be no difference in the reliability or longevity of the compressor. Therefore, this alternative evaluation is based on the less expensive rotary lobe compressors since no energy savings would be expected. For estimating purposes, the cost of the rotary lobe compressor includes a sound proof enclosure or other sound proofing measures. Modifications that would be involved with this alternative would include:

- Replacement of the three compressors that are nearing the end of their useful life with new rotary lobe compressors with sound enclosures,

- Installation of an in-line grinder/cutter on the suction side of the existing recirculation pumps to reduce accumulation of debris, and

- Modification of the pressure settings and bypass pipe controls as stated above to stabilize compressor operation

Gas Metering Improvements

During the kickoff/brainstorming meeting, plant staff also indicated that the gas flow measurements appear to be off by a considerable amount. Accurate gas flow measurement is needed to calculate emissions from the flares and boilers as a part of the plant's air permit. If pressure spikes in the gas mixing system are causing the gas pressure relief valve on top of the gas dome to open, this may also be a reason for inaccurate gas measurements. The South Central Coast Air District requires flow measurements for both production and gas utilization. Review of the existing gas flow metering system has revealed some potential causes for these inaccuracies:

- Thermal dispersion flow meters require a minimum of 20 pipe diameters upstream of the flow element to avoid turbulence that can cause errors in flow measurement. The existing flow elements at each digester only have about 7 diameters of straight pipe upstream.
- Thermal dispersion flow elements installed on horizontal runs of pipe should be place in the 4 or 5 o'clock position to allow condensate to drip away from the thermal flow element. The existing flow element is installed at the 12 o'clock position that could be allowing condensate to collect at the tip of the element leading to inaccuracies
- Gas flow from one digester may be flowing back into another digester causing a negative flow reading from the flow meter.
- Possible discharge of digester gas during gas mixing startup and normal operation without the center lance set to operate continuously could result in unmeasured gas flow being vented to the atmosphere.

Regardless of whether the digester gas mixing system is retained, the thermal dispersion flow elements should be relocated to the vertical 6-inch lines coming down from the roof of the digester at least 10 feet below the fitting at the top of the tank. To make it more accessible, it could be located at a convenient level above the ground. The flow elements should be installed horizontally in the pipe with the thermal element located on the downstream side of the pipe. To prevent reversing the flow back into the digester from the other digesters check valves should be installed on the low pressure gas lines leaving the digester gas domes Continuous operation of the center gas mixing lance could prevent unmeasured digester gas flow from venting out of the pressure relief valve.

The estimated cost for relocating the gas flow meters and installing check valves on the low pressure gas lines from the gas domes should be less than $30,000, assuming plant staff performs the installation. The local flow meter manufacturer's representative should assist with overseeing the installation and calibration after the flow meters are relocated.

Estimated Costs

Estimated construction costs have been prepared for each of the alternatives described above. Equipment costs are based on preliminary budget pricing provided by vendors. Copies of vendor quotes are attached for more detailed information. Estimated construction costs for each alternative is presented in Table 3. Engineering costs have not been included in these estimates. Some additional engineering would be needed to implement these alternatives.

Table 3. Estimated Construction Costs of Digester Mixing Alternatives ($1,000)

	External Recirculation	Dynamic Mixing	Draft Tube Mixing	Vortex Ring Mixing	Modified Gas Mixing
Material & Labor	517	375	340	420	197
Contractor's General Conditions	52	38	34	42	20
Contractor's Markup	95	74	88	80	27
Taxes	33	20	21	26	13
Bonds & Insurance	70	51	48	62	27
Subtotal	767	558	531	630	284
Contingencies 30%	230	167	159	189	85
Total Construction	997	725	690	819	369

Assumptions:

Contractor's General Conditions – 10% of Material and Labor cost
Contractor's Markup = 18% Labor, 15% Materials, and 15% Equipment costs
Taxes = 7.75% of Material and Equipment
Bonds and Insurance costs = 10%
No energy rebates have been included in construction costs for the draft tube mixing alternative

A present worth (PW) comparison of each alternative is presented in Table 4.

Table 4. Present Worth Comparison of Digester Mixing Alternatives ($1,000s)

	External Recirculation	Dynamic Mixing	Draft Tube Mixing	Vortex Ring Mixing	Modified Gas Mixing
Construction cost	997	725	690	819	369
Annual Energy cost	157	58	29	39	48
O & M	24	24	32	24	15
PW Annual costs	$2,460	$1,114	$829	$856	$856
Total Present worth	$3,457	$1,840	$1,519	$1,736	$1,240

Assumptions:
Period = 20 years
Interest Rate = 4%
Energy cost = $0.10 per kWh

Conclusions and Recommendations

As shown in Table 4, the modified gas mixing system has the lowest present worth cost. If the pressure fluctuations can be mitigated by making the suggested adjustments to the operating pressure and valve settings on the bypass line to the center lance, the existing gas mixing system should continue to provide the necessary mixing for the digesters. Pending the results of these adjustments, Brown and Caldwell recommend that the GSD continue using the gas mixing system for mixing the digesters. If the suggested adjustments to the gas mixing system fail to produce a more reliable gas mixing system the draft tube mixing system is the next best alternative. Regardless of the type of mixing system that is chosen, the GSD can improve gas flow measurement by making the changes described above. The Vortex Ring mixing system appeared to be a promising technology, however, present worth costs are significantly higher than the two lowest alternatives.

As described above, recommended modifications to the existing gas mixing system include:

1. Replacement of the existing gas compressors with a new gas compressors with sound enclosures.
2. Installation of in-line grinders/cutters on the sludge recirculation systems to reduce accumulation of debris.

3. Relocation of the thermal dispersion flow elements to vertical gas piping coming down from the roof of each digester. Have the local flow meter representative assist with overseeing installation and calibration.
4. Installation of check valves on low pressure gas lines on the roof of each digester.

APPENDIX H

System Integration and Plant Optimization TM

TECHNICAL MEMORANDUM –FINAL (Revision 3)

DATE:	OCTOBER 26, 2006
TO:	ANGELA MORROW, CITY OF ESCONDIDO
FROM:	VICTOR OCCIANO, BROWN AND CALDWELL
PREPARED BY:	ERIC WAHLBERG, BROWN AND CALDWELL RION MERLO, BROWN AND CALDWELL RON APPLETON, BROWN AND CALDWELL KEN FONDA, BROWN AND CALDWELL VICTOR OCCIANO, BROWN AND CALDWELL SEVAL SEN, BROWN AND CALDWELL
SUBJECT:	CITY OF ESCONDIDO HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – SYSTEM INTEGRATION AND PLANT OPTIMIZATION

INTRODUCTION

Due to rapid growth in the Hale Avenue Resource Recovery Facility's (HARRF's) service area, the City of Escondido (City) contracted with Brown and Caldwell to evaluate the capacity of the existing treatment facilities at HARRF, the capacity of the Escondido Land Outfall, and the capacity of the San Elijo Ocean Outfall, through which the HARRF effluent and the effluent from the San Elijo Water Pollution Control Facility (SEWPCF) is discharged. The ocean outfall is owned and managed by the San Elijo Joint Powers Authority (SEJPA) who leases 79 percent of the current ocean outfall capacity rating of 25.5 million gallons per day (mgd) to the City. In addition to determining the capacity of these three treatment and flow elements, Brown and Caldwell also was tasked with determining the treatment technologies that would maximize the capacity of the existing HARRF site while meeting potential reclaimed water demands and probable discharge requirements. This is the main topic of this Technical Memorandum (TM).

A series of TMs was submitted sequentially as the evaluation of each process train at HARRF was completed. Presented in this TM is a summary of the findings contained in the separate TMs as well as an initial discussion on projected flows and disposal options. A Project Report is currently being prepared and will be submitted to the City in the future that combines the contents of this TM and discussions on the land and ocean outfall capacity evaluations. The Project Report will include

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

estimated costs of the recommended improvement and will therefore provide the City with the overall summary of the Wastewater Treatment and Disposal Facilities Capacity Study.

PROJECTED FLOWS

Sewer flows collected and transported by a sanitary collection system are comprised of both dry weather and wet weather flows. Dry weather flows are generated primarily of residential, commercial, and industrial land uses. The land use impacts the magnitude of flow, and the daily and seasonal patterns. In addition to land-use based flows, dry weather flows usually are comprised of ground-water infiltration flows generated from a variety of man-made and natural sources. These flows enter the collection system via pipe cracks, fissures, illegal connections, and private laterals.

Wet weather flows are generated by rain-related inflow and infiltration flows entering the collection system. The magnitude and timing of these flows typically creates a "worst-case" peak flow scenario impacting both the collection system and the treatment facilities. The magnitude of these flows is dependent on the structural condition of the collection system. For example, an old system with significant cracks and fissures will create high wet weather peak flows, whereas a new system will generate significantly lower wet weather flows. Therefore, the system age, current condition and future rehabilitation projects will all impact future wet weather flows.

The method deployed to estimate the projected average annual flows at HARRF involved the following steps:

1. Identify and classify land development projects (planned, under-construction, or complete) from 2006 through to 2010.
2. For each development project, identify the land-use type, building size, dwelling units, and appropriate unit flow factors.
3. Calculate the average daily flow generated from each development project and totalize for each future year.
4. Develop cumulative annual average flows from 2006 through to 2010 by adding the future flows to the existing 2005 flow at HARRF.
5. Using historical and future flows through 2008, plot a linear relationship between flow and year. Note, future flows from 2009 and 2010 were not used as limited knowledge of planned developments skewed the projection.
6. Compare projected flows with historical sewer connection trends to verify analysis.
7. Estimate time period (year) when the projected average daily flow will reach the current rated plant capacity of 18.0 mgd and the estimated build-out average daily flow of 27.5 mgd.

The following assumptions were used during the flow projection analysis:

- No increase in ground water and wet weather inflows resulting from a "trade-off" between increased development and collection system rehabilitation improvements.
- Future annual average wet weather and ground water flows remain constant and equal to the 2005 wet weather flows.

- Sewer discharge per capita flow rates remain constant through to build-out.
- Sewer discharge unit flow rates:
  - Residential: 250 gallons per day (gpd)/dwelling unit
  - Commercial: 1500 gpd/acre
  - Industrial (Light) 2000 gpd/acre
  - Industrial (Heavy) 5000 gpd/acre
  - Hospital: 210 gpd/bed
  - Restaurant: 30 gpd/seat
- Future land development is not limited by available developable land. This assumption is countered by future developments occurring through densification.

Land Developments

The land development projects (proposed, in-design and under construction) were summarized by the City and presented to Brown and Caldwell for analysis. Additional information describing the land use, acreage, dwelling units and estimated year of completion were obtained from further discussions with the City, specific plan documents and the City's Planning Commission meeting minutes obtained via the City's public web site.

The development projects, as summarized in Table 1, were allocated into appropriate years of completion ranging from 2007 through to 2010. The projects collated and presented in this analysis are expected to change due to economic, environmental and political issues. In addition, projects listed in 2009 and 2010 are considered under-estimates and will most likely increase as the City continues to grow.

The following assumptions were used during the analysis of the development projects:

- North County Transit District cleaning facility assumed to use "significant" water usage, hence 5,000 gpd / acre unit flow rate.
- Escondido Research Technology Center (ERTC) hospital (Phase 1) completed in 2008
- ERTC hospital (Phase 2) completed in 2010
- ERTC Stone Brewery expansion on-line in 2008 (additional 40,000 gpd)
- ERTC vacant lots sold, built and occupied by 2007

Table 1. Land Development Flows

ID	Year	Residential	Commercial	Industrial	Total Flow (mgd)
1	2007	26	8	3	0.431
2	2008	18	5	3	0.158
3	2009	27	6	1	0.283
4	2010	2	0	2	0.042

Flow Projections

The basis of this analysis is to estimate the times when the average annual daily flows at HARRF reach the current rated capacity of 18.0 mgd and the build-out flow of 27.5 mgd. The projected flows were derived by linearly extrapolating both historical flows (from 2000 to 2005) and estimated "development" flows (from 2006 to 2008). Note, the flows calculated for 2009 and 2010 were eliminated from the analysis as these under-estimated flows skewed the projection, delaying the years at which 18.0 and 27.5-mgd capacities are reached.

Figure 1 depicts the projected flow relationship along with the estimated years when 18.0 and 27.5 mgd flows are reached. The chart also displays a projected sewer connection trend-line extrapolated from new sewer connections added from 2000 to 2005. Although the projected sewer connection trend is "flatter", the overall trend is comparable with the flow projection trend-line.

The average annual flow at HARRF is projected to increase on average by 0.352 mgd per year resulting in the following events:

- Current rated capacity of 18.0 mgd will be reached in 2014.
- Master plan build-out (ultimate) flow of 27.5 mgd will be reached in 2041.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

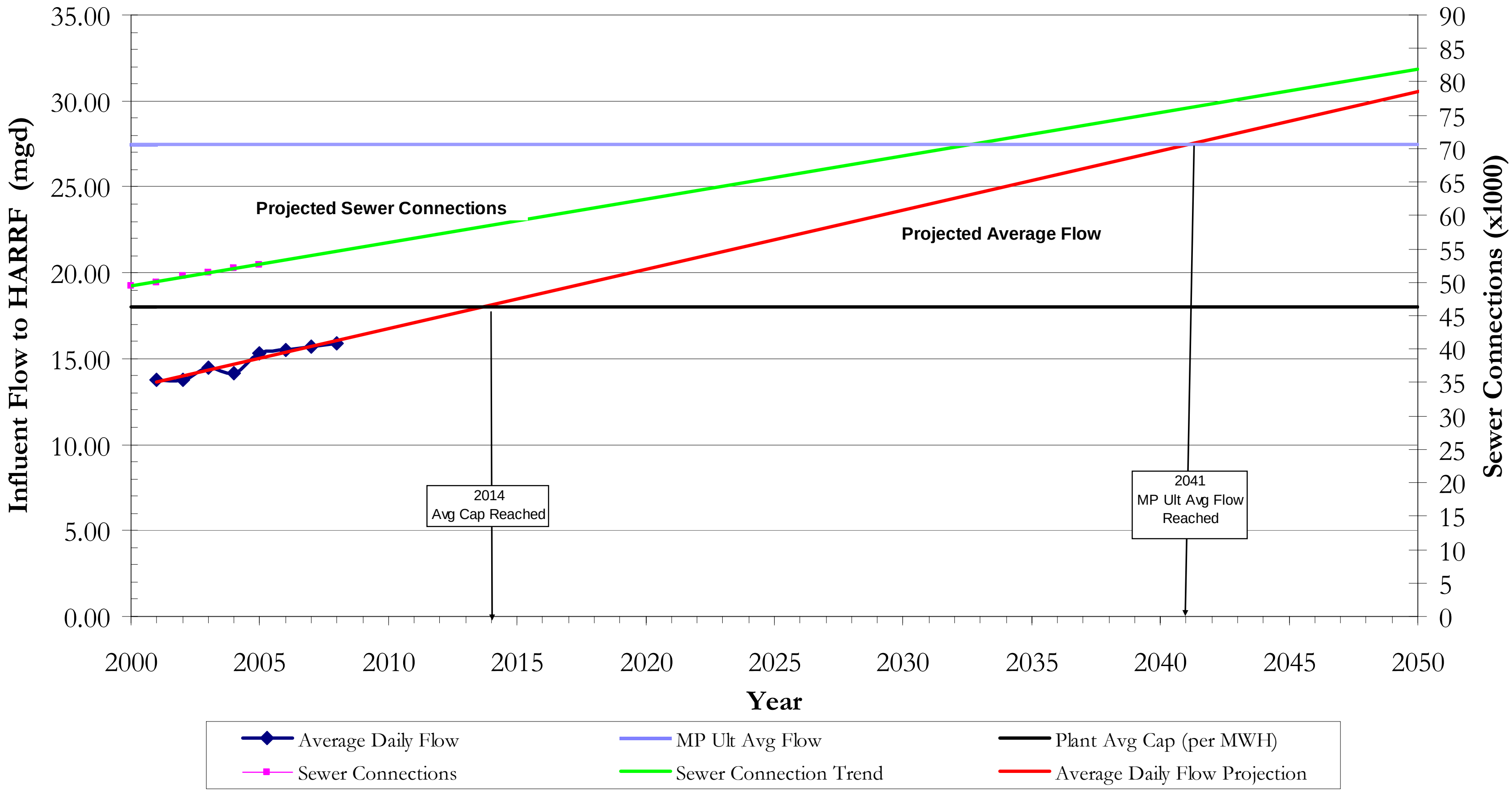


Figure 1. Influent Flow Projection Graph

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

DISPOSAL OPTIONS

The main focus of this TM is to discuss improvements needed to the HARRF process train to ensure that it can treat incoming wastewater flows at its current rated capacity of 18.0 mgd and future buildout capacity of 27.5 mgd. Upon implementing the recommended changes contained herein, disposal of the treated effluent becomes the next critical consideration. The disposal option ultimately chosen will significantly impact the selection of the appropriate process to implement at HARRF.

Briefly described below are options available to the City. A more detailed discussion of disposal options will be presented in the Wastewater Treatment and Disposal Capacity Evaluation Project Report to be prepared separately.

Land and Ocean Outfall Disposal

A majority of the treated wastewater from HARRF is disposed through the land and ocean outfall. The Escondido Land Outfall (ELO) stretches 14.3 miles from the HARRF fence line to the San Elijo Ocean Outfall (SEOO) Regulator Structure and is comprised of a series of 30-, 33-, and 36-inch diameter pipelines. Treated effluent within the upper nine miles of the ELO flows by gravity; it flows under pressure for the remainder of the way. The SEOO consists of two main segments: the land segment, which extends from the property line of the SEWRF to the Cardiff State Beach, and the ocean segment, which extends approximately 8,000 feet off shore. The SEOO was constructed in two phases. Phase I was completed in 1965, consisting of a 4,000-foot long, 30-inch diameter reinforced concrete pipe (RCP) that terminated at a depth of approximately 55 feet. A 192-foot long, 30-inch diameter pipe was added at a southward right angle bend from the 4,000-foot main line. This pipe and the final 120-foot section of the main line included diffusers that provided a minimum initial dilution of 120 to 1. The Phase I outfall system was rated at 15 mgd and most notably, the main line was designed for an internal pressure of 50 feet. [*Hale Avenue Resource Recovery Facility Phase II Treatment Process Upgrades and Enhancements Facility Plan*, March 1999]

In the following years, diverting the HARRF discharge from the Escondido Creek to the ocean outfall required increasing the capacity of the ocean outfall. Phase II of the outfall construction was completed in 1974. The outfall modification extended the terminus another 4,000-ft towards the ocean, consisted of 48-inch double rubber-gasketed RCP, and terminated at a depth of 148 feet below mean sea level. New diffusers were installed within the final 1,200-ft segment of the new extension while the old diffusers along the 1965 outfall were capped. A total of 200 diffusers exist, providing 237 to 1 initial dilution of the discharged effluent.

The current NPDES permit (CA0107981, Order No. R9-2005-0101) limits the monthly average HARRF effluent discharge flow rate to 18.0 mgd. However, the permit allows the City to discharge as much as 20.1 mgd, or 79 percent of the current rated capacity of the ocean outfall of 25.5 mgd, at peak conditions. The percentage is a contractual agreement with the City and manager of the outfall, the SEJPA, who share the use of the ocean outfall. The remaining portion, 21 percent or 5.4 mgd, is reserved for SEJPA to allow discharge of treated wastewater from the SEWPCF. The outfall rating is mainly based on the most sensitive segment of the outfall alignment - the nearshore pipes which are rated to withstand up to 50 feet of internal pressure.

Results of the latest capacity assessment were reported in the Brown and Caldwell Technical Memorandum entitled *Land Outfall Capacity Analysis*. It was determined that the land outfall has a hydraulic capacity of 23.7 mgd and the ocean outfall has a capacity of 26.8 mgd. The land outfall capacity is primarily limited by 'throttle' pipes restricting the flow resulting in localized spills in the vicinity of Manhole 73. Further analysis and model tests of the land outfall demonstrate the capacity could be increased to 25.2 MGD if the siphon inlet/outlet manholes are sealed. Regarding the pressurized section of the land outfall (upstream of the Regulator Structure) capacity improvements and spill reductions can be achieved by sealing all manholes downstream of Manhole 69 while ensuring the pressure head does not exceed 100 psi.

The ocean outfall hydraulic capacity is limited by the 50-feet pressure rating of the 30-inch RCPP pipe under the shoreline. Minimal improvements may be realized by modifying the operational logic of the regulator valve to accounte for variable tide levels. Significant capacity gain will only be obtained by constructing a new ocean outfall within the constraints of the ocean discharge permit.

Equalization Options

An alternative to expanding the capacity of the land and ocean outfalls is to equalize the effluent flow before discharging into the land outfall (equalization of primary effluent is another option, and is discussed in a subsequent section). Equalization reduces peak flows by 'shaving' and storing the flows in large detention basins/tanks. This study is examining the effect of peak flow reduction, hence reducing outfall capacity needs, by storing secondary effluent and recycled water in the existing equalization basin and recycled water storage tanks (existing and under construction). In addition, future equalization storage capacities will be evaluated as an alternative to expanding the existing land and ocean outfalls. Although equalization initially appears to be an attractive alternative to constructing a new outfall, the following constraints will determine the viability of using equalization:

- Available land for constructing potentially 'large' equalization basins
- Detention time (a significant issue if equalizing effluent flows)
- Operational issues and associated costs

The storm flow analysis described in the Brown and Caldwell Technical Memorandum entitled *Storm Flow Modeling*, details the relationship between effluent storage and storm frequency. The analysis assumed the effluent flow discharged from HARRF is limited by the existing outfall capacity of 23.7 mgd. The analysis can be used to estimate storage needs for "desired" storm events. For example, a 10-year storm event (i.e. an event occurring on average once every 10 years) was estimated to require approximately 12 million gallons of storage to equalize the outfall flow. The completion of additional secondary effluent and recycled water storage tanks along with the existing storage facilities provides 7 million gallons of storage; hence an additional 5 million gallons would be required to meet equalization requirements for the 10-year design storm event.

The City must be aware that selecting a specific storm event and return frequency as a basis for designing treatment and conveyance facilities has certain risks associated with the choice. For example, the choice to design and implement a system capable of handling a 10-year recurring event will have associated with it a 10 percent chance that a spill will occur. Designing on the basis of a 20-yr event will have a 5 percent risk, and so on. However, designing facilities based on "zero spills" also may be fiscally irresponsible, i.e., providing a system that can treat, store, and/or dispose of raw/treated wastewater to avoid any spills will require exorbitantly large structures and pipes that will cost several millions or billions of dollars.

Intermittent and Continuous Live Stream Disposal

On December 10, 2003, the SDRWQCB adopted Order No. R9-2003-0394, NPDES Permit No. CA0108944, allowing the City to discharge up to 9 mgd of tertiary treated effluent provided that ALL of the following conditions are met [Section A.1.3 of the Order]:

1. *The discharge to the San Elijo Ocean Outfall from the HARRF and the San Elijo Water Pollution Control Facility exceeds the maximum capacity of the outfall.*

2. *All emergency in-plant storage has been used.*

3. *Stream flows recorded at the County of San Diego's stream gauging station located approximately 100 yards upstream of the HARRF, exceed an average flow of 300 cubic feet per second during the discharge and are not below 100 cubic feet per second at any time during the discharge.*

4. *The mouth of the San Elijo Lagoon is open or the Regional Board Executive Officer approves otherwise.*

5. *The discharge occurs between November 1 and April 30.*

This important Order effectively reduces the amount of equalization required at the plant. Future equalization needs must consider revising the waste discharge requirements in the permit to enable increasing the quantity of discharge to Escondido Creek, particularly if there is very little land available for an equalization basin at or near HARRF. However, the cost of expanding the existing tertiary process units and conveyance facilities related to the increased disposal to the creek must be compared to the cost of providing equalization capacity. The most cost effective option should be selected.

Another option is to continuously discharge to the Escondido Creek an amount that exceeds the outfall capacity. Effluent standards are likely to be the same as those prescribed for the intermittent live stream discharge (i.e., Order No. R9-2003-0394). The most notable effluent standards are those for phosphorus and nitrogen which reads (for the Escondido Creek Hydrologic Subarea – HAS's 904.61 and 904.62):

> *"Concentrations of nitrogen and phosphorus, by themselves or in combination with other nutrients, shall be maintained at levels below those that stimulate algae and emergent plant growth. Threshold total Phosphorus (P) concentration shall not exceed 0.05 mg/L in any stream at the point where it enters any standing body of*

water, nor 0.025 mg/L in any standing body of water. A desired goal in order to prevent nuisances in streams and other flowing waters appears to be 0.1 mg/L total P. These values are not to be exceeded more than 10 percent of the time unless studies of the specific water body in question clearly show that water quality objective changes are permissible and changes are approved by the Regional Board. Analogous values have not been set for nitrogen compounds; however, natural ratios of nitrogen to phosphorus are to be determined by surveillance and monitoring and upheld. If data are lacking, a ratio of N:P = 10:1 shall be used."

This standard essentially limits total P to 0.1 mg/L and total nitrogen to 1.0 mg/L. It is very restrictive, ultimately limiting the selection of the appropriate process to implement at HARRF to nutrient removal processes. Biological methods are limited to certain effluent concentrations which are above the noted criteria. The standard will have to be met by biological treatment combined with physical/chemical treatment, likely requiring treatment by reverse osmosis (RO). Pursuing this option will need extensive work, including process and environmental impact evaluations. Furthermore, a pilot test will be needed to determine the effectiveness of certain treatment processes in achieving the discharge criteria at local conditions. Finally, the practice of continuous live stream discharge will be somewhat a pioneering endeavor for the San Diego region and, consequently, will be a challenge for the City to implement.

Water Recycling

This method of disposal provides a means to decrease the annual mass loading of permitted pollutants (e.g., TSS, BOD, etc.) to the ocean by directing it to uses on land, typically irrigation. Thus, it may increase the allowable volumetric discharge rate to the ocean (requires further discussion with SDRWQCB). However, the SDRWQCB requires the City to have a means for emergency disposal of the treated effluent during wet weather periods when irrigation demand is low or non-existent. Recycled water use, therefore, does not offer relief during wet weather periods unless reuse occurs year around.

The program to recycle the treated wastewater from HARRF began in 1991. Demands identified early in the program include industrial and irrigation uses within the City and in the Rincon Del Diablo Municipal Water District (Rincon). The program was divided into three phases. Phase I was estimated to involve a total of 3,400 acre-feet per year (afy) or 2.6 mgd average annual use while Phase II consisted of 900 afy of demand, increasing the average annual demand to 3.3 mgd. The ultimate reuse system will reportedly provide more than 4,500 acre-feet per year (afy) or 4.0 mgd average and 7.9 mgd peak day demand (*Hale Avenue Resource Recovery Facility Phase II Treatment Process Upgrades and Enhancements Facility Plan*, March 1999).

In 1993, the SDRWQCB adopted Order No. 93-70 which allowed the City to discharge to reclamation 3 mgd average annual and 5 mgd peak day flow of wastewater treated to Title 22 standards. Uses identified included irrigation of golf courses, parks, street landscape, schools, agriculture, and other landscape areas which previously used potable water for irrigation. In 1999, Order No. 93-70 was renewed, increasing the allowable peak reuse rate to 9.0 mgd. In the same renewed Order, it was identified that ultraviolet (UV) light would replace chlorination for disinfection of the tertiary-treated wastewater. The specified minimum UV dose required under worst operating conditions was 140 milliwatt seconds per square centimeter (mW-s/cm^2). The revised Order also indicated that coagulation was not required as long as the filter effluent turbidity did not exceed 2 nephlometric turbidity units (NTU) and a coagulation system can be automatically activated if the influent turbidity exceeded 5 NTU.

Recent demands for the HARRF recycled water has a significant impact on the effluent disposal. The Palomar Energy Project (PEP), a 550-megawatt power plant constructed by Palomar Energy LLC, an entity of Sempra Energy, intends to use 3.0 to 5.3 mgd of recycled water for cooling purposes. The reuse water quantity depends on the number of power generating engines being operated, which is governed by the power demands of the area served. During the cooling process, water is lost to the environment through evaporation - estimated to be up to 2.7 mgd. At full power production, this will conceivably increase the average monthly allowable discharge rate to 20.7 mgd: 18.0 mgd to the ocean outfall and 2.7 mgd through evaporation. [*Order No. R9-2005-0139 Fact Sheet*]

The consequence of the cooling process is the concentration of dissolved solids in the process stream, creating a brine solution that will be returned to HARRF through a dedicated pipeline called the Industrial Brine Collection System (IBCS) for dechlorination and mixing with the HARRF effluent prior to final disposal through the ELO and SEOO. It is expected that an average of 1.0 mgd and a maximum of 1.4 mgd of brine from the PEP will be returned to HARRF. Minor amounts of brine discharges from Boncor, Culligan and Goal Line L.P. also will be returned along the IBCS for a total brine discharge of 1.5 mgd to the outfall.

Groundwater Recharge at Various Basins

A limited amount of information was available to the project team related to this disposal method. Information summarized below was extracted from the March 1999 *Hale Avenue Resource Recovery Facility Phase II Treatment Process Upgrades and Enhancements Facility Plan*, and conversations with City of San Diego staff.

The City evaluated the possibility of recharging the following three groundwater basins with tertiary effluent from HARRF:

- Escondido Basin
- San Dieguito Basin
- San Pasqual Basin

The Escondido Basin was found to be too small and was not suited for cost-effective recharge. The San Dieguito Basin was too far from City facilities; it was discovered that it was more cost-effective for agencies closer to the basin to conduct the recharge operation. A majority of the San Pasqual basins is occupied by an agricultural preserve owned by the City of San Diego. San Diego staff indicated that recharge of tertiary effluent to the San Pasqual Basin was abandoned after receiving significant and very vocal opposition from farmers, citizens and politicians in the area. It appears that groundwater recharge may not be a feasible alternative. In addition, depending on the groundwater management that occurs within the basin, it may not offer the year-around disposal opportunity needed to offset discharge through the land and ocean outfall.

PROCESS ALTERNATIVES FOR HARRF

Influent Pump Station

The hydraulic capacity of the influent pumping station (IPS) was evaluated briefly for this study. The existing IPS consists of a bifurcated wet well with three single-stage vertical centrifugal pumps installed in each wet well and a 24-inch x 36-inch cast iron sluice gate separating the two halves. Each pump installed in one wet well has a matching pump in the other wet well. This arrangement and the ability to isolate the wet wells with the gate were envisioned to allow cleaning of one wet well while the other remained active. However, HARRF staff have indicated that the gate is not operable and the wet wells cannot be isolated.

The pumping station was designed with six pump bays, but only four pumps were installed initially – two in each wet well. In the 1980s, the largest pair of pumps was added. The pumps, drives, and motors were not changed, but some impellers were replaced with larger diameter impellers. The current IPS design criteria are summarized in Table 2 and the pump arrangement is shown on Figure 2 [source: HARRF-staff supplied information on May 31, 2006 and the 1981 *Hale Avenue WWTP Expansion* drawings (Sheet G-4)].

Table 2
HARRF Influent Pumping Station Design Criteria

Item	Unit	Pair 1	Pair 2		Pair 3
Pump number	-	1 & 6	2 & 5		3 & 4
Manufacturer	-	Fairbanks Morse	Fairbanks Morse		Allis-Chalmers
Type	-	Vertical centrifugal angleflow	Vertical centrifugal angleflow		Vertical centrifugal mixed flow
Drive	-	Variable	Constant		Variable
Capacity, each	gpm	5,060	4,600		9,000
Total dynamic head (TDH)	feet	30	30		40
Maximum speed	RPM	855	875		880
Motor horsepower	hp	50	50		125
Impeller diameter	inch	15.40	14.9375		17.75
Operating Strategy			Lead	Lag	
▪ Pump on (water depth)	inch	60	94	102	70
▪ Pump off (water depth)	inch	41	60	84	52

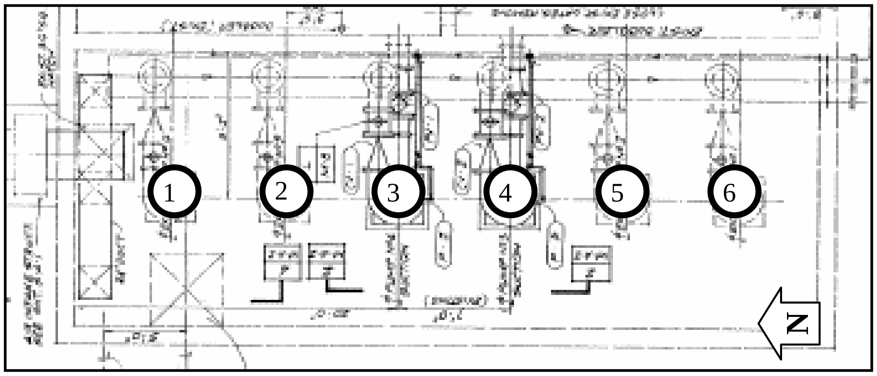


Figure 2
HARRF Influent Pumping Station Arrangement

The 2004 Capacity Rerating Study letter report stated that the IPS was rated for a peak capacity of 43.5 mgd with one pump out of service. Pump performance curves were not available for the smaller pumps, so the pumping station capacity could not be evaluated in detail for this study (nor was a detailed evaluation in the scope of this study). We recommend that a copy of the pump performance curves be obtained and that a system-head curve be developed for a detailed analysis of pumping station capacity. Based on the available information, the following observations and recommendations are provided:

- The IPS discharge force main is a combination of 30-inch and 36-inch diameter pipes, transitioning from 30-inch to 36-inch approximately 280 feet downstream (Sta 3+59.35 per Sheet C-14 of Phase 2 Drawings) of the IPS.

- The ultimate (buildout) peak wet weather flow rate to the IPS is 48.2 mgd, consisting of 44.4 mgd raw sewage and 3.8 mgd of in-plant recycle flows. At the buildout flow rate, the velocities in the 30-inch and 36-inch pipes are approximately 15 feet per second (fps) and 10.6 fps, respectively. To avoid significant erosion of the pipe walls and excessive frictional energy loss, prudent design practice limits the velocity to between 8 and 10 fps. Accordingly, the 30-inch pipe should be replaced with a larger pipe to reduce the maximum velocity. Based on the 2004 Capacity Study, the larger pipe size will reduce the total dynamic head (TDH) at the pump discharge and the existing motors, drives, and pump impellers should be adequate. However, a new system-head curve should be developed as part of a detailed pumping station analysis to confirm (1) the revised capacity of the existing pumping station with the larger pipe, and (2) if the existing motors and drives are adequate for continued service.

- The 9,000-gpm pumps (Pump Nos. 3 and 4) could be operated at 10-12 percent higher than the current design speed of 880 RPM to provide the additional capacity to handle buildout flows. A field torsiograph test should be conducted to identify torsional resonance issues and determine if the existing VFDs can operate at speeds greater than 60 Hz.

- A lateral resonance study should be conducted to determine if the pump foundation, frame and motor supports, and rotating system can withstand the dynamic forces resulting from operation at the higher speeds.

- The motor manufacturer should be contacted to determine if the motor design is adequate to handle the additional electrical current and voltage at the higher speed. Additionally, the VFD manufacturer must be consulted regarding the capacity of the existing drives and their ability to overspeed the system. Other checks of the electrical system will be needed to determine if there is sufficient capacity to carry the additional load.

- Any increase in motor size or overspeeding may require the upgrade of feeders to the IPS. The HARRF staff has reported the following:

  "*..the cabling to the influent pump stations MCC is single run (3 phase) of "500 MCM" type XHHW. The branch circuit breaker is set at 300 amps. It appears we would need to increase the size of the MCC feeder if we make a large change in the horsepower rating of any pumps.*

  The 125 hp pumps use a 125 KVA Toshiba 130-H2 drive which appears to [be] short of the 150 hp rating. At this time the drives are only eighty percent loaded."

- Given the age of the existing equipment, a comprehensive condition assessment of the equipment (e.g., gates, operators, valves) must be conducted to determine if any system components must be upgraded or replaced.

Preliminary Treatment

The existing preliminary treatment system consists of two 24-foot diameter Schloss forced vortex grit collectors (type CTP Grit Collector) with a 10-hp paddle mixer each, four Wemco horizontal recessed impeller grit pumps (two for each grit collectors) with 15-hp motors, two Schloss grit cyclone separators and classifiers, and two self-dump hoppers.

The reported capacity of the grit chambers differs according to the source. The 1999 Phase 2 Treatment Upgrades and Reclamation Facilities contract drawings indicate a peak flow and average flow capacity of 29.0 and 14.5 mgd, respectively, for each grit collector. The subsequent 2004 Capacity Rerating Study letter report rates each grit collector at 21.0 mgd average flow. The manufacturer recently stated the following capacity and performance information [*Telephone conversation between Brown and Caldwell and Schloss Engineered Equipment, Inc. on June 2, 2006*]:

- Peak capacity of one 24-ft diameter unit is 70 mgd peak flow.

- Grit collector is designed to provide the following particle removal efficiencies at the rated peak capacity, assuming a sand particle with a specific gravity of 2.65:

- 95 percent of 50 mesh size
- 85 percent of 70 mesh size
- 65percent of 100 mesh size

Based on past experience, Brown and Caldwell has found that the reliable capacity of this type of grit removal system is typically 50 percent of the manufacturer's stated capacity. Short periods of grit carryover will not have a significant impact on downstream unit processes, but continuous grit carryover during average flow conditions due to an undersized system can cause primary sludge pump wear, grit accumulation in aeration tanks, and/or grit accumulation in digesters. Although peak flow periods may have a short duration, they can represent a significant peak grit load due to "first flush" conditions. For future flow conditions, a third grit chamber may be necessary.

We find that the hoppers are typically undersized, causing bridging of grit particles if the pump is not sized to pump the maximum expected rate of inflow of grit at less than about 1 percent solids concentration. There are improvements to the typical design that can be implemented to ensure that clumping of grit particles do not occur. Many designers, including Brown and Caldwell, include an air scouring system to fluidize the grit particles prior to pumping. The current system does not have this provision, but includes water agitation of the grit hopper. It is our opinion that water is not as effective as air scour for this purpose.

An alternative to constructing a third grit chamber would be to allow the existing grit chamber to treat the incoming flows and allow any uncaptured grit to settle out in the primary clarifiers. This is a common practice in many plants that do not have a grit removal process. For a separate sludge thickening option, the raw sludge could be degritted before going to the digesters. Degritting the sludge prior to digestion provides added benefits to digester operation by reducing the amount of grit that would accumulate in the digester that reduces active digester volume and reducing the percent of inert solids that could cause digester upsets. Degritting of primary sludge will require a solids concentration of approximately 1 percent solids. Because of the lower solids concentration, separate thickening in this application is not feasible for HARRF because of digester limitations; feeding a lower concentration primary sludge will require more digester volume.

DAFT provides a convenient place for a second chance to remove the grit contained in the primary sludge when both primary and secondary solids are co-thickened in the DAFT. Co-thickening in the DAFTs is discussed in more detail later in this TM as an alternative mode of operation which should be considered to enhance the performance of the solids processing facilities. The DAFT process removes grit from the primary sludge in much the same way as an aerated grit chamber does. Air bubbles released as a part of the flotation process cling onto the particles with a lower specific gravity, allowing them to float to the surface. The more dense particles that do not float would settle out to the bottom of the DAFT where it could be removed as a part of the bottom sludge.

Typically Brown and Caldwell designs DAFT thickened bottom sludge pumping systems to recirculate from 4 to 10 percent of the raw sludge flow back to the influent feed to the DAFT. To keep from recirculating grit contained in primary sludge, this flow is generally passed through a sludge degritting system. Eutek makes a vortex grit removal system that can be utilized in the sludge degritting process. The City of San Diego at its Metro Biosolids Center (MBC) has been successfully using a Eutek Tea Cup™ degritting system for over seven years to remove grit from raw sludge prior to the thickening centrifuges.   Figure 3 shows the Eutek system being used at MBC The units are designed to operate best at a specific flow rate that produces the proper velocity in the fully enclosed vortex chamber. The size required for degritting the DAFT bottom sludge from the cothickening process would be need to be coordinated with the size of the bottom sludge pumps. Since the bottom sludge is removed on an intermittent basis there is some flexibility in adjusting the size of the bottom sludge pumps to the optimum flow rate for the Eutek Teacup degritting system.

Figure 3 – Eutek TEACUP degritting system at MBC

In summary, as currently configured the grit removal system does not have sufficient capacity for buildout conditions. The reliable capacity is likely between 14.5 and 21 mgd average flow as reported in the 2004 Capacity Rerating Study; additional testing and verification is needed. In addition, implementing an air scour system will likely ensure that the capacity is at the upper end of the range. Note that the cost estimates presented later in this report include a third grit chamber similar to the two existing units or a degritting unit for the thickened bottom sludge at the DAFT.

Primary Treatment

Currently, the HARRF has four primary clarifiers. For future conditions, it is recommended that an additional primary clarifier be constructed to provide redundancy; plant staff would like to have the flexibility to take one clarifier out of service during dry weather. However peak flows cannot be treated with five primary clarifiers unless modifications are made to remove the hydraulic bottleneck existing downstream of the primary effluent launders. The hydraulic capacity of the primary clarifiers can be increased with the following improvements:

- Increase the size of the influent well orifice of the secondary clarifiers
- Increase the number of gates at the aeration basin influent and effluent channels to a total of 8 gates per basin (currently, there are 4 gates per basin)

Using the primary clarifier model developed based on historic data and verified with on-site stress testing, the estimated TSS removal with the existing four clarifiers is 58 percent at future average daily flows. Constructing an additional primary clarifier will reduce the surface overflow rate (SOR) to 1,030 gpd/ft^2 and will result in an increased TSS removal of approximately 62 percent. The TSS removal resulting from the additional primary clarifier is considered to be minimal.

For some of the detailed options for secondary treatment listed below, chemically enhanced primary treatment (CEPT) was considered. The objective of CEPT is to reduce organic and solids loading to the secondary processes so that the mixed liquor suspended solids (MLSS) concentration is reduced, thereby lowering aeration requirements and increasing the capacity of the existing aeration basin tankage and secondary clarifiers. CEPT for HARRF would require chemical dosing with ferric chloride (doses ranging from 20 to 60 mg/L). Addition of a polymer will act to strengthen flocs and may provide better performance at doses ranging from 0.1 to 0.5 mg/L. Jar testing was performed on May 16, 2006, to determine TSS and COD removal with chemical addition and these results will be used to make predictions of performance. Figure 4 shows results of the jar testing. Chemical addition resulted in an increase in TSS removal from 55 percent to as high as 71 percent at 62 mg/L ferric chloride dose. A 60-mg/L dose of ferric chloride is considered relatively high, and a TSS removal of 68 percent observed at 40 mg/L ferric chloride dose was assumed representative of the future condition. From the bench-scale testing, it was concluded that CEPT could result in a relative increase in TSS removal of 25 percent. For four primary clarifiers, TSS removal is estimated to be 72 percent with CEPT and with five primary clarifiers the TSS removal is estimated to be 77 percent. The application of CEPT for HARRF will result in elevated primary sludge loading to the solids handling processes due to the higher removal efficiency and inorganic, chemical precipitates.

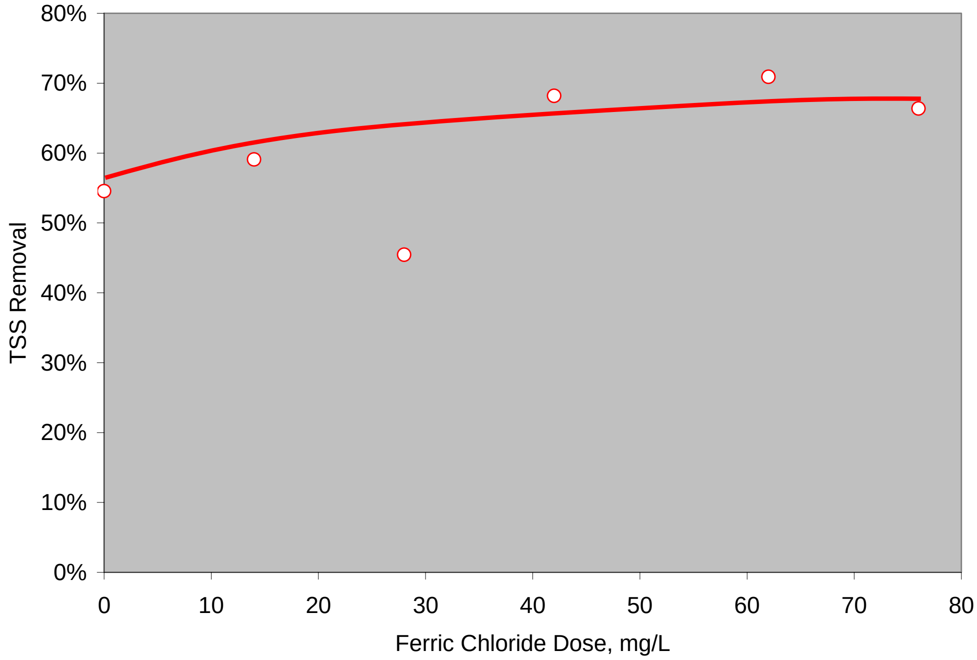


Figure 4. Results of CEPT jar tests using ferric chloride.

Secondary Treatment

The secondary treatment facilities (aeration basins, secondary clarifiers, and supporting systems) at HARRF were determined to be limited by the existing aeration system. The existing blowers are rated for an average dry weather flow (ADWF) of approximately 15.0 mgd (it is recommended that the existing fine-bubble aeration system be replaced to provide higher airflow rates). If the existing aeration system were able to meet the requirements of the influent loading, HARRF would be capable of treating approximately 14.8 mgd while operating at the current 2.75-day solids residence time (SRT) as detailed in Brown and Caldwell's *Biological Process Evaluation Technical Memorandum.*

In large part, the capacity of the existing secondary treatment facilities is a function of sludge settleability. The 90th percentile sludge volume index (SVI) value used to rate the secondary clarifiers was determined to be 203 mL/g based on historic data (2000-2005). In determining the future plant requirements, it was assumed that the 90th percentile SVI could be reduced to 125 mL/g with the addition of a biological selector in the aeration basins. Selectors (either anaerobic or anoxic) act to prevent over proliferation of filamentous organisms that impair sludge settleability and decrease capacity. In the absence of a biological selector, SVI can be controlled with chemicals (e.g., RAS chlorination or polymer addition). In addition, mixed liquor wasting was recommended to allow for better SRT control.

Secondary clarifier performance also is a function of the available return activated sludge (RAS) pumping capacity, where higher pumping may increase capacity. Currently, secondary clarifiers 3 and 4 have a RAS pumping capacity of 8.64 mgd each, and secondary clarifiers 1 and 2 have a RAS pumping capacity of 4.32 mgd. At the future sludge settleability condition (i.e., SVI of 125 mL/g), the clarifiers are not limited by RAS pumping capacity; capacity is limited by sludge settleability. In other words, the sludge can be pumped out of the clarifiers faster than it can settle; therefore additional RAS pumping capacity will not increase capacity and is not recommended. However, it is recommended that additional pumps be installed for redundancy (one pump to service secondary clarifiers 3 and 4 and one pump to service secondary clarifiers 1 and 2).

In the recent past, the tertiary treatment processes at HARRF required excessive levels of coagulant (approximately 60 to 80 mg/L polyaluminum chloride or PACl and approximately 3 mg/L polymer) to produce an effluent suitable for filtration and eventual use as reclaimed water. It has been determined that the poor performance of the tertiary processes corresponded to high nitrite concentrations in the secondary effluent caused by incomplete nitrification and low MLSS concentrations (less than 1.0 g/L) due to the low SRT. Upon Brown and Caldwell's recommendation, the plant increased the operating SRT from approximately 2.75 d to approximately 5.0 d. This process change resulted in lower turbidity levels in the filter effluent and reduced chemical requirements. The improvement in tertiary performance is attributed to better bioflocculation due to a longer SRT (coincident with nitrification) and due to higher MLSS concentrations. Typically, activated sludge settles as a blanket resulting in the removal of smaller particles due to a "filtering" action of the settling biomass. Operating at MLSS concentrations less than 1.0 g/L results in a diluted sludge where benefits of this "filtering" action are lost and effluent can have higher levels of colloidal material.

Based on the recent observations in the tertiary system, it was determined that: the future plant must produce an effluent suitable for tertiary filtration, the existing tertiary system must be optimized, or a new tertiary system must be installed capable of treating the existing secondary effluent. In addition, the activated sludge process must produce a non-bulking sludge (90^{th}-percentile value of 125 mL/g) with the addition of a selector or with chemical addition (polymer addition or RAS chlorination). Currently, there is room for the addition of one primary clarifier, one aeration basin, and two secondary clarifiers.

Tertiary Filtration

In addition to meeting final effluent discharge limits for ocean disposal, the goal of the primary and secondary treatment options discussed above is to produce a secondary effluent that is suitable for tertiary treatment with the existing HARRF facilities – granular media filters preceded by mechanical flocculation and followed by chlorine. The existing granular media filters were designed for a maximum influent flow of 10.0 mgd with one filter out of service, which corresponds to a maximum hydraulic loading rate of 5.0 gpm/ft^2 as allowed by California Department of Health Services (DHS) for recycled water treatment. A portion of the filter effluent is recycled continuously to the influent pump station as waste washwater. At the design waste washwater flow rate of approximately 80 gpm per filter, the total waste washwater flow is 0.8 mgd, which results in a net filtered water production of approximately 9.0 mgd. A higher waste washwater flow would reduce the net filtered water production. However, the filters have never been able to operate at their rated hydraulic loading of 5.0 gpm/ft^2 (9 mgd) and comply with the 2 NTU filter effluent limit for "disinfected tertiary" quality recycled water. A large portion of this filter performance limitation is due likely to poor secondary effluent quality– specifically high nitrite nitrogen concentrations that interfere with chemical coagulation.

Disinfection Process

The UV disinfection system was designed based on the 1993 NWRI guidelines (National Water Research Institute, *UV Disinfection Guidelines for Wastewater Reclamation in California and UV Disinfection Research Needs*, September 1993). These guidelines were superseded by the 2000 NWRI/AWWRF guidelines (National Water Research Institute/American Water Works Research Foundation, *Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse*, December 2000). The 2000 NWRI/AWWRF guidelines required that the capacity of the HARRF UV disinfection system be tested at full-scale operating conditions before DHS would validate the recycled water treatment facilities. A series of three commissioning tests were conducted in early 2003 to validate the delivered UV dose. DHS developed an interim operations plan in the fall of 2003 that stated a maximum UV disinfection system capacity of 4.0 mgd. The San Diego Regional Water Quality Control Board approved the 4.0 mgd disinfection capacity based on the DHS interim operations plan.

Because of the limited UV system capacity, the old “squircle” clarifiers were converted into a single multiple pass chlorine contact tank with a capacity of approximately 10 mgd. The chlorine contact tank is used currently for filtered effluent chlorination. The current chlorine disinfection operations costs are relatively high because the high nitrite nitrogen concentration exerts a significant chlorine demand (approximately 10 mg chlorine/mg nitrite nitrogen) that must be met before recycled water disinfection can be achieved. The costs determined for the future alternatives using chlorination should include supplemental ammonia addition to maintain a chloramine residual to provide a controllable system, maximize compliance with recycled water treatment requirements, and minimize chlorine costs.

Although the chlorine contact tank provides adequate capacity for future recycled water demands, we recommend that the City revisit the 4.0 mgd UV disinfection capacity approved by DHS in the fall of 2003 as there are several potential opportunities to increase system capacity. First, the commissioning testing identified an unequal flow split between the two UV channels. The channel with the higher flow was tested to simulate conservative performance. The delivered UV dose (and system capacity) would increase if the flows were balanced equally. Second, the ambient filter effluent UV transmittance had to be reduced by adding decaf coffee to conduct the tests at 55 percent UV transmittance. A higher UV transmittance (and increased system capacity) could be demonstrated by collecting six months of UV transmittance data based on three grab samples per day. Finally, the City should check with the manufacturer to see if there have been any changes in DHS-approved operations parameters (end of lamp life factor, quartz sleeve fouling factor) for their low-pressure low-intensity UV system that could increase the rated capacity. Even if the UV disinfection system was not used for recycled water production, the UV system could be used for disinfecting filtered effluent for live stream discharge to avoid any concerns of final effluent chlorine residual. Increased UV system capacity would provide an alternative to chlorine disinfection.

Solids Handling

The *Solids Handling Process Evaluation (SHPE) TM* concluded that the capacity of the existing DAFT system and anaerobic digester does not have adequate capacity with one unit out of service to handle projected solids for 18.0 mgd plant influent flow. Therefore, the plant must operate all thickening and digestion facilities to reliably treat this average daily flow. Solutions for the existing flow and the future buildout condition are discussed below.

PROCESS EVALUATION TO RESTORE CAPACITY (AVERAGE FLOW = 18.0 MGD)

As discussed in the *Biological Process Evaluation TM*, the current operation of the HARRF plant is not sufficient to treat 18.0 mgd. The *SHPE TM* identified issues with the thickening and digestion process that need to be addressed for 18.0 mgd of influent flow. An interim solution to bring the treatment capacity of HARRF to 18.0 mgd is discussed for the liquid and solids process streams below. Improvements related to increasing the average daily treatment capacity to 27.5 mgd will be discussed later.

Liquid Stream Processing

In order to increase plant capacity to 18.0 mgd, the solids inventory in the aeration basins must be reduced, the sludge settleability must be improved, and the aeration system capacity must be increased.

To reduce the solids inventory, it is proposed that CEPT be used to reduce organic loading to the secondary process, which will reduce MLSS concentration. This would involve injecting ferric chloride (and potentially polymer) into the primary clarifier influent. One option is to add the ferric chloride at the Parshall Flume to take advantage of the turbulence resulting from the hydraulic jump. A similar approach is employed at the City of San Diego Point Loma Wastewater Treatment Plant where they experience as high as 90 percent TSS and 60 percent BOD removal efficiencies with CEPT.

To improve SVI, a modified approach to chemical use is suggested consisting of RAS chlorination and polymer addition to the aeration basins. However, controlling SVI with chemical addition will require added operator attention involving frequent microscopic analyses of mixed liquor and SVI analyses to prevent overdosing of chemicals. For instance, if SVI values are low (i.e., less than 80), additional RAS chlorination may break up floc and reduce effluent quality. With an improved approach to chemical addition, it is assumed that the plant will have better control of SVI and that a 90^{th} percentile SVI of 150 mL/g can be maintained.

The application of CEPT will reduce the organic loading to the secondary system and therefore reduce the aeration requirements. Aeration requirements can be further reduced my reducing the SRT to 2.0 d to suppress nitrification. However, even with this reduction, the aeration requirements are too high for the existing aeration system (limited by the flow of air through the aeration panels).

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

To supplement the aeration system, a supplemental aeration system can be used on an "as needed" basis. There are several alternatives that can be implemented to supplement the aeration system:

- Pure oxygen gas can be metered in the air piping to supplement the airflow from the existing blowers. However, this method is relatively inefficient and would be costly.
- Pure oxygen or ambient air can be injected into the RAS line using a Venturi-type injector. This would reduce the capacity of the RAS pumps.
- Coarse bubble aerators can be installed down the length of each aeration basin on the side. The coarse bubble aerators would instigate a spiral roll that will result in longer contact times of fine air bubbles (from the existing aeration basin) with liquid increased oxygen transfer efficiency. This would require taking an aeration basin out of service.
- Surface aerators can be installed on the surface of the aeration system or pure oxygen can be metered directly to the aeration basins. These options should be the simplest solution.

Solids Processing

The current DAFT does not have adequate capacity to process 18.0 mgd of equivalent plant influent flow.

To increase the surface solids loading rate and thereby increase the DAFT capacity, the polymer dosage could be increase. However, there is a limit to how high this dosage rate can be increased before polymer is wasted in the effluent stream. Because of the interim solutions to the liquid stream will produce a secondary sludge with an SVI of approximately 150 mL/g, it is possible polymer addition could be decreased and still maintain the higher surface loading. Another possible solution to thickening WAS without overloading a DAFT is to cothicken WAS with the primary sludge in the primary clarifiers. This practice is being done at the Goleta Water Reclamation Plant and being done at the Point Loma Wastewater Treatment Plant (PLWTP) when raw sludge from the South Bay Water Reclamation Plant is discharged to the sewer and eventually thickened in the primary sedimentation tanks at the PLWTP. Brown and Caldwell does not recommend this method of operation because of the potential for hydrolyzing the solids and increasing the soluble BOD load to the activated sludge system. But, it is an option that may be considered in an emergency if both DAFTs were out of service.

The *SHPE TM* identified several process improvements that could be implemented to improve the performance of the DAFTs without making major structural modifications. One modification is to change the location of the polymer injection point closer to the pressurized flow discharge point in the DAFT. This could improve the efficiency of the polymer system and increase the thickened solids concentration, thus reducing the flow the digesters. Two simple modifications to the control of the pressurization tank would increase the amount of air dissolved in the pressurized flow which would improve the solids removal efficiency and possibly produce a thicker sludge. The primary sludge concentration was assumed to be 4.4 percent for separate thickening. It is doubtful that a higher solids concentration could be achieved in the primary clarifiers within the limits of the interim solution recommended for the liquid treatment train.

To increase the hydraulic retention time in the digesters, some municipalities are using recuperative thickening at the digesters. The City of Santa Rosa currently practices recuperative thickening of their digested sludge to maintain sufficient digester capacity to meet the EPA 503 regulations. To provide 20-day HRT with one unit out of service at projected solids production at 18.0 mgd, 0.055 mgd of liquid would need to be removed from the digesters. This could be accomplished by using temporary rental centrifuges or gravity belt thickeners to thicken a portion of the flow going through the digesters. If the recuperative thickening system is operated on a 12 hr/day/5 day/week schedule, the recuperative thickening system would need to have a capacity of 200 gpm. Since there is adequate capacity to handle average daily flow above 18.0 mgd when all digesters are in service, this would only need to be operated when a digester is taken out of service. As stated in the SHPE TM, the most likely reason for a digester to be out of service would be as a result of a process upset. Process upsets can be avoided by carefully monitoring the pH and alkalinity of the digesters and by keeping toxic substances out of the solids stream. Therefore, since providing additional digester capacity on an interim basis cannot be accomplished without additional equipment, careful monitoring and control of the digester process as well as maintenance of equipment to avoid unexpected equipment failure is the easiest interim solution until additional digester capacity of improved thickened sludge concentration can be achieved.

PROCESS EVALUATION TO TREAT BUILD-OUT FLOWS (AVERAGE DAILY FLOW = 27.5 MGD)

Several processes were evaluated to meet the future flows and loads at HARRF. Potential processes that would be appropriate for HARRF were identified. The initial list was reduced to eight options, which were evaluated in more depth and reduced to two options deemed the most viable. Costs were determined for each of these two viable options. In identifying viable processes, one of the objectives was to make use of the installed tankage and associated infrastructure as much as possible. For instance, trickling filters (TF) were not considered because this would require not only abandoning the aeration basins, but potentially demolishing them to make room for the TF units. However, TF in conjunction with activated sludge, or TF/AS, was considered because it made use of the existing aeration basins. Table 3 summarizes processes that were initially considered.

Table 3
List of Options Evaluated for HARRF

Preliminary Options	Potential Options	Viable Alternatives
Conventional Activated Sludge (CAS) w/ Nitrification	Conventional Activated Sludge (CAS) w/ Nitrification	High-Rate Activated Sludge (HRAS)
Bioaugmentation Reaeration (BAR)	Bioaugmentation Reaeration (BAR)	Moving Bed Biological Reactor (MBBR)
High-Rate Activated Sludge (HRAS) Membrane Bioreactor (MBR)	High-Rate Activated Sludge (HRAS) Membrane Bioreactor (MBR)	
Flow Equalization	Flow Equalization	
Moving Bed Biological Reactor (MBBR)	Moving Bed Biological Reactor (MBBR)	
Sludge Reaeration Activated Sludge (SRAS)	Sludge Reaeration Activated Sludge (SRAS)	
Biological Contact Process (BCP) High-Purity Oxygen Activated Sludge (HPOAS)	Biological Contact Process (BCP)	
Sequencing Batch Reactor (SBR) Trickling Filter/Activated Sludge (TF/AS)		
Trickling Filter/Solids Contact (TF/SC) Integrated Fixed-Film Activated Sludge (IFAS)		
Biological Aerated Filter (BAF)		
Step Feed Activated Sludge (SFAS)/Contact Stabilization Activated Sludge (CSAS)		

Conventional Activated Sludge (CAS) with Nitrification The CAS process is the process currently in use at HARRF consisting of five plug-flow aeration basins followed by four secondary clarifiers for solid-liquid separation. The existing plant is not sufficient to treat future flows and loads. Constructing a sixth aeration basin will not provide the required tankage necessary for installing an anoxic selector and increasing the SRT to 5.0 days to produce a nitrified effluent. In order for this option to be viable, the organic loading to the secondary system must be reduced with the implementation of CEPT. In addition, the tanks must be lengthened to increase volume. This was considered a potential option and was further investigated.

High-Rate Activated Sludge (HRAS) The HRAS process is similar to the CAS process. For the HRAS, the process SRT would be reduced to 2.0 d to suppress nitrification and lower solids inventory. An anaerobic selector would be installed in each aeration basin to minimize sludge bulking. The existing plant is not sufficient to treat future flows and loads even at the reduced SRT and constructing a sixth aeration basin will not provide the required tankage necessary. In order for this option to be viable, the organic loading to the secondary system must be reduced with the implementation of CEPT. This was considered a potential option and was further investigated.

Flow Equalization For this option, primary effluent would be equalized using a storage tank. Flow from the tank could be controlled so that the solids loading rate to the secondary clarifier is maintained below the critical loading. For this option, it is assumed that the secondary process would be HRAS. Because HRAS operates at a low SRT, the solids inventory in the aeration basins is lower than for a nitrifying system which minimizes the volume of the equalization tank. This was considered a potential option and was further investigated.

High-Purity Oxygen Activated Sludge (HPOAS) The HPOAS process is a modification of CAS where in lieu of a fine-bubble aeration system, the aeration tanks are covered and high-purity oxygen gas is introduced to the headspace above the liquid surface. Mechanical aerators are used to transfer the oxygen into the mixed liquor. The advantage of the HPOAS is a reduced reactor volume. However, most HPOAS plants in operation are operated at reduced SRT (less than 2.0 days) to mitigate biological foaming; the HPOAS is a trapping system and is prone to, sometimes severe, biological foaming events. The HPOAS process was eliminated due to issues with biological foaming and cost (both capital and O&M).

Sequencing Batch Reactor (SBR) The SBR is a semi-batch activated sludge process where a reactor containing activated sludge is filled with influent, allowed to react under aeration, settled to separate solids from liquid, and decanted to removal clarified effluent. The advantage of the SBR is that the aeration basin and secondary clarifier are combined into one unit process. However, an SBR is not a good option for a plant that undergoes peak wet weather events and would require large amounts of flow equalization. The SBR process was not considered further for HARRF.

Step Feed Activated Sludge (SFAS)/Contact Stabilization Activated Sludge (CSAS) SFAS is a modification of CAS where primary effluent is introduced at different locations along the length of the aeration basins. The advantage of SFAS is that the aeration requirements are evened out across the tank reducing the maximum aeration requirements. In addition, the concentration of the MLSS gets progressively less towards the end of the aeration basin so that the solids loading rate (SLR) to the secondary clarifiers is reduced. For periods of high flow, primary effluent can be directed only to the downstream end of the aeration basin (or contact zone). In the contact zone, the soluble BOD is oxidized and particulate BOD becomes enmeshed in the floc. The sludge is settled in the secondary clarifier, and while in the stabilization zone, the particulate BOD is solubilized and oxidized. The MLSS concentration is much higher in the stabilization zone compared with the contact zone and results in a reduction of SLR to the secondary clarifiers. SFAS/CSAS was not considered a viable option for HARRF because it would be difficult to reconfigure the existing aeration basins to allow multiple primary effluent feed locations.

Sludge Reaeration Activated Sludge (SRAS) The SRAS process is similar to the CSAS process where primary effluent is introduced in a contact zone downstream of where RAS is introduced. The difference between SRAS and CSAS is that contact times are much longer in the SRAS process; 2 to 4 hours for SRAS versus 30 minutes for CSAS. For HARRF, only modifications to the RAS would be necessary, primary effluent could still be introduced through the existing influent channel. SRAS was further considered for HARRF as a potential option.

Bioaugmentation Reaeration (BAR) The BAR process is a modified activated sludge process. Similar to the CSAS, RAS is directed to the head of the aeration basin (reaeration zone) and primary effluent is introduced downstream. As a result, the MLSS concentration directed to the secondary clarifiers is reduced allowing smaller reactors. Recycle streams from the thickening and dewatering processes containing high levels of ammonia (which imparts a significant aeration requirement) are sent to the reaeration portion of the aeration basin. By sending the solids processing recycle streams to the reaeration zone, nitrification is possible at reduced SRT due to seeding of nitrifying organisms to the rest of the system. The BAR process was identified as a viable option for HARRF.

Trickling Filter/Activated Sludge (TF/AS) or Trickling Filter/Solids Contact (TF/SC) The trickling filter (TF) process is a fixed-film process where primary effluent is fed to the top of tank filled with media and biofilm on the media oxidizes organic material. A TF can be place upstream of an activated sludge (TF/AS) to reduce the organic loading to the activated sludge so that smaller aeration basins can be used. Typically, an activated sludge process will produce a better effluent when compared with TF effluent; TF effluent may contain large amounts of colloidal material because TF sludge is not a flocculent sludge. Coupling a TF process with activated sludge will produce a better effluent. If the activated sludge process is operated at a lower SRT (approximately 1 d) the process is called a TF/SC process where aeration basins are smaller and the solids contact portion of the process acts to flocculate sludge to improve effluent quality. TF/AS and TF/SC processes are typically designed for carbonaceous BOD removal and may nitrify. However, biological nutrient removal may not be feasible. To continue discharging to the land outfall does not require nutrient removal, but future regulations may change as may disposal options. Converting HARRF to a TF/AS or TF/SC process would limit future options available to HARRF. TF/AS and TF/SC were not further considered as viable options.

Integrated Fixed-Film Activated Sludge (IFAS) The IFAS process uses suspended carrier media (e.g., plastic media) placed in some or all of the aeration basin. Screens are placed at the effluent end of the aeration basin are used to retain media. The advantage of the IFAS process is that nitrification is possible below the minimum SRT required for activated sludge because nitrifying organisms grow on the carrier media. Sludge from the secondary clarifier is recycled back to the head of the aeration basin so that the system carries a MLSS inventory. The presence of MLSS can improve effluent quality due to particle flocculation. As a result, the process can operate at a lower SRT and result in either a lower MLSS concentration or less tankage. The IFAS process is ideal for cold-temperature application where nitrification is necessary. For HARRF, the operating SRT could be reduced with the IFAS process but would result in a high-rate process and elevated oxygen uptake rates (OUR). High OUR conditions have been shown to result in sludge bulking. Because a well settling sludge is essential for HARRF, the IFAS was not considered further because of the potential for sludge bulking.

Moving Bed Biological Reactor (MBBR) Similar to the IFAS process, the MBBR process uses carrier media suspended in aeration basins and screens retain media. The difference between the two processes is that in the MBBR process there is no sludge recycle from the secondary clarifiers and sludge that sloughs off of the carrier media settles out –generally quite easily– in the secondary clarifiers and is sent for solids processing. The MBBR process can be designed for both carbonaceous BOD removal and nitrification; longer hydraulic retention times and additional carrier media are required to achieve nitrification. The MBBR process was further considered for HARRF.

Biological Aerated Filter (BAF) BAF process is a submerged fixed-film biological reactor in which microorganisms, attached to reactor media and occupying the interstices of the media bed, reduce the carbonaceous and/or nitrogenous content of the incoming wastewater. The reactor media also retains insoluble solids present in the incoming wastewater and those generated within the reactor, thus eliminating the need for a separate clarification process. Excess microbial growth and trapped solids are purged from the reactor by backwashing with treated wastewater to make room for new microbial growth. Backwash cycling can be automated to initiate on differential pressure (headloss) or on run-cycle-time. BAFs can be configured for carbonaceous biochemical oxygen demand (CBOD) removal, nitrification and/or denitrification. The BAF system can completely replace the activated sludge process for CBOD removal. However, this would require abandoning and demolishing the existing aeration basins at HARRF and was not considered further as a viable option.

Membrane Bioreactor (MBR) As mentioned above, the MBR is a modified activated sludge process where a membrane (either ultrafilter or microfilter) is used to perform solid-liquid separation eliminating the need for secondary clarifiers. Although not necessary, the primary clarifiers at HARRF would be retained to reduce organic loading to the MBR. The capacity of an MBR is determined by the amount of flow that can pass through the membranes and peaking factors are typically 2.0 or less. Higher peaking factors will require additional membrane which will increase cost. For HARRF, the wet weather peaking factor is 2.0. MBR was further considered for HARRF as a potential option.

Biological Contact Process (BCP) The BCP is another process similar to the CSAS where primary effluent and RAS are combined for a short (approximately 30 minute) contact time where particulate BOD is adsorbed by mixed liquor. For the BCP process, an additional biological contact tank would be constructed. During high-flow events, the contact tank would be charged with RAS and a portion of primary effluent. The net result is that the SLR to the secondary clarifiers is reduced while maintaining effluent quality. BCP was further considered for HARRF as a potential option.

EVALUATION OF POTENTIAL OPTIONS FOR HARRF

From the initial evaluation of processes suitable for HARRF, eight potential alternatives were further investigated:

- Alternative 1 - Conventional Activated Sludge (CAS) w/ Nitrification
- Alternative 2 - Bioaugmentation Reaeration (BAR)
- Alternative 3 - High-Rate Activated Sludge (HRAS)
- Alternative 4 - Membrane Bioreactor (MBR)
- Alternative 5 - Flow Equalization
- Alternative 6 - Moving Bed Biological Reactor (MBBR)
- Alternative 7 - Sludge Reaeration Activated Sludge (SRAS)

- Alternative 8 - Biological Contact Process (BCP)

The process requirements for each of the eight alternatives were determined based on dry weather and wet weather loading conditions of the plant. MLSS content was determined using the calibrated BioWin model (model was not used for Alternative 6) and a dynamic simulation was performed using the maximum month condition identified from the historic record (March 1, 2001 to March 31, 2001). A 30-day simulation was performed using the loading patterns determined from the historic record and applying the diurnal variation determined from the wastewater characterization study. The flow rates for the modeling were increased to reflect the future ADWF of 27.5 mgd and peak wet weather flow (PWWF) of 53.4 mgd. For each alternative where secondary clarifiers are used (except the MBBR option) a 90^{th} percentile SVI value of 125 mL/g was assumed. A 125-mL/g SVI will be possible with a biological selector (either anaerobic or anoxic). For periods where CEPT is used, anaerobic selector performance may be compromised due to the reduction of phosphorus resulting from chemical addition; phosphorus removal in the anaerobic selector is necessary to mitigate sludge bulking. For periods of CEPT, it is assumed that chemical addition (polymer or RAS chlorination) can be used to control SVI. Table 4 shows the capacity of the existing secondary clarifiers and capacity assuming the addition of two more clarifiers. For some of the options, the addition of secondary clarifiers was necessary to meet treatment goals.

Table 4
Secondary Clarifier Requirements for Alternatives 1, 2, and 3 at PWWF of 53.4 mgd
(90^{th} Percentile SVI=125 mL/g)

Item	Total Clarifier Surface Area (ft^2)	Maximum RAS Capacity (mgd)	Recycle Ratio (%)	Critical Solids Loading Rate (lb/ ft^2-d)	Critical MLSS (mg/L)
6 clarifiers *four 110-ft diameter and two 80-ft diameter*	48,100	43.2	80	43.0	2,580
5 clarifiers *three 110-ft diameter and two 80-ft diameter*	38,600	34.6	64	43.1	2,280
4 clarifiers *two 110-ft diameter and two 80-ft diameter*	29,000	25.9	48	37.5	1,650

The solids processing requirements of viable alternatives were also determined. Two scenarios were investigated: separate thickening of primary and secondary sludges and co-thickening of primary and secondary sludges. It is assumed that if separate thickening were performed, primary sludge would be thickened in the primary clarifiers to 4.4 percent solids. For co-thickening, it was assumed that primary sludge is 2.0 percent solids. For all alternatives (except Alternative 4), it was assumed that the WAS would be 0.15 percent solids. By sizing the thickening units for a concentration typical of unsettled mixed liquor, there exists the flexibility to implement mixed liquor wasting for better process control. This is particularly advantageous for the Alternative 3 where high-rate CAS is used;

a lower SRT operation is more susceptible to process changes and mixed liquor wasting would act to stabilize operations.

The criteria for the sludge thickening and dewatering units is based on one unit out of service during average loading conditions and all units in service during peak loading. Currently, there exists a bottleneck from the secondary digester to the centrifuges and for all alternatives and this would need to be corrected. The anaerobic digesters are sized for a 15-d SRT at peak loading conditions. Digester sizing for the seperate thickening option and co-thickening option DAFT thickened solids concentration of 5.0 percent and 6.0 percent respectively are assumed. Based on lower SVI for the WAS going to the thickening process from enhancements to the secondary treatment system, this thickened sludge concentration is believed to be achievable. Experience at other plants that are co-thickening with DAFTs also has shown that this thickened solids concentration is reasonable. Pilot testing would be needed to confirm actual performance for both thickening options. Digester diameters are calculated based on an assumed sidewater depth of 25 feet to match the existing digesters.

Alternative 1 – Conventional Activated Sludge (CAS) w/ Nitrification

Liquid Processes for Alternative 1

For Alternative 1, it is assumed that one additional primary clarifier will be constructed and the TSS removal with CEPT will be 77 percent. An anoxic selector (approximately 20 percent of the total aeration basin volume) would be installed at the head end of each aeration basin. In order to produce a consistently nitrified effluent (less than 2 mg-N/L), the secondary process would be operated at an aerobic SRT of 5 days (the volume occupied by the anoxic selector is not included in the SRT calculation).

Even with the increased organic reduction due to the CEPT, and the construction of an additional aeration basin, the MLSS concentration (peak MLSS of 3,800 mg/L) would be too high for the secondary clarifiers. In order to meet SLR requirements of the secondary clarifiers, the existing aeration basins will need to be lengthened by 50 percent. Deepening the aeration basins is another option, but is considered too costly and difficult to stage. Expanding the aeration basins will mean that they would encroach on the area currently occupied by the chlorine contact basin and the two older secondary clarifiers. Increasing the tankage by 50 percent will reduce the peak MLSS to 2,530 mg/L and would require constructing three additional secondary clarifiers (one secondary clarifier to replace the two 80-ft clarifiers).

From conversations with plant staff, Alternative 1 was considered not feasible due to the additional tankage and secondary clarifiers required and was eliminated. Solids processing requirements evaluation was not conducted.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Alternative 2 – BAR

Liquid Processes for Alternative 2

For Alternative 2, it is assumed that an additional primary clarifier will be constructed and the TSS removal with CEPT will be 77 percent. An additional aeration basin would be constructed and an anoxic selector (approximately 20 percent of the total aeration basin volume) would be installed at the head end of each aeration basin. All basins would be lengthened to 50 percent as in Alternative 1. All solids processing recycle streams (DAFT subnatant and centrate) would be equalized and sent to the reaeration zone. Results of the model simulation showed that the MLSS would be 2,230 mg/L. As in Alternative 1, secondary clarifiers 1 and 2 would be demolished to make room for the lengthened aeration basins. However, two additional secondary clarifiers would need to be built, as opposed to the three necessary for Alternative 1; the reaeration portion of the BAR process allows the elimination of one of the secondary clarifiers. However, the results of the BioWin simulation showed more breakthrough of ammonia during peak loading conditions when compared to Alternative 1. This may be a result of the reduced contact time caused by the additional volume occupied by the anoxic zone and the reaeration zone.

From conversations with plant staff, Alternative 1 was considered not feasible due to the additional tankage and secondary clarifiers required and was eliminated. Solids processing requirements evaluation was not conducted.

Alternative 3 – HRAS

Liquid Processes for Alternative 3

For Alternative 3, one additional primary clarifier would be constructed and the TSS removal with CEPT is estimated to be 77 percent and, without CEPT, is estimated to be 62 percent. The CAS process would be operated at an aerobic SRT of 2.0 d. The head end of the each aeration basin would be converted to an anaerobic selector (approximately 20 percent of the total aeration basin volume) to mitigate sludge bulking and improve sludge settleability. In addition, a new pump station would be constructed for mixed liquor wasting. Mixed liquor wasting provides for better process control over settled sludge wasting. The existing settled sludge wasting system would be retained. During dry weather operation, CEPT will not be necessary. However, during PWWF conditions, CEPT is necessary to control MLSS concentration. Using CEPT for this application requires its use as a preventative measure. During periods of the year where wet weather events are known to occur, the CEPT must be used to control the MLSS concentration to the critical solids concentration (Table 5 – The 4-Clarifier scenario) so that if a PWWF event occurs, solids washout does not occur. Therefore, this option requires operations to actively track MLSS inventory, in addition to SRT, to ensure process performance.

Alternative 3 does not require that any additional secondary clarifiers are constructed if CEPT is used during the wet weather season. However the peak capacity of the plant (determined by PWWF conditions) was determined at a 90th percentile SVI of 115 mL/g instead of 125 mL/g. This reduced value is necessary to control solids loading rate to the secondary clarifiers and to prevent construction of an additional clarifier. It is assumed that at this peak condition, the reduction of SVI from 125 mL/g to 115 mL/g is possible with polymer addition and/or RAS chlorination.

Solids Processes for Alternative 3

Table 5 summarizes the sludge production for Alternative 3. Primary sludge production will be higher due to the chemical addition for CEPT (CEPT was assumed to occur continuously as a conservative estimate of process requirements; note that CEPT will not be necessary during dry weather conditions). As a result, the secondary sludge production is reduced due to the reduction in organic loading to the aeration basins. Because the HRAS is operated as a high rate system (SRT=2.0 d), the process is more susceptible to changes in influent loadings. Therefore, it is recommended that sludge wasting be performed using mixed liquor wasting to achieve improved process control.

Table 5
Sludge Production Criteria for Alternative 3

Item	Sludge Production (lb/d)	Sludge Flow (mgd)	
Primary Sludge		**Separate Thickening**	**Co-Thickening**
Average Day	65,400	0.177	0.392
Peak Two Week	102,100	0.277	0.612
Peak Day	134,700	0.366	0.808
Secondary Sludge[a]			
Average Day	28,970	2.32	2.32
Peak Two Week	36,440	2.91	2.91
Peak Day	41,320	3.30	3.30

Co-thickening primary and secondary sludge would require two additional 37-ft diameter DAFTs and one additional 109-ft diameter anaerobic digester. If sludge dewatering were performed 24 hours per day, no additional centrifuges would be necessary. However, 12-hour operation is currently performed which would require one additional centrifuge.

If separate thickening of primary and secondary sludges were performed, one additional 36-ft diameter DAFT unit and one additional 141-ft diameter anaerobic digester would be necessary. As before, if 24-hour per day dewatering were performed, no new centrifuges would be necessary. However, to maintain the current 12-hour per day operation, two additional centrifuges would be necessary.

After analysis and conversations with plant staff, Alternative 3 was considered feasible.

Alternative 4 – MBR

Liquid Processes for Alternative 4

For Alternative 4, the existing primary clarifiers would be used and the TSS removal is estimated to be 58 percent. The existing aeration basins would be retained and an additional aeration basin would be constructed. A separate set of tanks would be constructed that would contain the submerged membranes and additional space for blowers, RAS pumps and ancillary equipment would be required. For this analysis, it was assumed that the membrane equipment would be located where the existing chlorine contact basin is located. This would require that the chlorine contact basin be re-located. The head end of each aeration basin would be converted to an anoxic selector (25 percent of total aeration basin volume) to recover alkalinity consumed by nitrification; sludge bulking is not an issue in an MBR because membranes are used for solid-liquid separation. The anoxic zone also will provide some oxidation of organic material and reduce the oxygen requirements in the aerated portion of the aeration basins.

For Alternative 4, an 8-d SRT was assumed. Operation at an 8-d SRT will produce a nitrified effluent and will result in MLSS concentrations acceptable for membrane operation (peak MLSS concentration of approximately 7,500 mg/L). However, the peak OUR value at the head of each aeration basin is approximately 175 mgO_2/L-h, which is high and may not be possible with a fine-bubble aeration system. This could be mitigated by constructing additional aeration basins, which is feasible because of the additional land available due to the elimination of secondary clarifiers. Another option would be to use CEPT to reduce the loading to the MBR instead of constructing new aeration basins.

From conversations with plant staff, Alternative 4 was considered not feasible due to the additional tankage and was eliminated. Solids processing requirements evaluation was not conducted.

Alternative 5 – Primary Effluent Flow Equalization

For Alternative 5, it was assumed that flow equalization of primary effluent would be used with HRAS. However, flow equalization may be possible with other processes. Flow equalization would be used in place of CEPT to reduce flows to the secondary system; loadings would be relatively unchanged. Assuming no new secondary clarifiers are constructed and a maximum MLSS concentration of 2,400 mg/L (determined from BioWin modeling), the maximum flow that secondary clarifiers can treat approximately 36 mgd. Assuming that the flow equalization tank is emptied every day, the HARRF would require a 8-MG of equalization volume. Two 4-MG tanks can be constructed in the two 110-ft diameter areas which are currently reserved for future secondary clarifiers. This available area is not wide enough to provide the optimal height to width ration for the 4-MG tank. As a result a taller tank needs to be constructed, which will increase the cost. The construction cost for the two 4-MG tank is estimated to be more than $12 Million. This alternatives has inherent disadvantages; namely, the need for a significant odor control system and solids removal system in the tank.

It was concluded that using flow equalization is not a feasible option due to space requirements for influent storage, high cost, and odor concerns. Solids processing requirements evaluation was not conducted.

Alternative 6 – Moving Bed Biological Reactor (MBBR)
For the MBBR options for HARRF, the first 50 feet of each aeration basin would be converted to a MBBR. The MBBR would require constructing a concrete wall to isolate the media from the downstream tankage. Media would be placed in the MBBR zone (approximately 49 percent fill) and screens would be installed to retain media. The MBBR would remove a majority of the carbonaceous BOD; no nitrification will occur. The downstream portion of the aeration basins would be used for solids contact to improve remove residual BOD and flocculate sludge to improve settleability. Solids that slough off the carrier media from the MBBR zone would contribute to the MLSS in the solids contact zone. The solids contact zone would be operated with a 1-d SRT and settled sludge from the secondary clarifiers would be recycled to the head of the solids contact zone. Operation at a 1-d SRT would promote sludge flocculation and suppress nitrification to reduce aeration requirements.

The MBBR option would not require any additional aeration basins or secondary clarifiers. However, conversion of the plant would require that an aeration basin would be taken out of service reducing the total number of aeration basins to four. With the implementation of an improved sludge settleability control and CEPT, HARRF can be operated with four aeration basins.

Based on estimates provided by AnoxKaldnes, the OUR values in the MBBR will be 143 mgO_2/L-hr under peak week loading, which means peak day loading will be higher. This value may be difficult to achieve. The solids processing requirements are assumed to be similar to Alternative 3. The MBBR process is considered a newer treatment technology and a pilot test is recommended to determine performance and estimate aeration and sludge production requirements.

After analysis, Alternative 3 was considered feasible. However, a pilot study is recommended to validate performance and determine aeration and sludge production requirements.

Alternative 7 – Sludge Reaeration Activated Sludge (SRAS)

<u>Liquid Processes for Alternative 7</u>

For Alternative 7, it is assumed that one additional primary clarifier will be constructed and the TSS removal will be 62 percent. The SRAS process would be operated at an aerobic SRT of 2.0 d and two of the aeration basins would be converted to reaeration zones where RAS would be sent. The head end of the each aeration basin would be converted to an anaerobic selector (approximately 20 percent of the total aeration basin volume) to mitigate sludge bulking and improve sludge settleability. For Alternative 7 to be cost effective compared with Alternative 3, it is assumed that CEPT would not be used and no additional secondary clarifiers would be constructed.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

For Alternative 7, one additional aeration basin is required due to aeration requirements. Even with the addition of a secondary aeration basin, the peak OUR values are predicted to be as high as 167 mg/L-hr which is at the limit possible with typical aeration systems. In addition, using two aeration basins for reaeration may make project staging difficult.

Upon further investigation, Alternative 7 was considered not feasible due to the high aeration requirements and issues with staging. Solids processing requirements evaluation was not conducted.

Alternative 8 – Biological Contact Process (BCP)

For Alternative 8, an additional primary clarifier and aeration basin would be constructed. In addition, a contact tank would be constructed where, during peak flow events, primary effluent and RAS would be combined for a short contact period. The overall effect is that the SLR to the secondary clarifiers is reduced. Several scenarios were modeled to determine the amount of flow that would require treatment through the contact tank and the volume of the tank. The conclusion was that the amount of flow bypassed and the volume of the contact tank were too high to be feasible. This is because during the peak month of BOD loading, the solids inventory in the aeration basins is relatively high, and the dilution effect possible with the contact process is not sufficient to reach the target SLR to the secondary clarifiers.

Upon further investigation, Alternative 8 was considered not feasible due to the size of the contact tank necessary.

TERTIARY TREATMENT

Recovery of process performance of the existing granular filters may be possible with further investigation, and their replacement may not be necessary. However, as a part of the analysis performed for future conditions, three options were identified as advanced treatment options in place of the existing granular filtration that could be used in conjunction with the secondary processes identified previously to produce water suitable for the reclaimed water system.

- Option A – Nitrifying BAF
- Option B – Membrane Filtration (MF)
- Option C – Sidestream MBR

The three options for Alternative 3 are shown in Figure 5 as an example (only options B and C would be used with Alternative 6). Each system would be sized to treat 9 mgd; there is no benefit to producing water suitable of water reclamation for ocean outfall disposal. It was estimated that, based on the existing performance of HARRF and the current permitted mass emission rate for TSS and cBOD, the annual mass loading to the Pacific Ocean at the buildout out flow rate of 27.5 mgd will not exceed the allowable, permitted mass loading. This means that the City will not likely have to perform an Antidegradation Analysis for the increased volumetric discharge under buildout conditions.

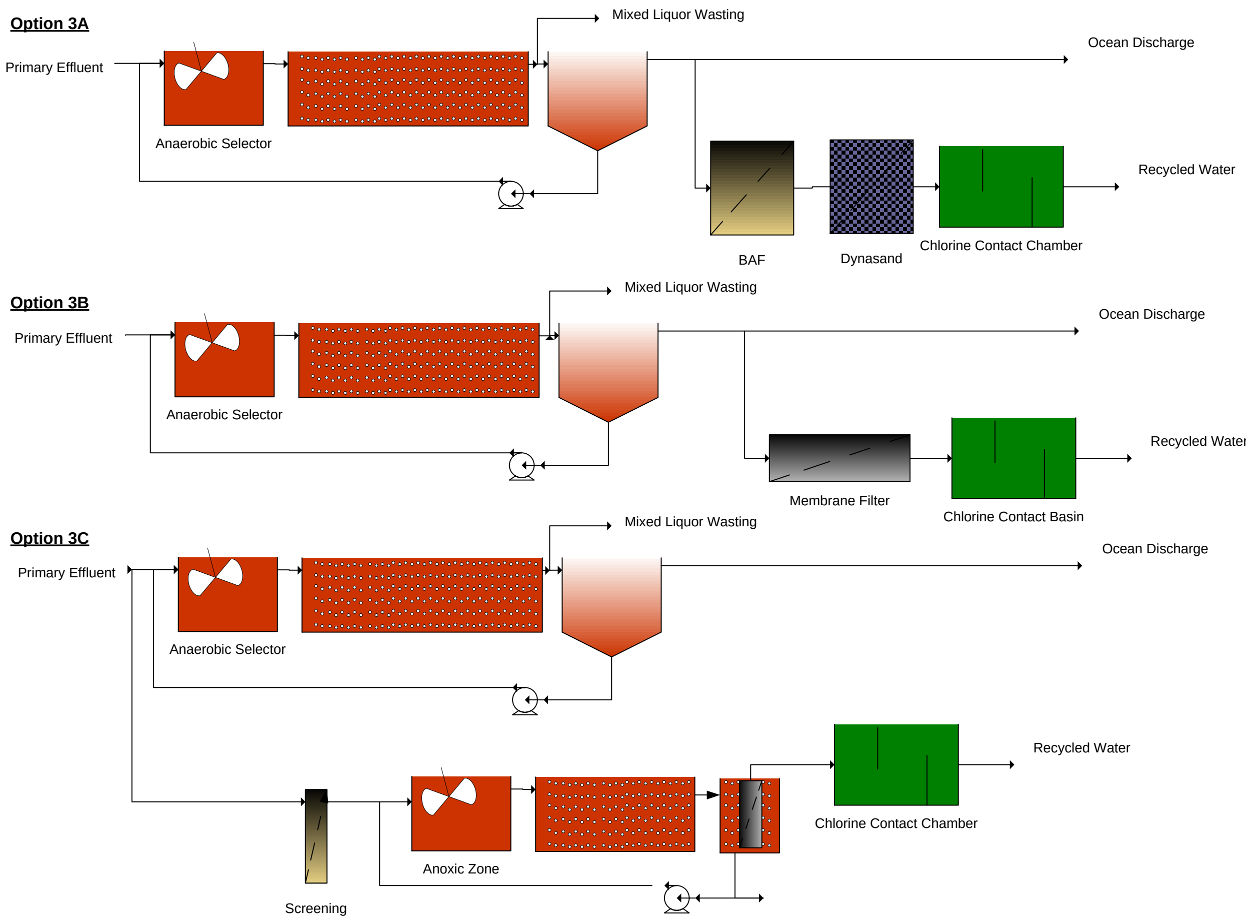


Figure 5. Advanced Treatment Options for Alternative 3.

Option A – BAF for nitrification. For Option A, a BAF unit capable of nitrification would be used. A Biofor system was used to determine the footprint requirements and process requirements. The Biofor reactors contain a submerged, fixed, and heavy media bed. Secondary effluent wastewater flows upward through the media, co-current with the air provided for aerobic decomposition of organics or nitrification. The media, called Biolite, is an expanded clay material with high specific surface area that ensures good biomass attachment. The media is of high density and has good resistance to attrition. It ranges in size from 1 to 5 millimeters (mm), depending on the application. For nitrification, the average media will likely be 3.5 mm. At the bottom of the media bed, directly above a plenum that contains nozzles through which wastewater passes, is an air distribution network consisting of pipes fitted with proprietary coarse bubble air diffusers called Oxazur. Diffused air and gas retention within the media results in oxygen mass transfer characteristics similar to fine bubble diffusion. The nozzles embedded in the plenum (i.e., media floor) encourage even distribution of the incoming wastewater. Screens with 2.5 mm openings are required as a pretreatment step to prevent these nozzles from clogging. Clogged nozzles will cause poor distribution and resulting reduced process efficiency as well as increased pressure headloss across the unit and reduced flow.

Biofor units are backwashed in an upward (co-current) direction. The water used for backwashing is typically Biofor effluent stored in a separate tank and pumped upward through the media during backwash sequences. Generally, backwashing is required every 24 hours or more. The backwash solids are stored in a separate basin sized to minimize the impact of the backwash waste (typically returned to the headworks or the head of the PSBs). For HARRF, the sludge production from the BAF unit will be relatively minimal because the unit is designed to nitrify the secondary effluent from the clarifiers.

Operation of a BAF on a non-nitrified secondary effluent may not produce an effluent with turbidity values less than 5 NTU. It is recommended that a pilot test program be implemented if Option A is pursued further.

Option B – Membrane Filtration. For Option B, a membrane unit would be used to treat secondary effluent. The membrane effluent would not require additional filtration from the existing filtration process at HARRF and could be sent directly to disinfection. There are several membranes manufactured that would be suitable for this application. A Zenon ultrafiltration (UF) system using the 500D membranes was used to determine footprint requirements. The Zenon membrane is a submerged membrane with a nominal pore size of 0.035 micron. The membranes are hollow fiber and are operated in an outside-in configuration. A portion of the liquid in the membrane tank is continuously removed and returned to the plant headworks. Backwashing is performed intermittently (approximately every 10 minutes) and chemical cleaning with sodium hypochlorite is performed as needed to restore membrane permeability (every 30 days or longer depending on operating conditions and influent water quality).

Because the secondary system would be operated as a high-rate system (with the exception of the MBBR Alternative), the effluent may contain a higher amount of colloidal and soluble material that could foul the membrane. It is recommended that a pilot test program be implemented if Option B is pursued further.

Option C – Membrane Bioreactor (MBR). For Option C, a MBR would used to treat 9 mgd of flow downstream of the primary clarifiers. An additional screening step (1-3 mm) would be required before the MBR. The sixth aeration basin that would be constructed would be used for the MBR aeration tanks and additional membrane tanks would need to be constructed. For Option C, a Zenon MBR system was used to determine footprint requirements using the 500D membrane.

For the MBR, it was assumed that an 8-d SRT is used. Operation at an 8-d SRT will produce a nitrified effluent and will result in MLSS concentrations acceptable for membrane operation. However, the peak OUR value at the head of the aeration basin is approximately 160 $mgO_2/L\text{-}h$, which is high and may not be possible with a fine-bubble aeration system. The MBR system could be designed to include an additional aeration basin which will reduce the OUR value. For the CAS plant, the peak MLSS is estimated to be approximately 1,420 mg/L with five aeration basins and

1,780 mg/L with four aeration basins, which means no additional secondary clarifiers would need to be constructed for either situation.

For Option 3C, co-thickening primary and secondary sludge would require two additional 37-ft diameter DAFT units and one additional 109-ft anaerobic digester. If sludge dewatering were performed 24 hours per day, no additional centrifuges would be necessary. However, 12-hour operation is currently performed which would require one additional centrifuge.

If separate sludge thickening were performed for Option 3C, the secondary sludge from CAS and MBR could be thickened together. Recent research has shown that there is uncertainty associated with the sludge thickening of MBR sludge using a DAFT. However, for Option 3C, the MBR sludge is a significantly smaller portion of the total secondary sludge and the DAFT is expected to be an appropriate thickening option. For the separate sludge thickening option, two 36-ft DAFT units and one 142-ft anaerobic digester are required. If 24-hour per day dewatering were performed, no new centrifuges would be necessary. However, to maintain the current 12-hour per day operation, two additional centrifuges would be necessary.

DISCUSSION OF OPTIONS

The existing capacity of HARRF is insufficient to meet the future average flow of 27.5 mgd and peak wet weather flow of 53.4 mgd. Several options were investigated that would increase the capacity of the plant. It was determined that from a process performance perspective, the existing primary clarifiers will provide adequate TSS removal for the future flows, however it is recommended that an additional clarifier be constructed to provide sufficient redundancy. There is a hydraulic bottleneck that limits the hydraulic capacity of the units. It is recommended that the hydraulic bottleneck be corrected.

Eight alternatives were identified as viable options for the secondary system. Of the eight options considered, only two were considered as viable alternatives for HARRF: Alternative 3 (HRAS) and Alternative 6 (MBBR). Three processes were identified as advanced treatment option alternatives that could replace the existing granular filters to produce water suitable for water reclamation: (A) BAF, (B) membrane filtration and (C) MBR. Option C adds additional complexity because it would require operation of two activated sludge plants and Options B and C are considered better selections. For Alternative 3, either Option A or B could be used. However, if Option A were used, it is recommended that a pilot test is performed. For Alternative 6, only Option B was considered for tertiary treatment. Table 6 summarizes the requirements for each secondary system alternative.

Our recommended tertiary treatment approach focuses on optimizing capacity and performance of the existing filtration system, increasing the rated capacity of the existing UV disinfection system and/or optimizing the chlorine disinfection facilities to comply with Title 22 requirement. Supplemental treatment facilities (e.g., reverse osmosis) would be needed for all alternatives if additional recycled water quality based requirements were set. Supplemental treatment facilities would also be needed if additional biological treatment for nutrient removal and tertiary treatment were required for an alternative effluent disposal scheme, such as live-stream discharge. Our recommended treatment approach and facilities layout anticipates such future requirements and leaves the maximum amount of available area within the plant site.

Alternative 3 is similar in design to what the existing plant staff are accustomed. The plant would be operated a reduced SRT (2 d) and an anaerobic selector to control SVI. An additional aeration basin would be necessary and intermittent CEPT would be required to control the mixed liquor inventory and would require that operations actively track MLSS concentration so that the secondary clarifiers are not overloaded during a peak flow event.

For Alternative 6, the construction of an additional aeration basin could be avoided by converting a portion of the existing aeration basins to a MBBR process. The MBBR portion would be operated to remove carbonaceous BOD only. The solids contact portion would be operated at a 1-d SRT to remove additional BOD and provide for solids flocculation to improve settleability. For Alternative 6, it is recommended that a pilot test is performed to verify process performance and determine aeration and sludge production values.

Table 6
Summary of Viable Secondary Process Alternatives for HARRF

Alternative	Secondary Process	Number of Additional Secondary Clarifiers Required	CEPT	Additional Requirements for Liquid Processes
3A	HRAS with CEPT and BAF	0	Yes	Install BAF
3B	HRAS with CEPT and Membrane Filtration	0	Yes	Install membrane filtration
3C	HRAS with CEPT and MBR	0	Yes	Install MBR and screening facility
6B	MBBR with Membrane Filtration	0	No	Install MBBR media and screens and membrane filtration
6C	MBBR with MBR	0	No	Install MBBR media and screens and MBR with screening facility

Solids processing requirements for each Alternative were also discussed and co-thickening of primary and secondary sludges was evaluated as a process alternative. In general, co-thickening when compared with separate sludge thickening, would require larger DAFT units, smaller anaerobic digesters, and less dewatering centrifuges. Table 7 summarizes the solids processing requirements for each Option used in conjunction with Alternative 3 and Alternative 6.

Table 7
Sludge Production Criteria for Alternatives 3 and 6

Item	Sludge Production (lb/d)	Sludge Flow (mgd)	
Thickening Process Flows			
Primary Sludge		Separate Thickening	Co-Thickening
Average Day	65,400	0.177	0.392
Peak Two Week	102,100	0.277	0.612
Peak Day	134,700	0.366	0.808
Secondary Sludge[a]			
Average Day	28,970	2.32	2.32
Peak Two Week	36,440	2.91	2.91
Peak Day	41,320	3.30	3.30
Secondary Sludge[b]			
CAS			
Average Day	22,230	1.88	1.88
Peak Two Week	27,960	2.36	2.36
Peak Day	35,350	2.98	2.98
MBR			
Average Day	7,130	0.13	0.13
Peak Two Week	8,510	0.16	0.16
Peak Day	10,190	0.19	0.19
Total Sludge Flow			
Options 3A & 3B			
Average Day	94,359	-	2.71
Peak Two Week	138,580	-	3.53
Peak Day	176,064	-	4.11
Option 3C			
Average Day	94,749	-	2.40
Peak Two Week	138,610	-	3.13
Peak Day	180,281	-	3.98
Anaerobic Digestion Process Flows			
Primary Sludge			
Average Day			-
Peak Two Week			-
Peak Day			-

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Table 7
Sludge Production Criteria for Alternatives 3 and 6

Item	Sludge Production (lb/d)	Sludge Flow (mgd)	
Anaerobic Digestion Process Flows			
Secondary Sludge- Options A & B			-
Average Day		.069	-
Peak Two Week		.087	-
Peak Day		.098	-
Secondary Sludge – Option C			
Average Day		0.070	-
Peak Two Week		0.087	-
Peak Day		0.108	-
Total Sludge Flow Options 3A & 3B			
Average Day		0.246	0.187
Peak Two Week		0.364	0.274
Peak Day		0.464	0.348
Option 3C			
Average Day		0.247	0.187
Peak Two Week		0.364	0.274
Peak Day		0.464	0.357

[a] Options A and B
[b] Option C

Space Requirements for Alternative 3

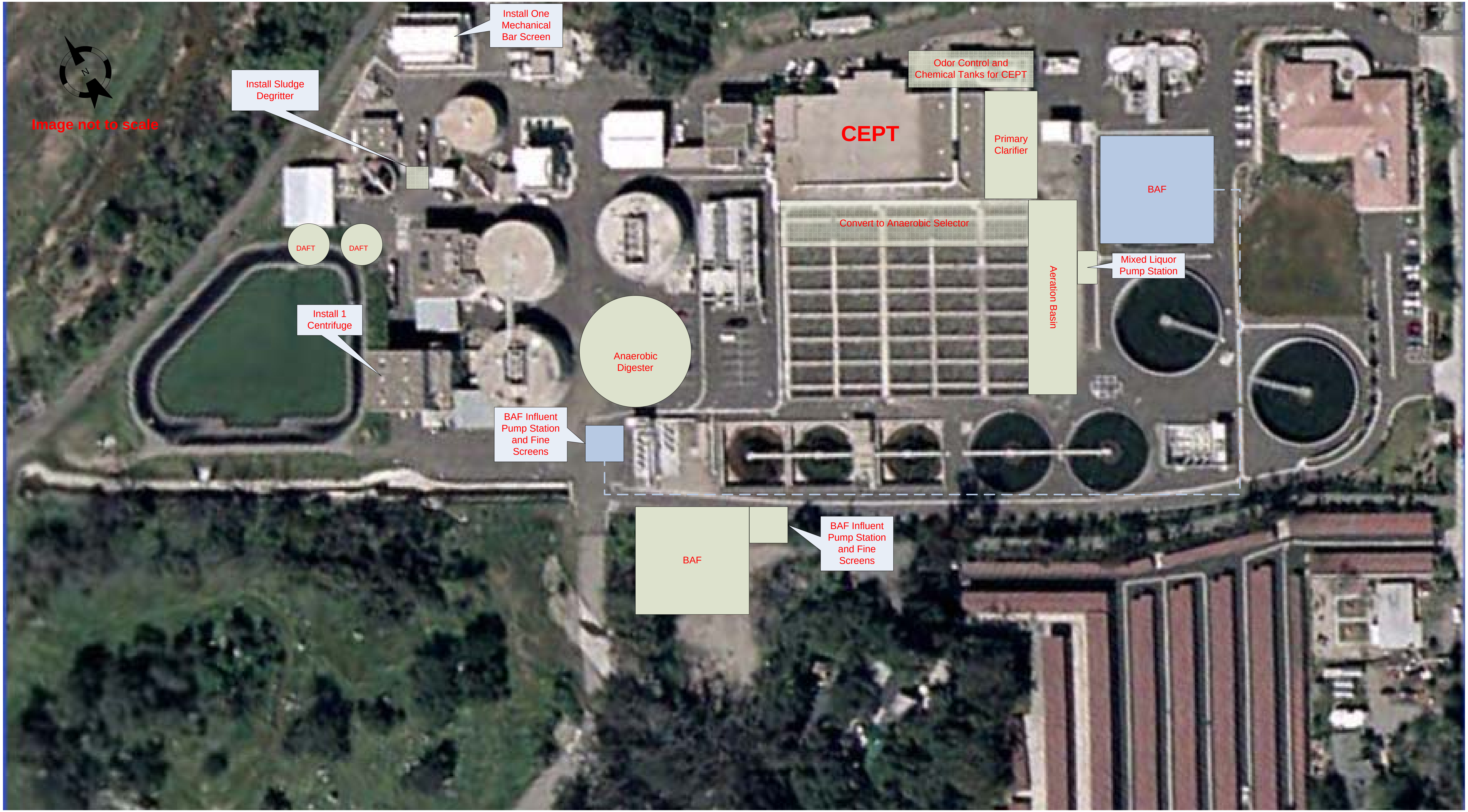
Attachment 1 shows an aerial view of HARRF with the requirements for Alternative 3, Option A assuming co-thickening is used. Attachment 2 shows HARRF for Alternative 3, Options A assuming separate thickening. For Option A, two locations for the BAF equipment were identified. If the BAF were located on site (indicated in blue in Attachments 1 and 2), it would be necessary to pump secondary effluent to the BAF unit, and then pump from the BAF unit to the chlorine contact basins. Locating the BAF equipment adjacent to the plant (indicated in green in Attachments 1 and 2) would require acquisition of approximately 12,320 ft^2 (110 ft X 112 ft) of additional land. However, this option alleviates some of the issues associated with piping and requires less pumping.

For Option B, it is assumed that the existing tertiary filters would be retrofitted with membranes. Attachments 3 and 4 show Option B with co-thickening and with separate thickening, respectively. Attachment 5 shows Option C assuming co-thickening is used. Attachment 6 shows Option C assuming separate sludge thickening.

Space Requirements for Alternative 6

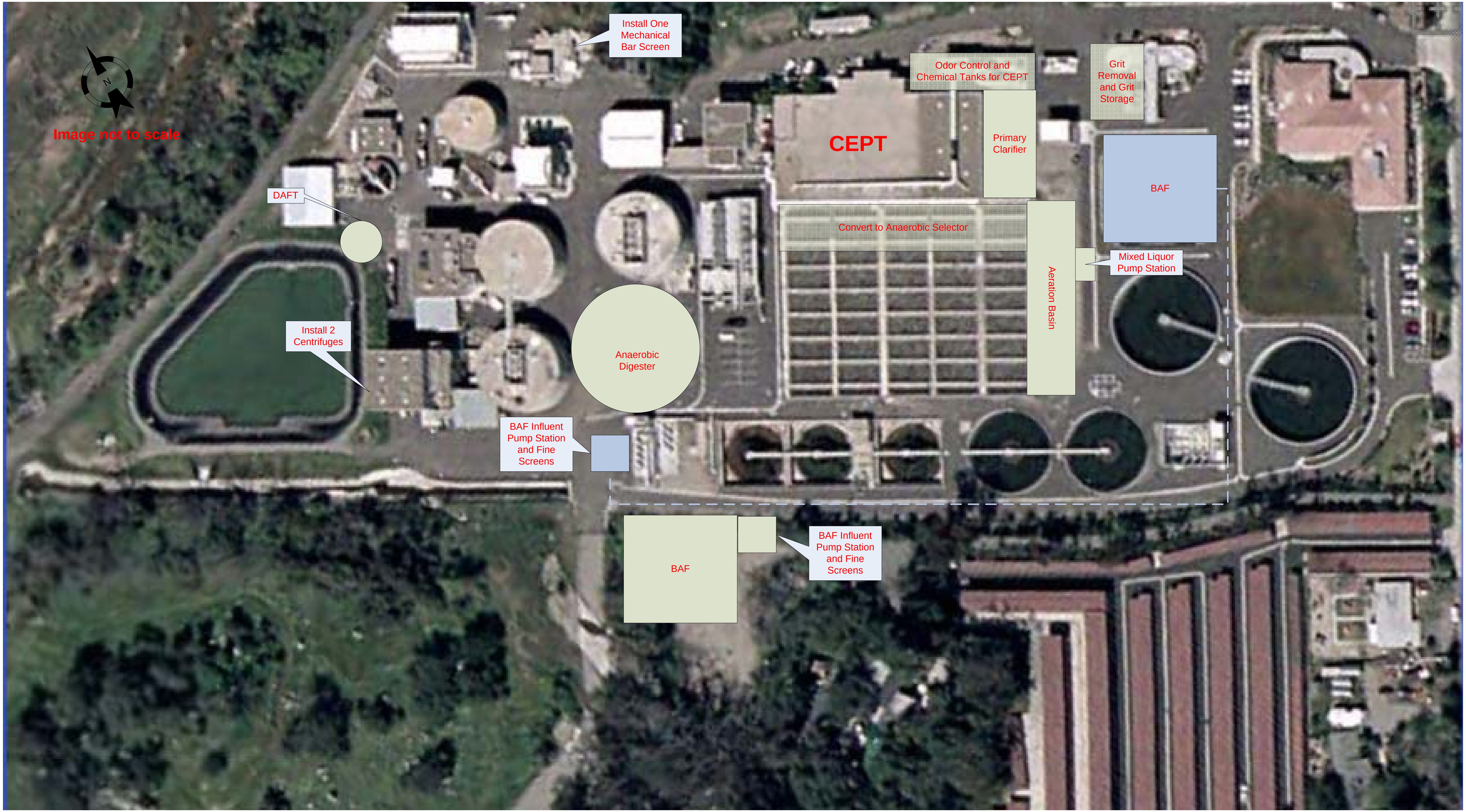
The solids processing requirements for Alternative 6 were assumed to be similar to the requirements for Alternative 3. For production of tertiary water, it was assumed that only Options B and C would be used. Attachment 7 shows an aerial view of HARRF with the requirements for Alternative 6, Option B assuming co-thickening is used. Attachment 8 shows HARRF for Alternative 6, Options B assuming separate thickening. Attachment 9 shows Option C assuming co-thickening is used. Attachment 10 shows Option C assuming separate sludge thickening.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833



Attachment 1. Plant layout for Alternative 3A at Ultimate Buildout – High-Rate Conventional Activated Sludge with BAF and CEPT and Sludge Co-thickening

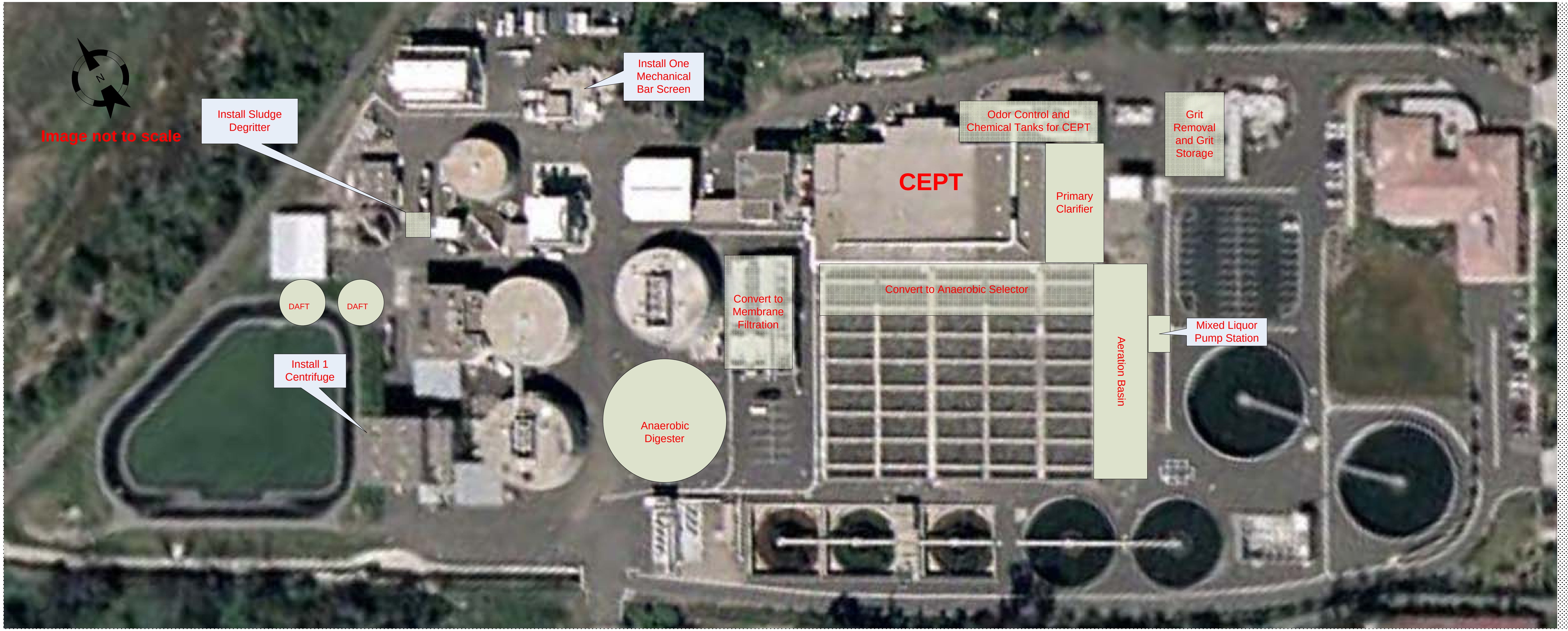
Notes:
1. Layout represents 27.5 mgd average annual daily flow



Attachment 2. Plant layout for Alternative 3A at Ultimate Buildout – High-Rate Conventional Activated Sludge with BAF and CEPT and Separate Sludge Thickening

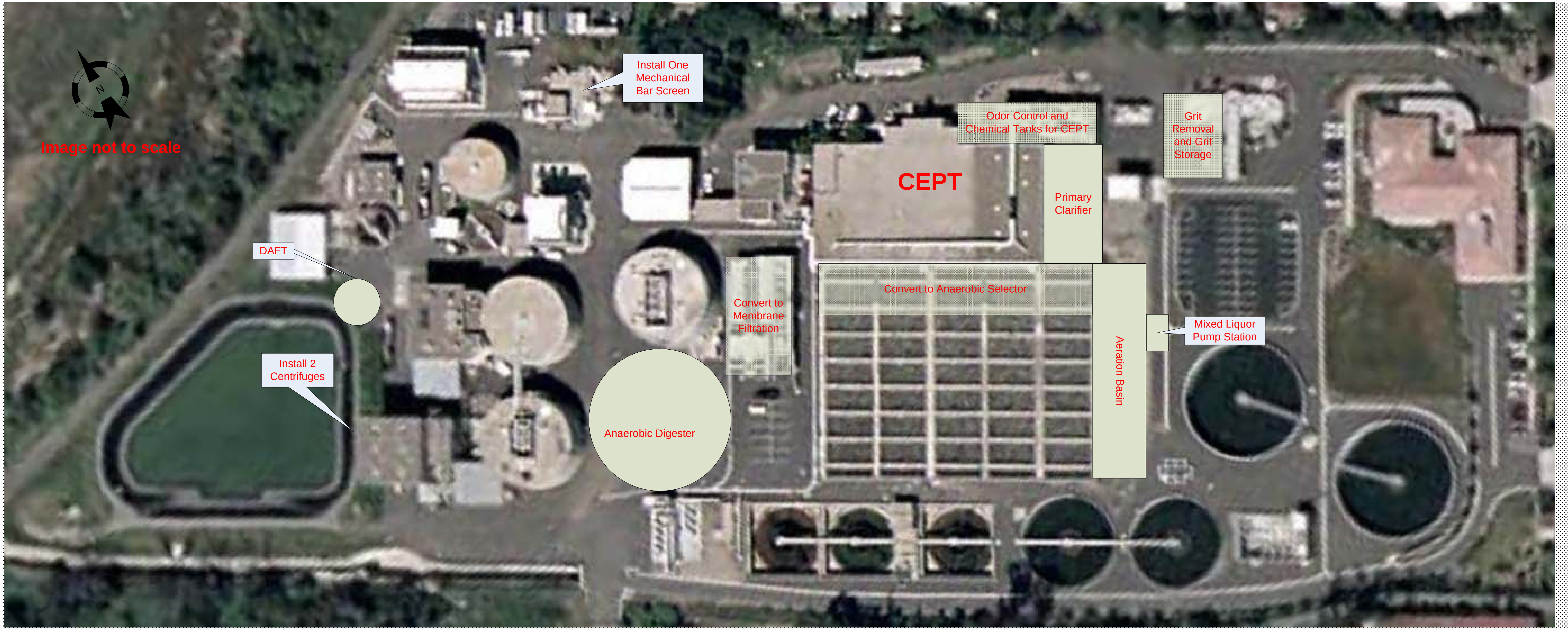
Notes:

1. Layout represents 27.5 mgd average annual daily flow



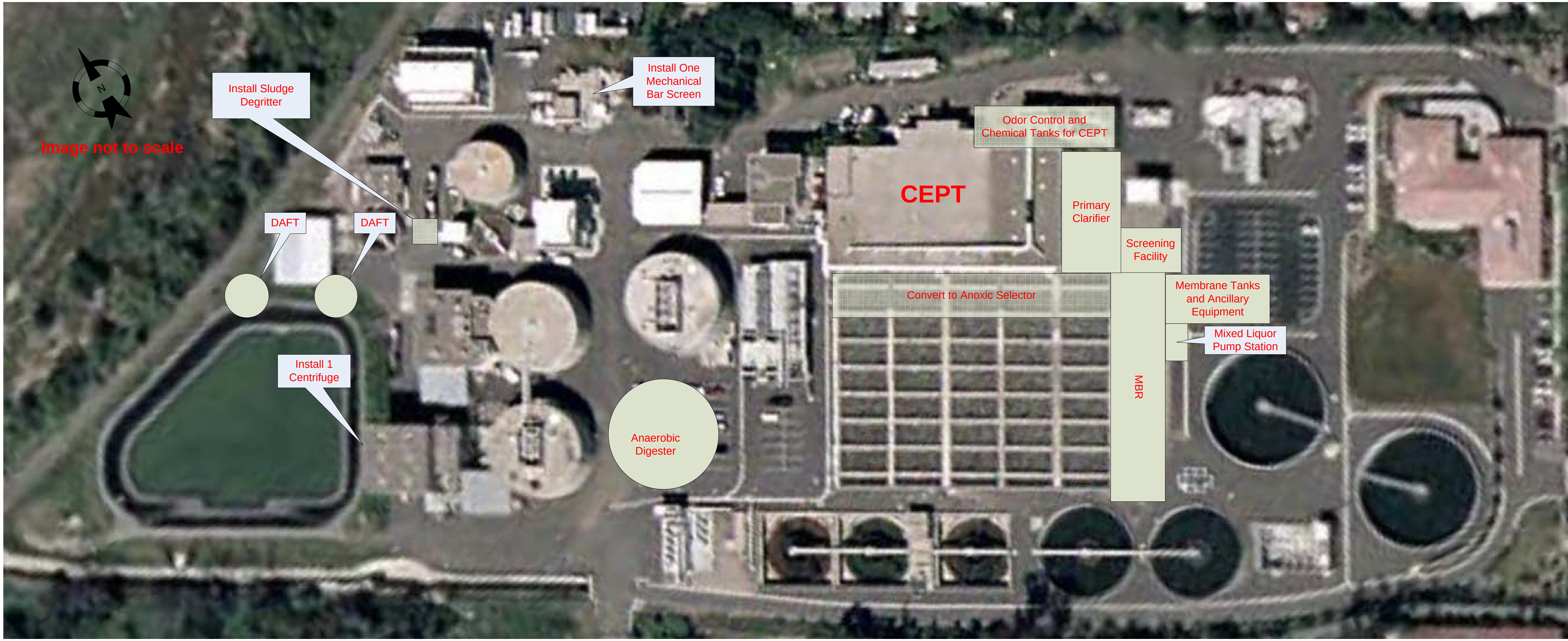
Attachment 3. Plant layout for Alternative 3B at Ultimate Buildout – High-Rate Conventional Activated Sludge with Membrane Filtration and CEPT and Sludge Co-thickening

Notes:
1. Layout represents 27.5 mgd average annual daily flow



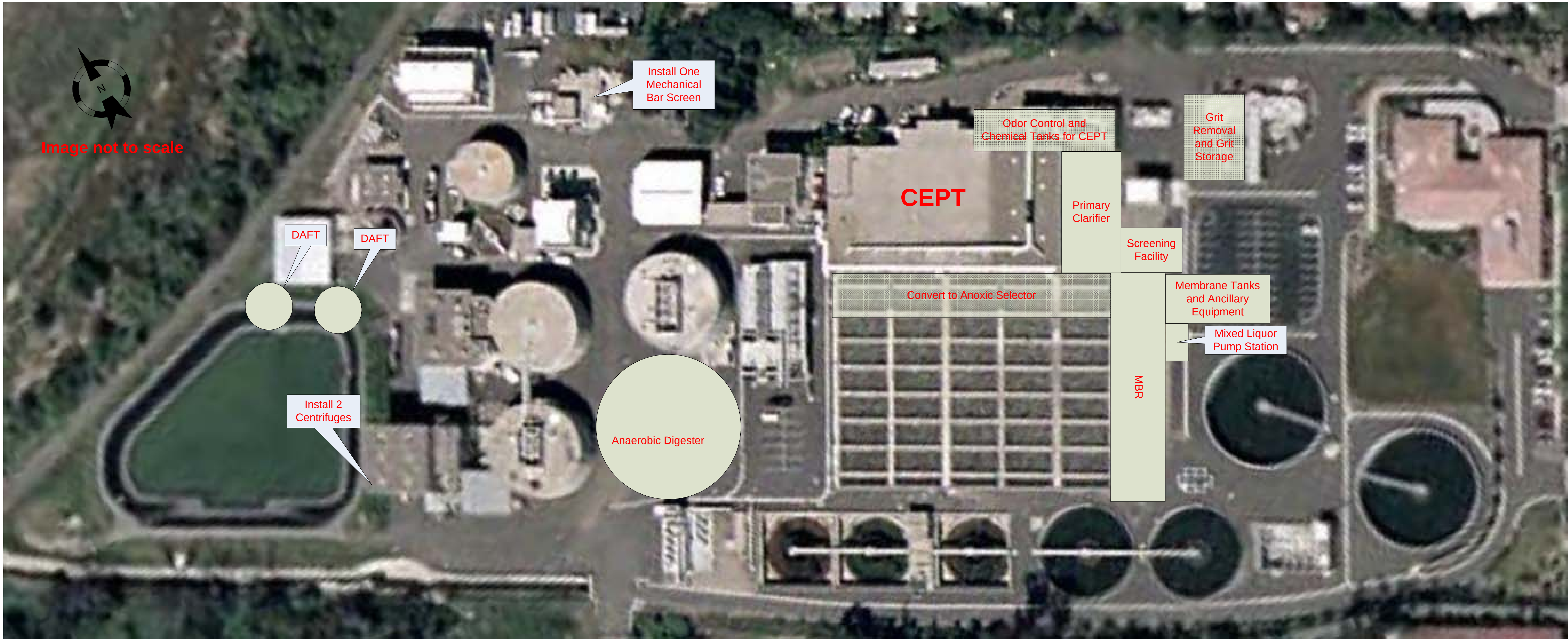
Attachment 4. Plant layout for Alternative 3B at Ultimate Buildout – High-Rate Conventional Activated Sludge with Membrane Filtration and CEPT and Separate Sludge Thickening

Notes:
1. Layout represents 27.5 mgd average annual daily flow



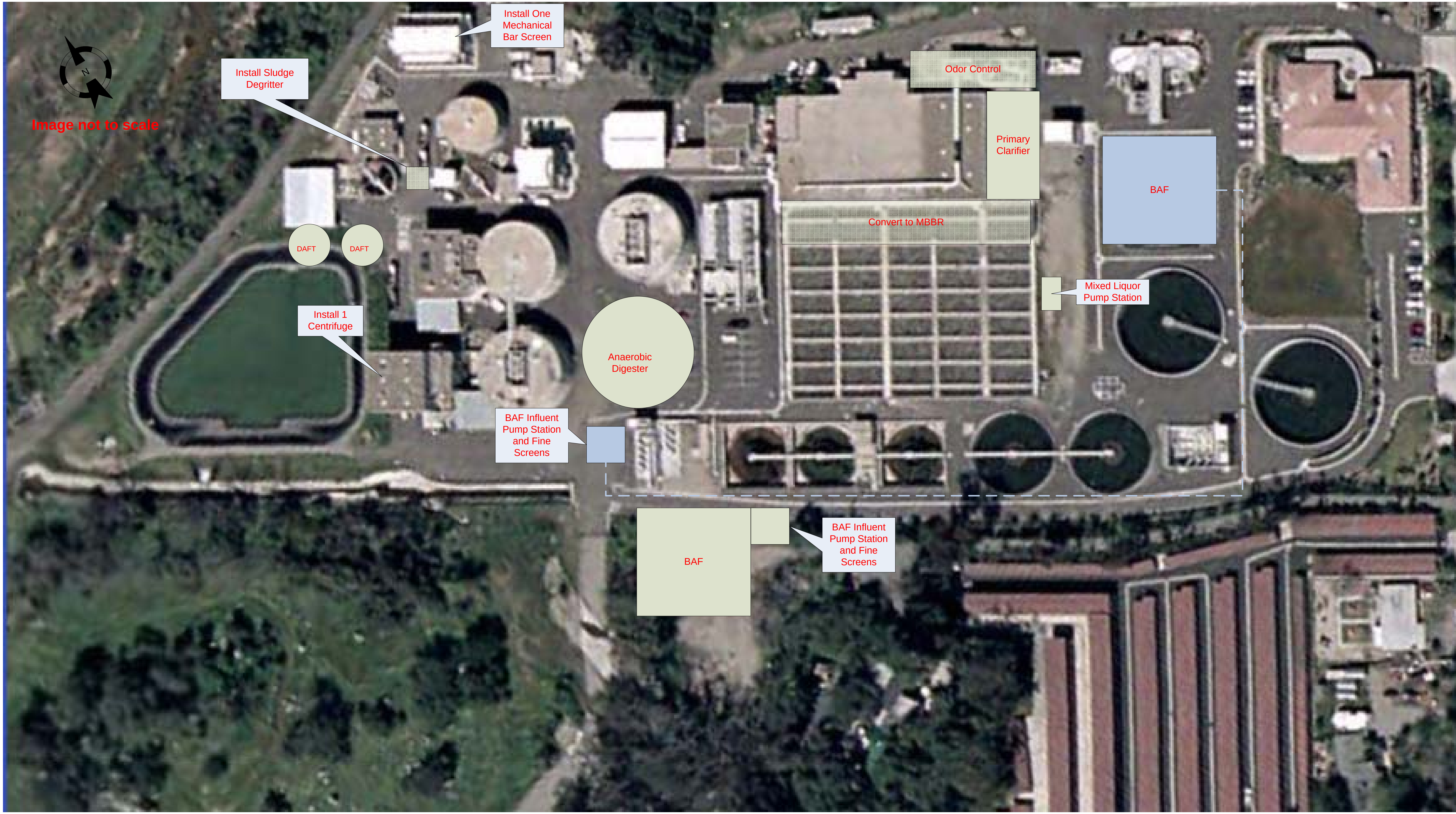
Attachment 5. Plant layout for Alternative 3C at Ultimate Buildout – High-Rate Conventional Activated Sludge with Membrane Bioreactor (MBR) and CEPT and Sludge Co-thickening

Notes:
1. Layout represents 27.5 mgd average annual daily flow



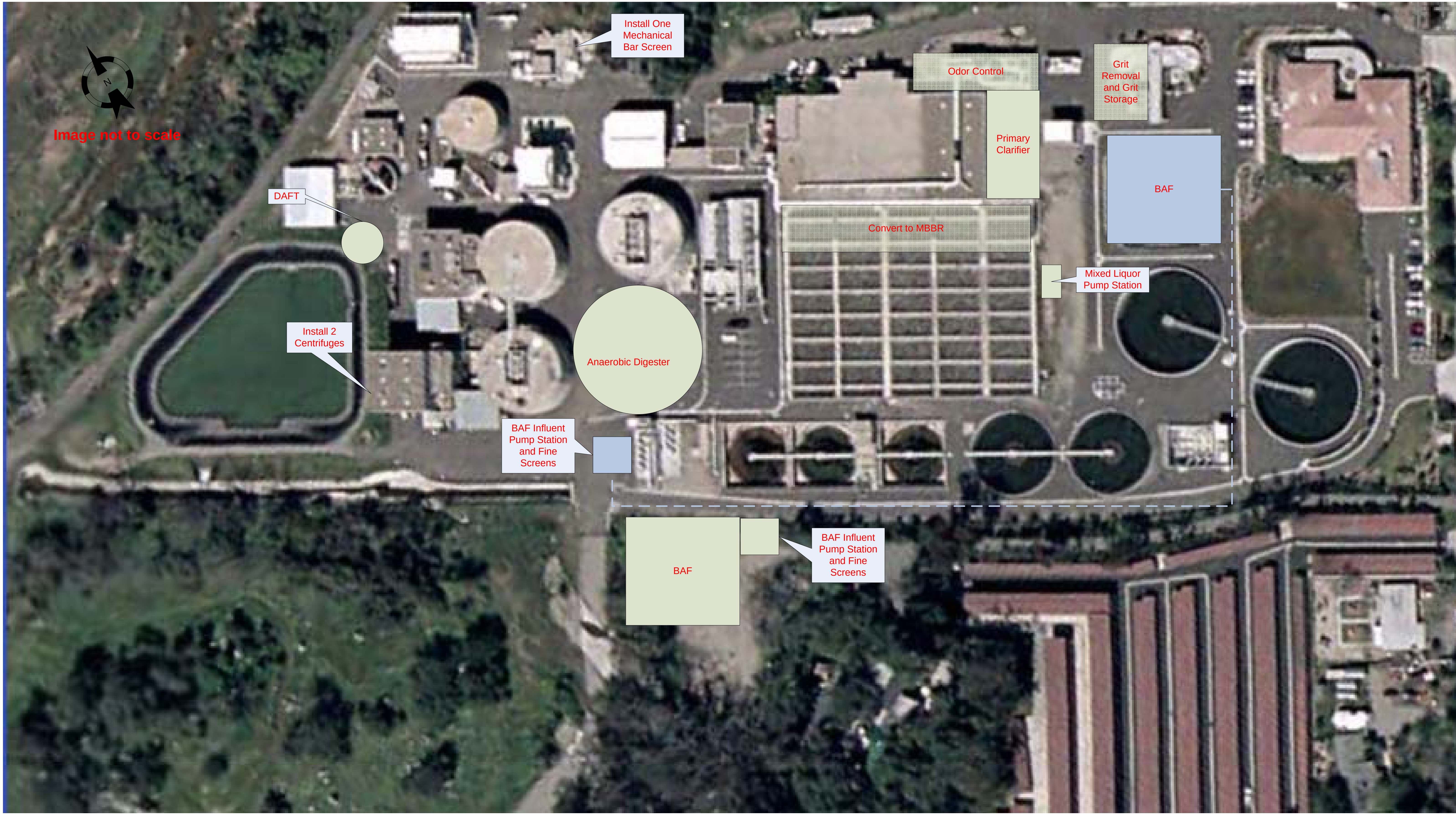
Attachment 6. Plant layout for Alternative 3C at Ultimate Buildout – High-Rate Conventional Activated Sludge with Membrane Bioreactor (MBR) and CEPT and Separate Sludge Thickening

Notes:
1. Layout represents 27.5 mgd average annual daily flow



Attachment 7. Plant layout for Alternative 6A at Ultimate Buildout
Moving Bed Biological Reactor with BAF and Sludge Co-thickening

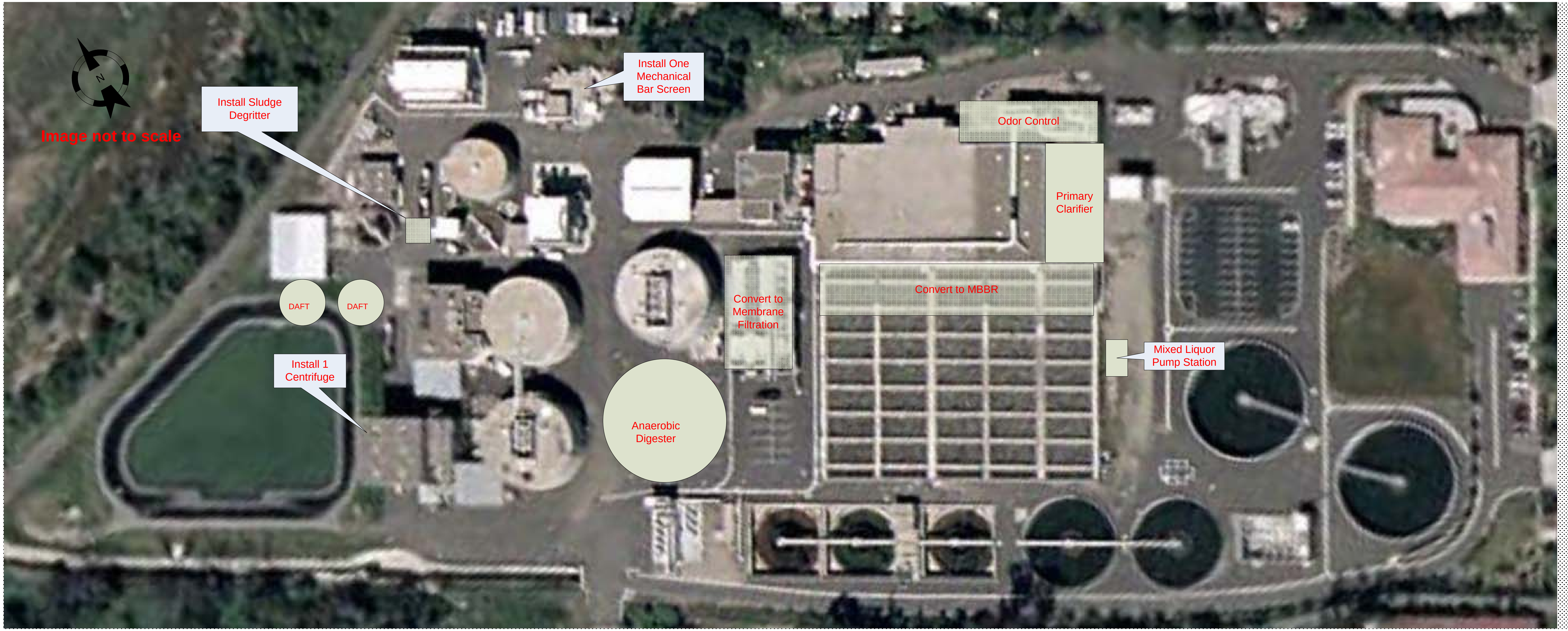
Notes:
1. Layout represents 27.5 mgd average annual daily flow



Attachment 8. Plant layout for Alternative 6A at Ultimate Buildout
Moving Bed Biological Reactor with BAF and Separate Sludge Thickening

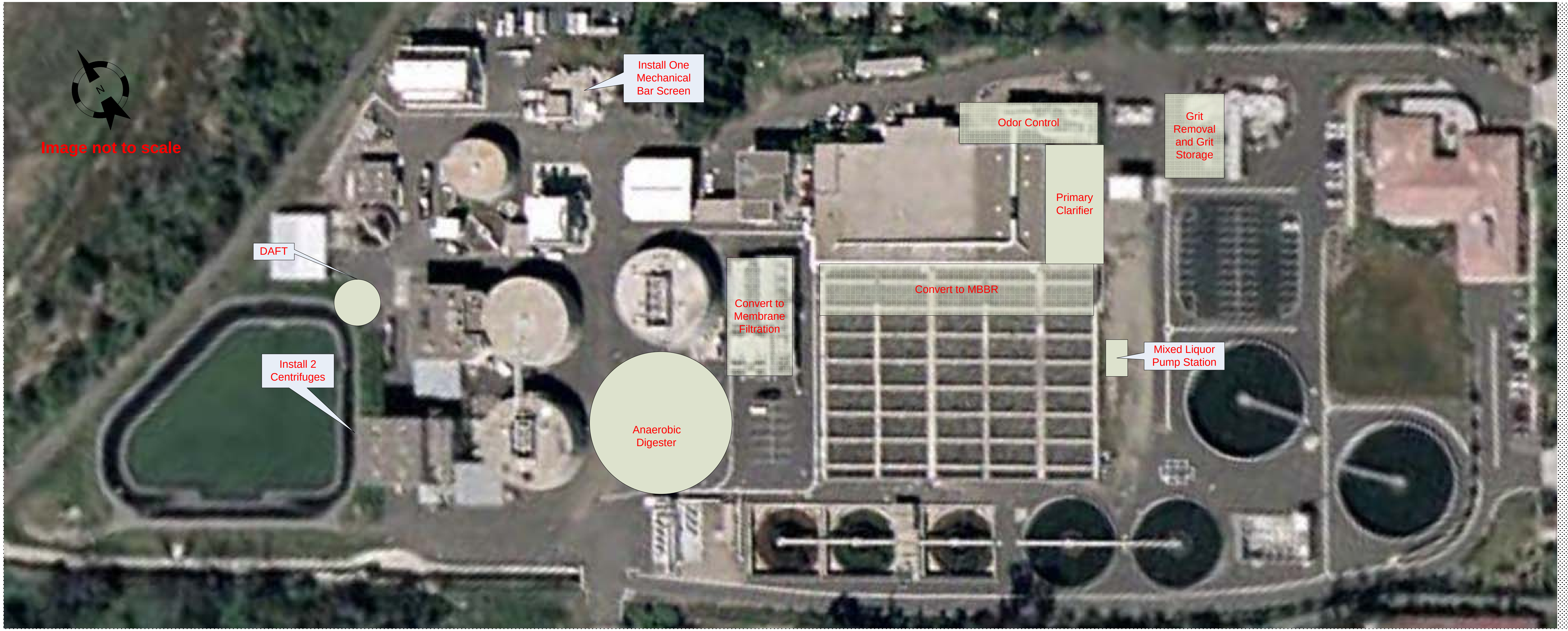
Notes:

1. Layout represents 27.5 mgd average annual daily flow



Attachment 9. Plant layout for Alternative 6B at Ultimate Buildout
Moving Bed Biological Reactor with Membrane Filtration and Sludge Co-thickening

Notes:
1. Layout represents 27.5 mgd average annual daily flow



Attachment 10. Plant layout for Alternative 6B at Ultimate Buildout
Moving Bed Biological Reactor with Membrane Filtration and Separate Sludge Thickening

Notes:
1. Layout represents 27.5 mgd average annual daily flow

APPENDIX I

ESCONDIDO LAND OUTFALL ANALYSIS TM

TECHNICAL MEMORANDUM – FINAL (Revision 2)

DATE:	AUGUST 10, 2006
TO:	ANGELA MORROW, CITY OF ESCONDIDO
FROM:	VICTOR OCCIANO, BROWN AND CALDWELL
PREPARED BY:	ANDREW BALDWIN, BROWN AND CALDWELL
SUBJECT:	CITY OF ESCONDIDO HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – LAND AND OCEAN OUTFALL CAPACITY ANALYSIS

SUMMARY

The hydraulic capacities of the land and ocean outfalls are summarized as follows:

- Land outfall (gravity section) 23.7 mgd
- Land outfall (pressurized section) 21.4 mgd
- Ocean Outfall 25.8 mgd

The capacities of the gravity and pressurized sections of the land outfall result from different hydraulic behavior. The gravity section is limited by 'throttle' pipes restricting the flow resulting in localized spills. Further analysis and model tests demonstrate the capacity could be increased to 25.2 mgd if the siphon inlet/outlet manholes are sealed. Regarding the pressurized section, capacity improvements and spill reductions can be achieved by sealing all manholes downstream of Manhole 69.

The ocean outfall hydraulic capacity is limited by the pressure rating of the RCPP pipe under the shoreline. Minimal improvements may be obtained by improving the operational logic of the regulator value by accounting for variable tide levels. However, significant capacity improvements will only be obtained by constructing a new ocean outfall within the constraints of the ocean discharge permit.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

1. INTRODUCTION

1.1 Purpose

The purpose of this document is to describe the hydraulic analysis of the land and ocean outfalls. The hydraulic analysis used a hydraulic model to predict existing and future hydraulic problems such as spills, pipe throttles, etc. The document describes the building and calibration of the hydraulic model, the development of existing and future flow scenarios, the prediction of existing and future problems, and the development of capacity improvements designed to alleviate immediate hydraulic problems.

1.2 Task Scope

The key objectives of the hydraulic analysis task are to assess the hydraulic capacity of the existing land and ocean outfalls, and create a hydraulic model for supporting the planning and design of future improvements. As the outfall consist of both gravity and pressurized sections, the hydraulic capacity of each section differs based on the flow conditions and operating policies of the regulator structure managed by San Elijo Joint Powers Authority (SEJPA). The goals of this task are summarized below:

- Build, calibrate and use a hydraulic model for:
    - Predicting existing outfall capacities
    - Developing capacity improvements
    - Evaluating storage and disposal strategies

- Identify existing hydraulic capacities for:
    - Gravity section
    - Siphons
    - Pressurized section (upstream of the regulator structure)
    - Ocean outfall section (regulator structure to diffuser)
- Identify minimum and maximum pressures in the pressurized sections

This report includes the analyses of the existing land and ocean outfalls providing an understanding of the hydraulic limitations and operational practices.

1.3 References

The following documents have been used during the work conducted for this task and development of this document.

1. InfoWorks Documentation v6.5 (Wallingford Software)
2. Ocean Outfall Study (Tetra Tech, November 2001)
3. Land Outfall Study (Tetra Tech, October 2001)
4. Land Outfall Study (JMM, February 1990)

2. MODEL DEVELOPMENT

This section describes the development of the hydraulic model of the land and ocean outfall, and the HARRF. The hydraulic conditions throughout the plant and outfall comprise of both gravity and pressurized (surcharged) flow conditions. These conditions along with the complex control structures (ie; the San Elijo regulator structure) required a robust hydraulic model capable of predicting current and future flows. The model selected for the project was InfoWorks.

InfoWorks is a dynamic model developed in England by Wallingford Software and distributed throughout the U.S. InfoWorks is widely used in Europe and is now also being used by many agencies and consultants in the U.S. The model is fully dynamic and uses an implicit computation scheme, that is extremely stable, to solve the Saint-Venant equations. The current version of InfoWorks has modules for the simulation of water quality, real time control, and sediment transport. A versatile flow generation routine is integrated into the model.

InfoWorks adds a GIS-based data management system to the hydraulic engine. This product adds the capability to graphically build and edit models, simplify networks, and display results along with other map layers in a GIS environment. Plan views may be set up to show location of flooding, surcharging, storage and overflow volumes, and flow direction. InfoWorks can import data from other sources, and model simulation results may also be exported for graphic and data post-processing.

2.1 Land Outfall Model

The InfoWorks hydraulic model comprises of nodes and links that represent 'asset-based' features such as pipes and manholes. The data required to build the hydraulic model of the land outfall was obtained from GIS data and as-built construction drawings.

The location of the pipes and manholes were extracted from the GIS layers provided by the City's GIS department. The data was imported into InfoWorks and used as basis for creating the model network. The pipe and manhole attribute data (eg; inverts, ground elevations and sizes) were extracted from the as-built drawings and manually entered into model database.

In order to accurately represent the profile through the siphons, 'dummy' nodes were added to the model network. See Figure 1 for an example siphon profile. The dummy nodes were 'sealed' in the model to ensure no flow escapes from the pressurized sections. Additional dummy nodes were added to the pressurized section downstream of Manhole 74 to ensure the maximum modeled pipe lengths were not exceeded. All manhole nodes, including the air-vacs in the pressurized section were sealed. All elevations used in the model correspond to the elevations and vertical datum (NGVD29) defined in the as-built drawings.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

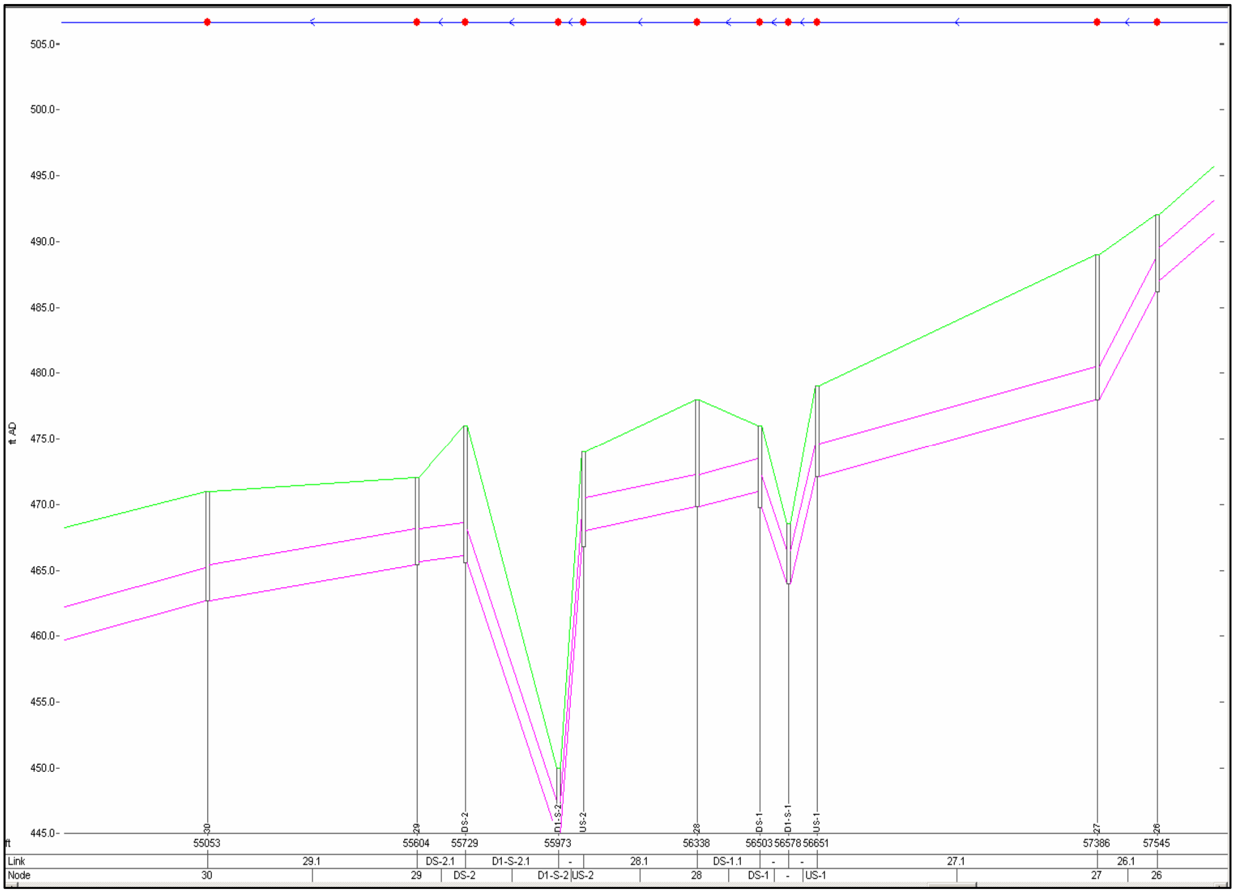


Figure 1. Example siphon profile

Following the data entry task, the data was validated using built-in data checking rules plus detailed inspections of the pipe profile. Pipe invert 'steps' were identified from the as-built plans and added to the model network. Finally, the hydraulic roughness (ie; Mannings, n = 0.013) was applied to each pipe segment.

2.2 Ocean Outfall Model

The ocean outfall comprises of the pressurized pipe segments from the regulator structure at the San Elijo Treatment Plant to the outfall diffuser. The elevation ranges from +8 feet above sea level to -139 feet below sea level.

As no GIS data was available, the spatial location of the ocean outfall was digitized from the original as-built plans obtained from San Elijo JPA (SEJPA). Elevations, pipe sizes and lengths were also extracted and manually entered into the InfoWorks model database. The nodes used to link the modeled pipe segments were sealed to replicate the pressurized flow conditions. The outfall diffuser comprises of a single 48 inch pipe section with up to 200 linearly distributed port openings. Based on a review of a recent ocean outfall inspection study conducted by SEJPA, approximately 100 diffuser ports are open and have been cleaned. In addition to modeling the current diffuser status, the study also examined the capacity increases if all 200 ports were open.

The diffuser headlosses were derived using a specialized modeling package and were represented in InfoWorks as an equivalent orifice. The graph in Figure 2 shows the relationship between headloss and flow through the diffuser with 100 and 200 port openings. The graph also depicts the headloss curve for the equivalent orifice.

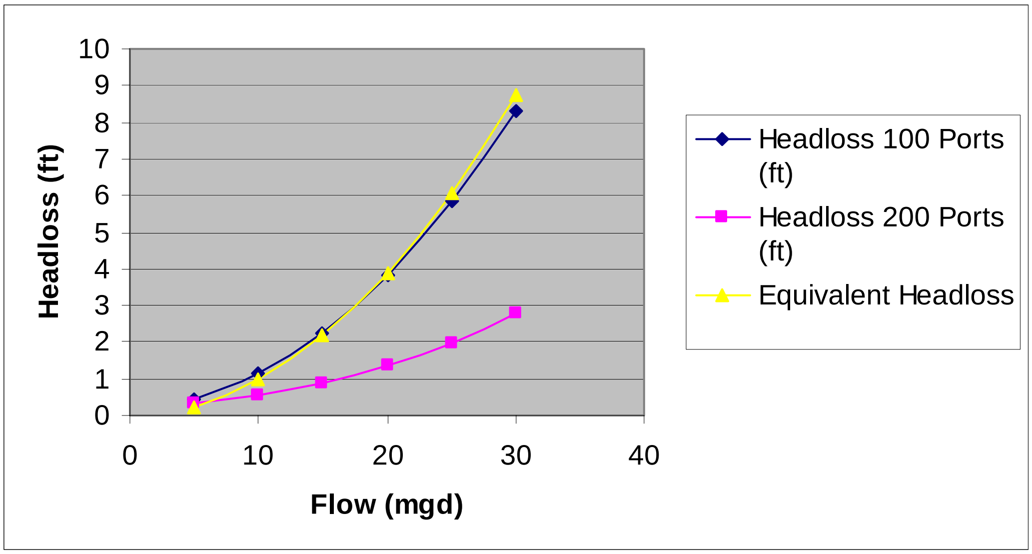


Figure 2.Ocean outfall diffuser headlosses.

2.3 Regulator Structure Model

The regulator structure is located at the junction of the land outfall, ocean outfall and the SEJPA plant effluent. The structure is designed to limit the flow entering the ocean outfall in order to prevent flows and pressures exceeding the allowable design limitations in the ocean outfall. In addition, the structure is used to 'shave' and divert peak flows to the SEJPA reclaimed water facility.

The regulator structure limits the flow from the land outfall by 'pinching' the flow using an automatically controlled valve located upstream of the ocean outfall and SEJPA effluent line. The control valve is duplicated with a parallel valve and pipe configuration used during maintenance periods. This study assumed one control valve and associated piping were active during the calibration period and capacity analysis period.

The model of the regulator structure comprises of the pipe segments and control links representing valves, venture meters and reducers. Appropriate discharge coefficients and headloss factors (ie; 'k') were assigned to the control links to model the total headloss throughout the regulator structure. Figure 3 shows the layout of the regulator structure model.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

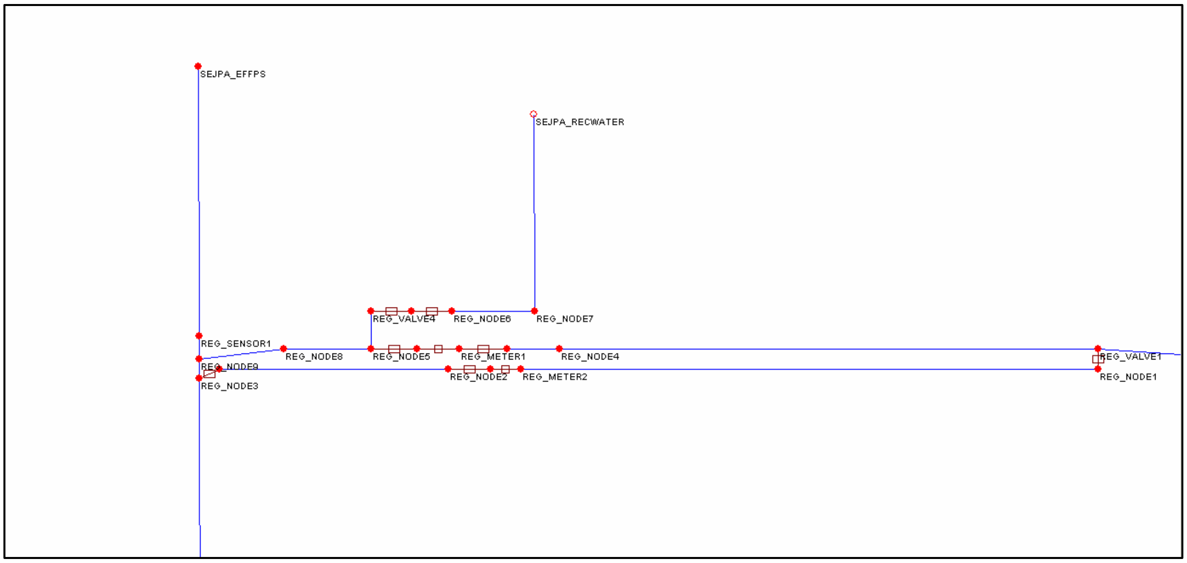


Figure 3. Regulator structure model layout.

2.4 HARRF Plant Model

The process equipment contained at HARRF impact the hydraulic characteristics of the influent flow resulting in an attenuated effluent flow. The attenuated flow occurs when the secondary effluent is routed through control structures (eg; the Secondary Effluent Pump Station) and storage units such as the equalization basin resulting in 'shaved' peak flows. The peak shaving is also controlled by the plant operators to ensure the final effluent flow does not exceed the outfall capacity.

The core process equipment and associated pipe work were added to the InfoWorks model. The layout of the components were digitized into the model via the use of background orthographic image data. (See Figure 4 for plant model layout). The physical data such as elevations, lengths, sizes etc were extracted from as-built drawings depicting the existing and upgraded facilities.

InfoWorks has the ability to simulate operational rules used to control regulators, valves and pumps. This feature, referred to as 'real-time-control', models set-points, control rules and pump switch on/off rules within the hydraulic model. The following process rules and control settings were programmed into the hydraulic model:

- Total RAS flow entering the RAS/WAS pump station equals 40% of plant influent flow.
- 21% of total RAS flow is diverted from Secondary Clarifier 1 into the RAS/WAS PS.
- 21% of total RAS flow is diverted from Secondary Clarifier 2 into the RAS/WAS PS.
- 29% of total RAS flow is diverted from Secondary Clarifier 3 into the RAS/WAS PS.
- 29% of total RAS flow is diverted from Secondary Clarifier 4 into the RAS/WAS PS.
- 89% of total flow from the RAS/WAS PS is routed back to the aeration basins.
- 89% returned flow (to influent PS) from DAF sludge processing.
- 0.64% primary sludge flow pumped from the primary clarifiers.
- Outfall flow limited to 20 mgd (controlled via variable weir at SEPS).

- Flow from equalization basin drained into SEPS when outfall flow is less than 20mgd.

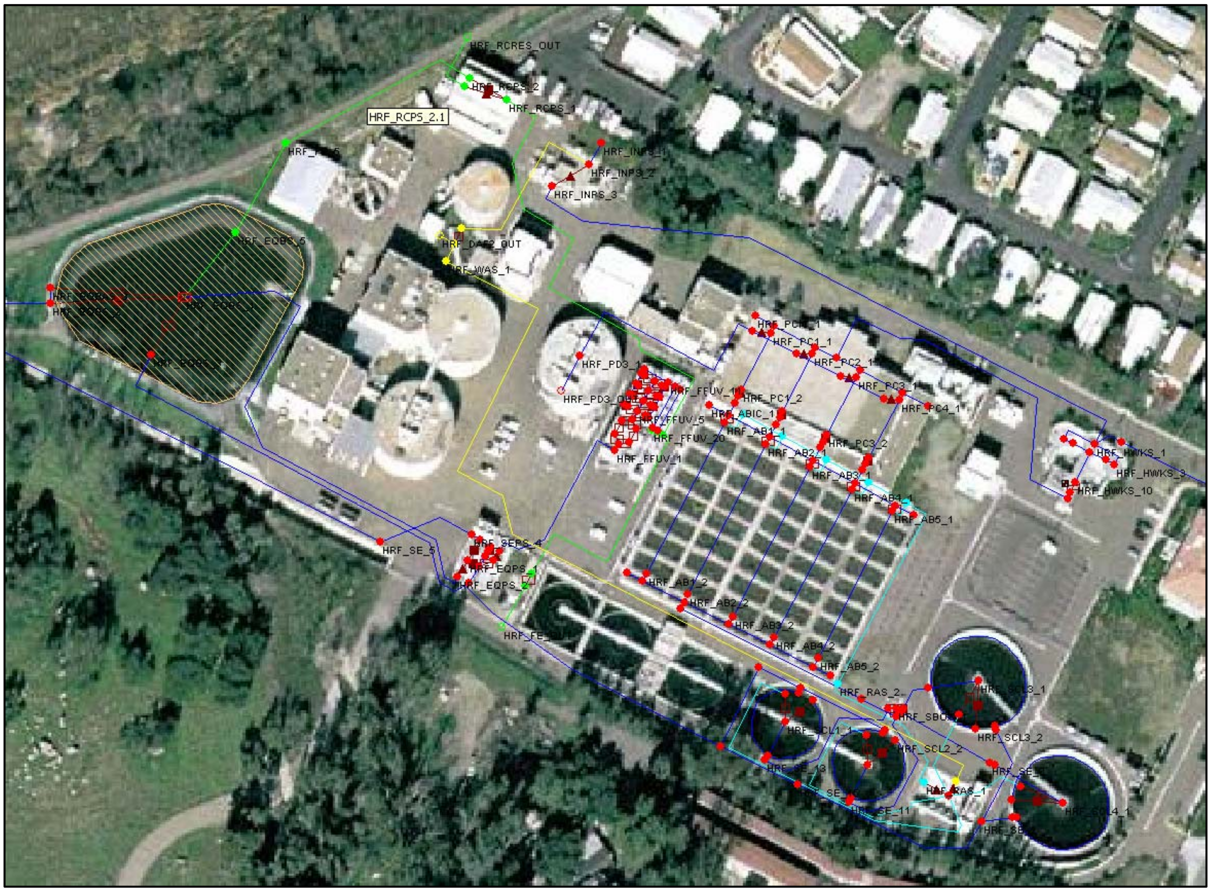


Figure 4. HARRF hydraulic model layout.

3. MODEL CALIBRATION

Model calibration is the process of comparing predicted model results with observed flow data to ensure the model simulates 'real-life' hydraulic behavior. The task involves comparing flow hydrographs of predicted and observed flow/depth data at key locations and validating hydraulic pressures and spill events with historical based observations.

The hydraulic model was firstly calibrated for HARRF using the effluent data collected from the temporary flow meter installed at Manhole 1 on the land outfall. The HARRF influent data was obtained from the flume located at the head-works. The outfall model was calibrated against flow and pressure data located at the regulator structure. The observed flow data obtained at Manhole 1 was used as inflow data during the outfall calibration phase. Additional inflow data included SEJPA effluent data which enters the ocean outfall downstream of the regulator structure.

3.1 Calibration Events

The model was calibrated for both dry and wet weather flow conditions to ensure both daily diurnal and wet weather peak flows are represented accurately. The calibration events were selected from the flow monitoring period (Feb 16th to March 28th 2006) and are listed in Table 1. Note the start date for the dry weather event was selected to match the available data obtained from SEJPA.

Table 1. Calibration Events

ID	Name	Start	End	Duration
1	DWF Calibration	03/06/06 7:00	03/08/06 7:00	2 days
2	WWF Calibration	03/09/06 0:00	03/17/06 0:00	8 days

3.2 Observed Meter Data

Observed flow data was obtained from a temporary flow metering study conducted by mgd, influent flow data obtained from the City, and flow and pressure data obtained from SEJPA's SCADA system. Table 2 lists the properties and uses for each flow meter.

Table 2. Observed Meter Sites

ID	Name	Equipment	Location	Source	Comments
1	Flume Influent Meter	SCADA	Head-works	City	Inflow to HARRF model
2	HARRF Effluent Meter	ADFM Meter	Manhole 1	mgd	Calibrate HARRF model
3	Regulator Flow Meter	SCADA	Regulator structure	SEJPA	Calibrate outfall model
4	Regulator Pressure Meter	SCADA	Regulator structure	SEJPA	Calibrate outfall model

3.3 HARRF Calibration Results

The hydraulic model of HARRF was used to predict dry and wet weather flows for the calibration period (March 2006). Influent data collected from the SCADA system was entered into the model at the head-works and routed through the model to predict effluent hydrographs at the outfall and level hydrographs in the equalization basin. The effluent hydrographs were compared with the observed flow data for both dry and wet weather periods. Figures 5 and 6 show the final calibration model fits for dry and wet weather flows respectively.

The dry weather hydrograph (Figure 5) shows the model results (red line) closely matching the observed flow data (blue line). These results demonstrate the model's ability to accurately predict the plant's hydraulic effect of the treatment processes. However, daily flow spikes observed in the effluent flow data occurring at approximately 1:00am were not predicted by the hydraulic model. Although these flow spikes will not effect the core use of the model (ie; to evaluate storage options), further investigation is recommended in order to determine the source.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

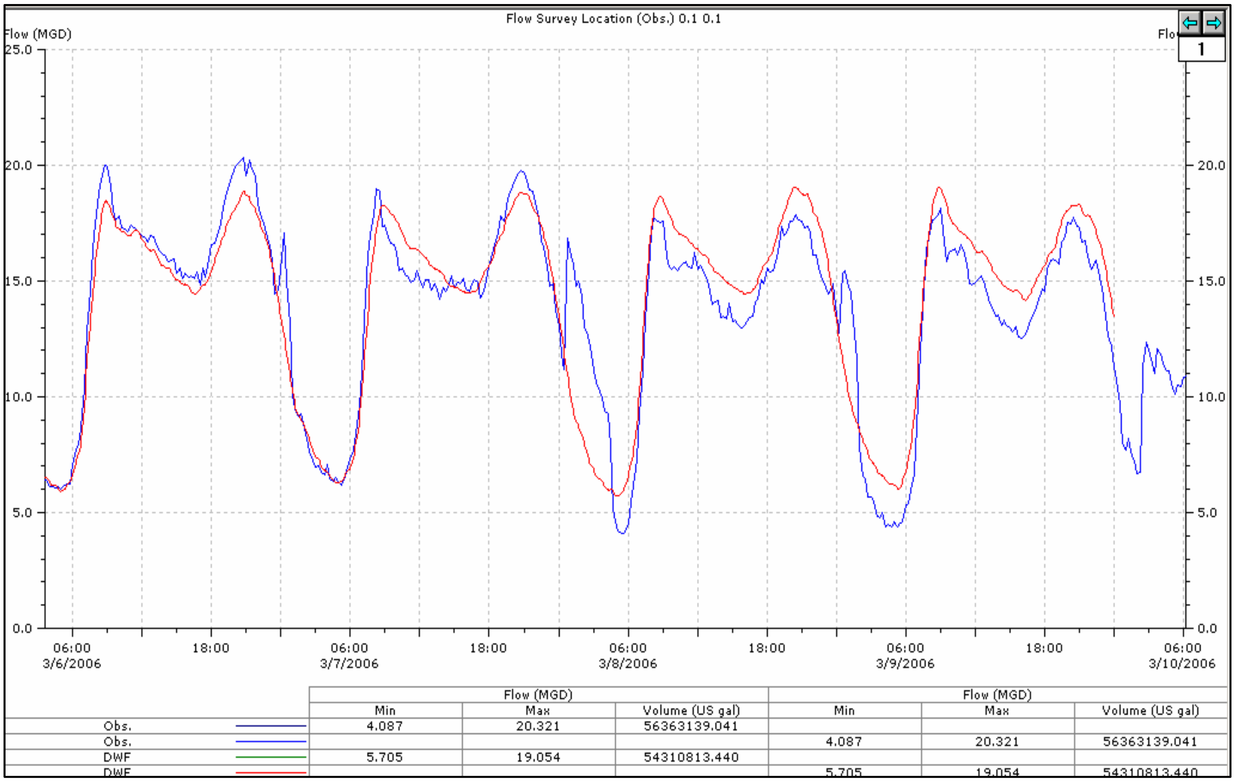


Figure 5. HARRF model vs. observed effluent flow – dry weather flow.

The wet weather flow hydrograph (Figure 6) shows the model results (red line) overlaid on top of the observed effluent data (green line). In addition, the rainfall data is plotted on the graph to indicate the relation between rainfall and wet weather flows. Following adjustment of the operational logic of the effluent pump station and control weir (SEPS), the model accurately predicts wet weather flows through the plant processes. The flow set-point used to limit the flow through the outfall was set to 20.0 mgd, which differs to the set-point (18.5 mgd) observed from the plant's operation center. The difference may be due to the accuracy of the meter used to monitor the flow discharged over the variable weir at SEPS.

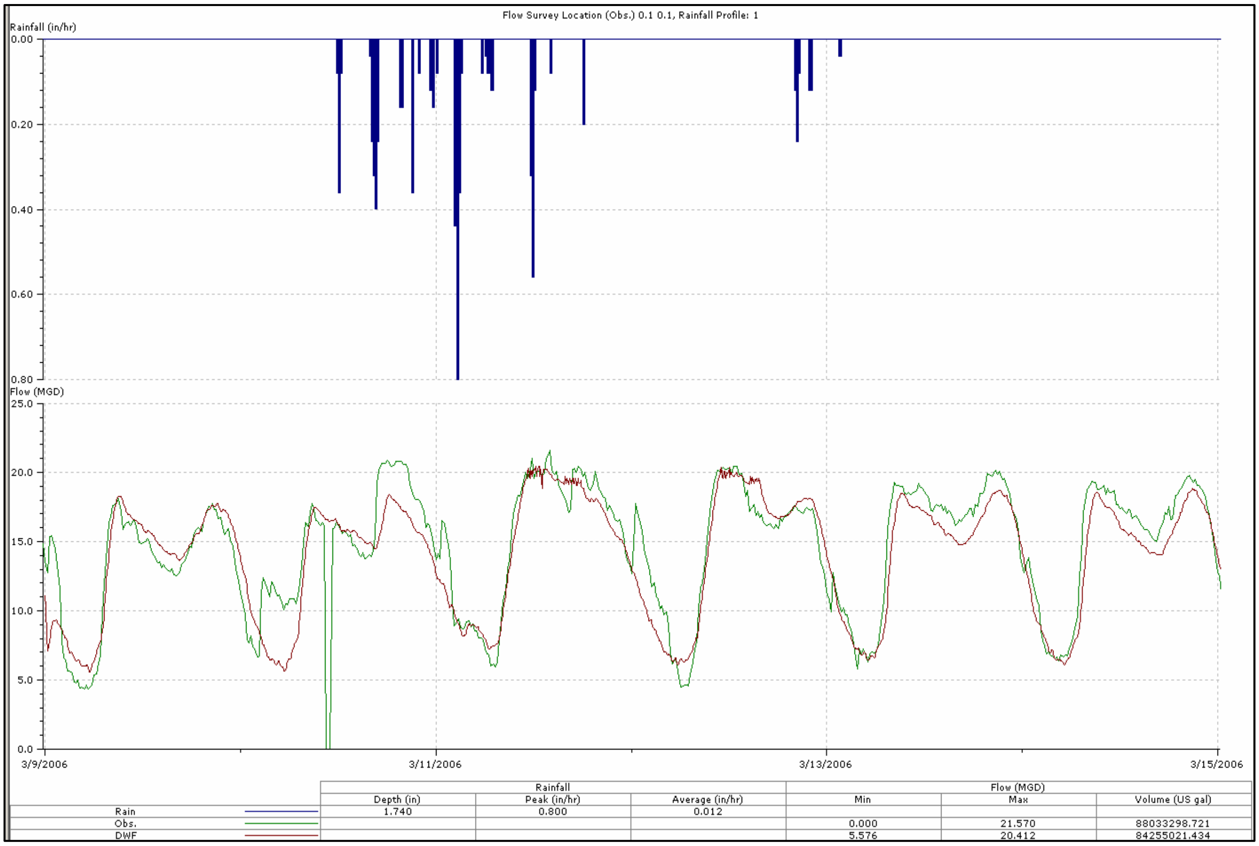


Figure 6. HARRF model vs. observed effluent flow – wet weather flow.

3.4 Outfall Calibration Results

The hydraulic model of the land and ocean outfalls was used to predict dry weather flows and compare against observed flow and pressure data at the regulator structure. The dry weather flow data was obtained from the SCADA system, managed by SEJPA, to control the regulator. Wet weather flows data from SEJPA was not available hence the current outfall model is only calibrated to dry weather conditions.

The flows and depths predicted by the hydraulic model as depicted by the red line in Figure 7 closely match the observed flows and depths at the regulator structure. The model hydrographs display a 'smoothed' response relative the 'peaky' observed data. This minor discrepancy results from the model over estimating in-line storage within the pressurized pipe sections. This is a limitation of the computational algorithm and cannot be adjusted by the user. However, the critical peak responses match well supporting the validity and accuracy of the hydraulic model.

At this stage of the analysis, the modeling effort focused on verifying the operation logic used to control the regulator structure. Following discussions with SEJPA, the control value begins to 'pinch' when the pressure in the ocean outfall exceeds 82 feet at the SEJPA effluent connection. A previous ocean outfall study (Reference 2) conducted for SEJPA, verifies the operating set-point to be 82 feet, or 89 feet HGL. Further investigation is recommended to verify the current operational logic of the regulator and obtain wet weather flow/depth data from SEJPA to finalize the model calibration.

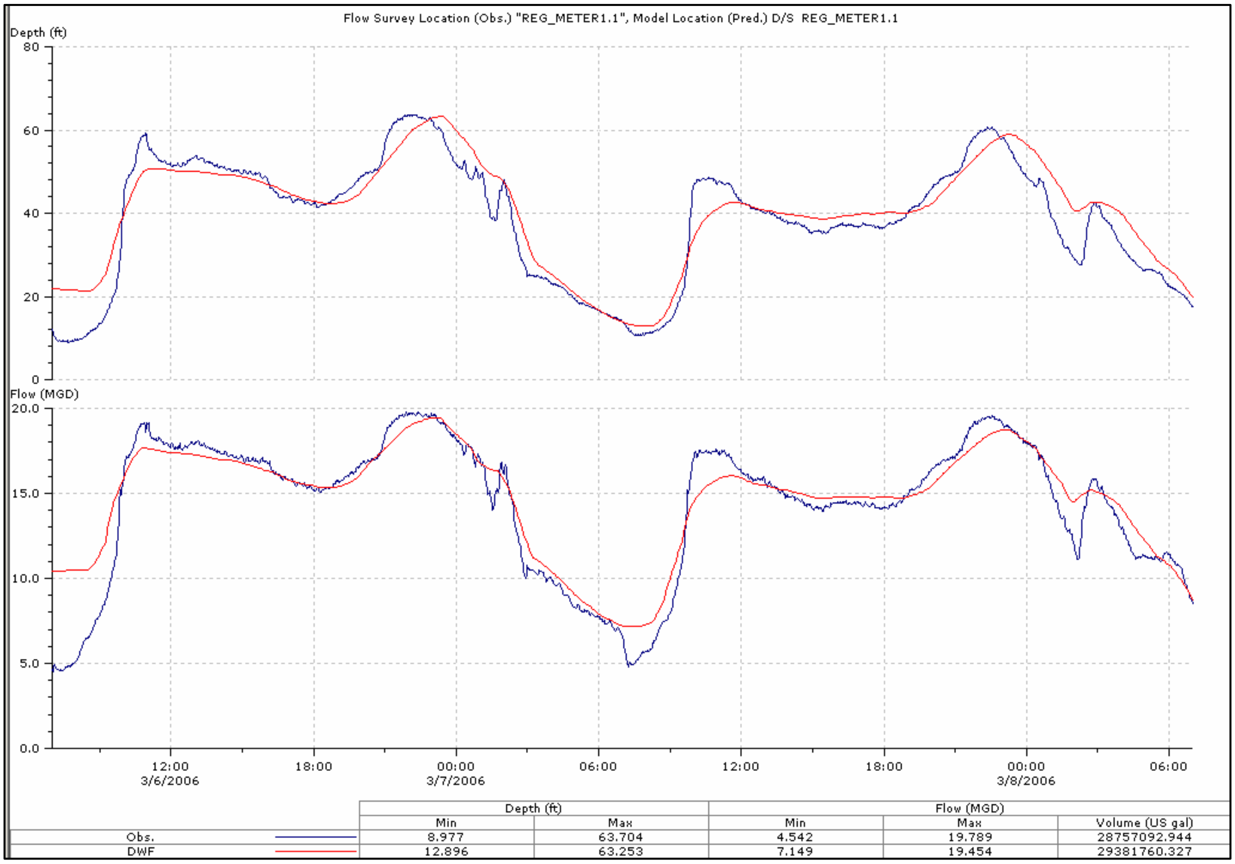


Figure 7. Land outfall model vs. observed flow / depth data – dry weather flow

4. OUTFALL HYDRAULIC ANALYSIS

The capacities of the land and ocean outfalls were evaluated by gradually increasing the effluent into the outfall up to a failure scenario such as a spill or exceeding pressure rating. This 'stress- test' approach identifies a maximum capacity of the existing outfall. The analysis was conducted for the land outfall (gravity and pressurized sections) and the ocean outfall. The following sections describe the analysis and findings.

4.1 Land Outfall – Gravity Section

The land outfall gravity section (from HARRF to Manhole 74) comprises of 73 pipe segments and 6 siphon structures. All manholes throughout the gravity section are un-sealed, hence allowing flows to spill when the hydraulic head reaches the ground elevation. Manholes upstream and downstream of the siphon structures are bolted to the manhole casing but not hydraulically sealed. Further inspection of these manholes local to the siphon structures is recommended to validate these assumptions.

The hydraulic capacity of the gravity section was determined by gradually increasing the flow entering the outfall at HARRF until spills were observed in the hydraulic profile. The maximum capacity of the outfall is the flow observed at the onset of the first spill.

The hydraulic model assumes all flow is 'lost' at a spill manhole, resulting in a constant head equal to the difference between ground and invert elevations. The resulting maximum head limits the discharge downstream of the spill manhole. This scenario is firstly observed at Siphon 1 (downstream manhole) where the first spill occurs in the gravity section. As a result the maximum flow observed downstream of this location was 23.7 mgd. The spill occurs due to backing up caused from downstream 'throttles' from Manhole 28 to Manhole 29. In particular, the 33 inch pipe downstream of Siphon 2 (Siphon 2 to Manhole 29) has a pipe full capacity of 16.0 mgd which is far less than the average pipe capacity for the gravity section.

During the analysis, flows were allowed to increase after the first spill occurred at Siphon 1. As a result, the model predicted a second spill at Manhole 3 which hence limited the downstream flow to 26.0 mgd. This spill occurred due to downstream pipes (Manhole 3 to 10) exceeding their pipe full capacity of 24.0 mgd. Note the flow downstream of Siphon 1 remains limited to 23.7 mgd due to the on-going spill at this location. Table 3 summarizes the sequence of spill events in the gravity section of the land outfall.

Table 3. Spill Event Sequence

HARRF Flow (mgd)	Total Spills	Spill Location	Max. Flow (mgd)	Comments
< 23	0	No spills	-	No spills
24	1	Siphon 1 - d/s	23.7	Check if manhole is sealed.
25	2	Manhole 3	26.0	Max. flow between HARRF and Siphon 1
> 26	2	No further spills	26.0	Flow is limited due to spill at Manhole 3.

The hydraulic profiles local to the spill locations at Siphon 1 and Manhole 3 are shown in Figures 8 and 9, respectively. The hydraulic analysis of the gravity section concludes the maximum hydraulic capacity is 23.7 mgd prior to the first spill. Allowing for a 1feet free-board (ie; maximum head is 1 feet below ground), the maximum capacity is reduced to 22.6 mgd.

Environmental Engineering And Consulting
9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

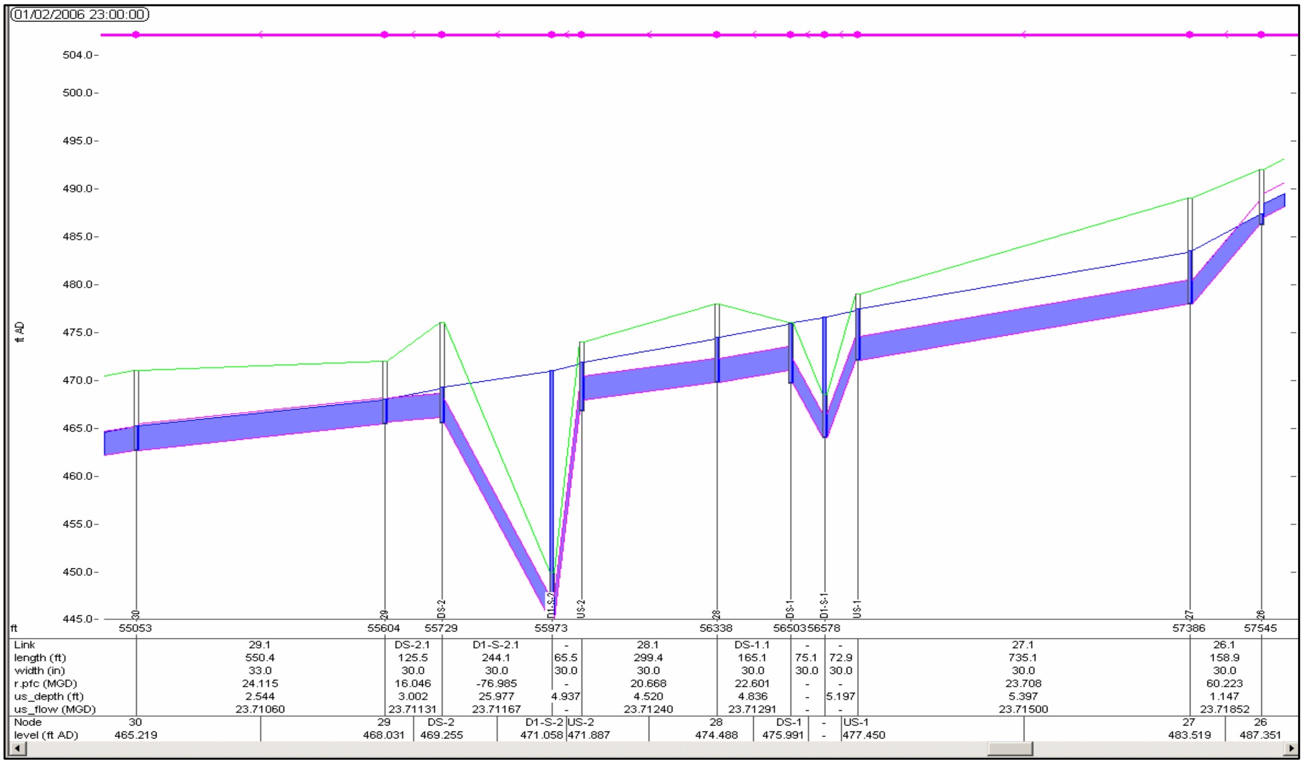


Figure 8. Hydraulic profile at Siphon 1.

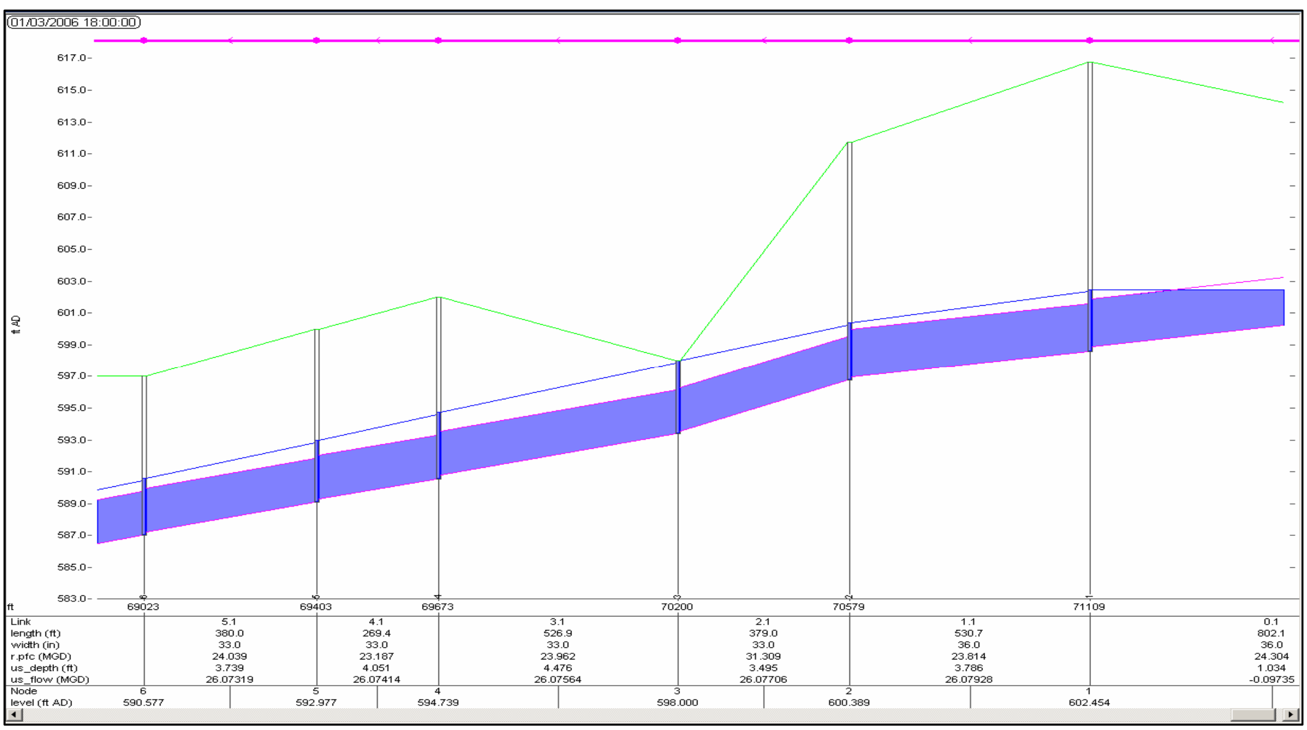


Figure 9. Hydraulic profile at Manhole 3.

4.2 Land Outfall – Pressurized Section

The pressurized section of the land outfall includes pipe segments from Manhole 74 to the regulator structure located at the SEJPA treatment facility. The manholes and air-relief structures are hydraulically sealed preventing flow spills. Manhole 74 is directly upstream of the pressurized section hence providing the first 'relief' manhole (ie; spill location) during periods of high downstream heads. Pipe segments up to Manhole 71, including Siphon 6, were also included in this part of the analysis as spills within this section typically result from downstream heads reflecting upstream.

The hydraulic capacity of the pressurized section was determined by gradually increasing the flow entering the outfall at HARRF until spills were observed upstream of the pressurized pipes. The capacity of the pressurized outfall is the maximum flow observed at the onset of the first spill or when the maximum head exceeds the allowable pressure rating.

As the flow increases in the pressurized section, the head (ie; pressure) increases which is reflected upstream into the gravity section. The model predicted the first spill occurring at Manhole 74 which corresponds to historic spills occurring at this location. The downstream flow is limited to 21.4 mgd when the spill occurs due to the constant head maintained at Manhole 74. The hydraulic profile (see Figures 10 and 11) shows the spill results from downstream heads backing upstream.

The hydraulic analysis of the pressurized section concludes the maximum hydraulic capacity is 21.4 mgd prior to the first spill. The maximum head, observed at air-relief structure 3, reaches 125.5 feet (54.3 psi) which is below the allowable pressure rating (100 psi) for the pressurized pipe section.

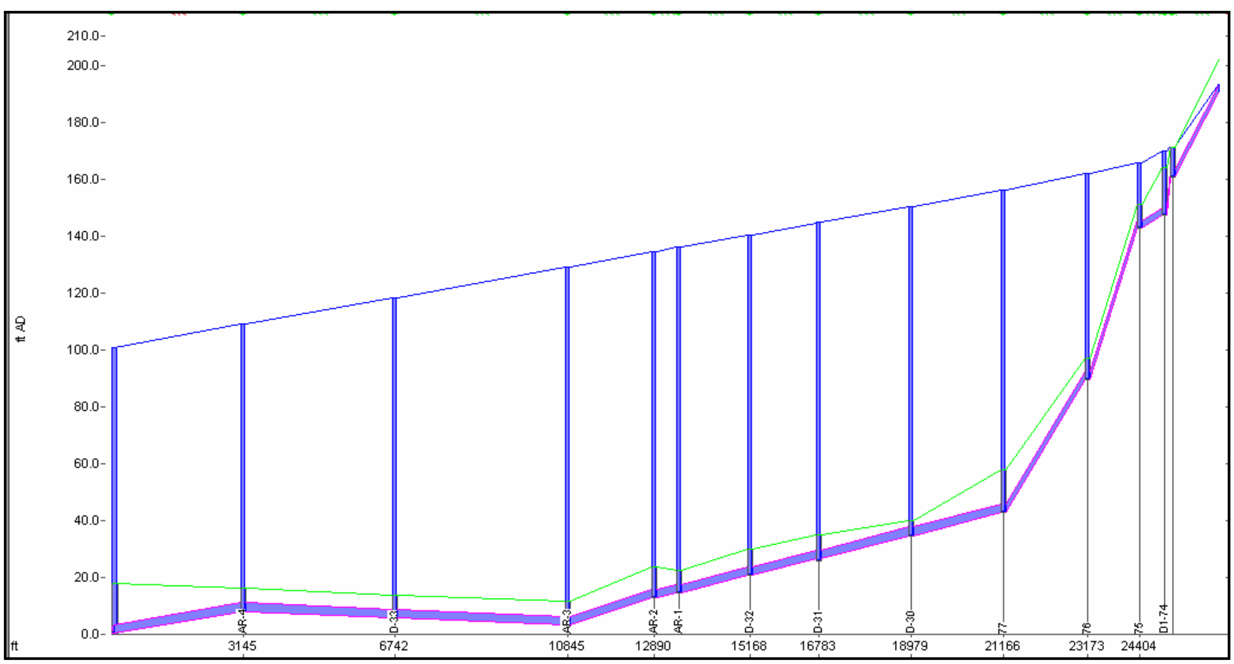


Figure 10. Hydraulic profile of pressurized section.

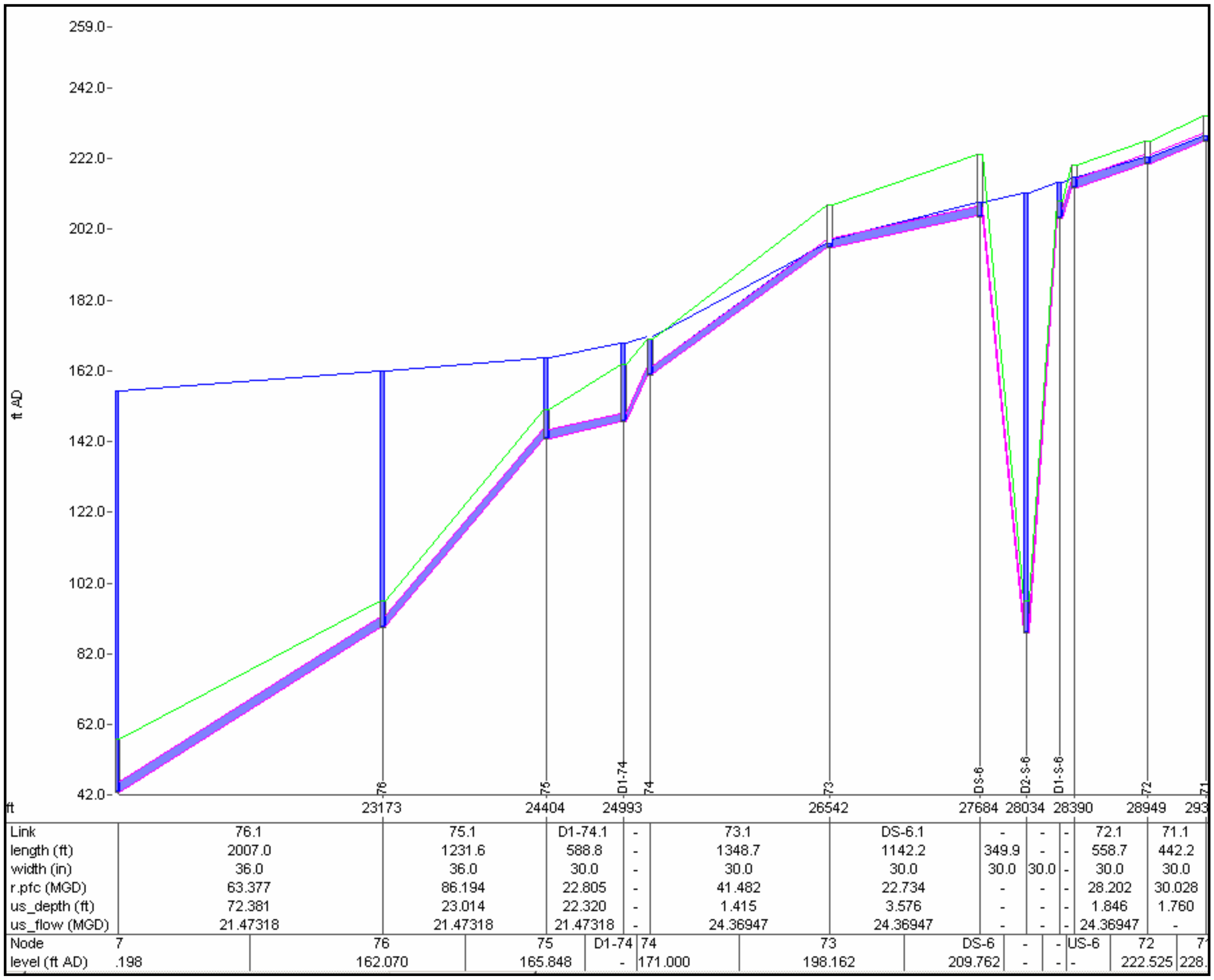


Figure 11. Hydraulic profile of Siphon 6.

4.3 Ocean Outfall

The ocean outfall pipeline comprises of a 30 inch pipe from the regulator structure to the original outfall diffuser, followed by a 48 inch outfall pipe extension connected to a new diffuser structure. The pipe section from the shoreline to the original diffuser section is a 30 inch reinforced concrete pressure pipe (RCPP) with a pressure limit of 50 feet at Station 0+00 (shoreline).

The ocean outfall disposes effluent from both the Escondido land outfall and effluent discharged from the SEJPA treatment facility. The analysis assumes SEJPA is discharging their maximum permitted flow (5.35 mgd). The regulator structure is designed to prevent pressure in the ocean outfall exceeding 82 feet by 'pinching' the flow discharged from the land outfall. This criteria is set by SEJPA. However, during high flow periods the regulator control valve reaches its minimum opening setting (50% open), and hence pressure in the ocean outfall may exceed the allowable pressure at Station 0+00 (model node D-12 located at the shoreline).

During the development of the hydraulic model, partial information was obtained from SEJPA regarding the operational logic of the regulator structure and observed flow and pressure data required for model calibration.

Environmental Engineering And Consulting
9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Similar to the land outfall analysis, the hydraulic capacity of the ocean outfall was determined by gradually increasing the flow entering the outfall until the outfall pressure reaches 82 feet at the SEJPA effluent connection. The flow at this head is deemed to be the maximum hydraulic capacity of the ocean outfall. The analysis was conducted assuming all 200 ports of the diffuser are open And a high tide level of +5.00 feet above datum. Figures 12 and 13 display the hydraulic profiles through the ocean outfall.

The predicted maximum hydraulic capacity of the ocean outfall is 25.8 mgd. This flow occurred when the head at SEJPA effluent connection reached 82 feet which limits the hydrostatic pressure of the RCPP sections at Station 0+00 located at the shoreline. The predicted outfall capacity correlates with the findings from a previous study (Reference 2), where the hydraulic capacity (also 25.8 mgd) occurs when the net internal pressure of the RCPP pipe at the shoreline is limited to 50 feet.

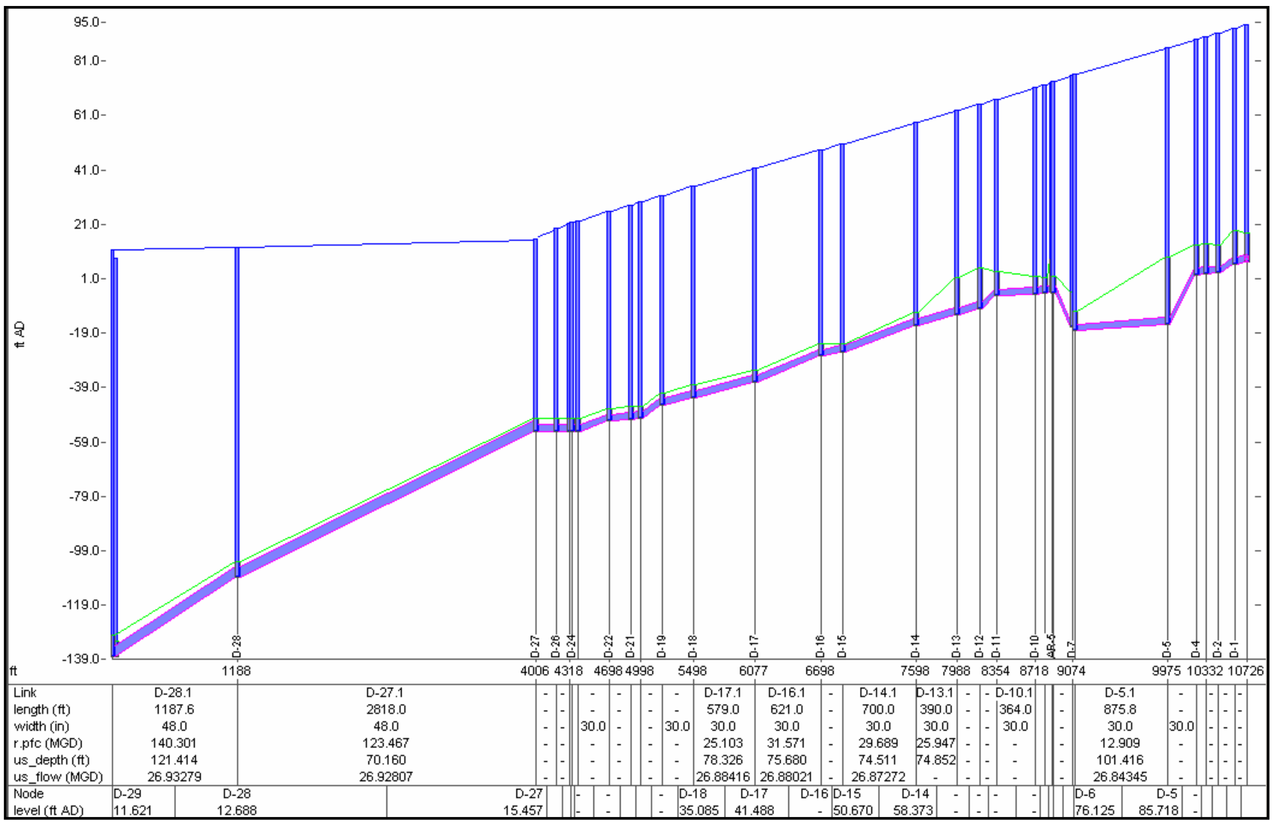


Figure 12. Hydraulic profile of the Ocean Outfall.

Environmental Engineering And Consulting
9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

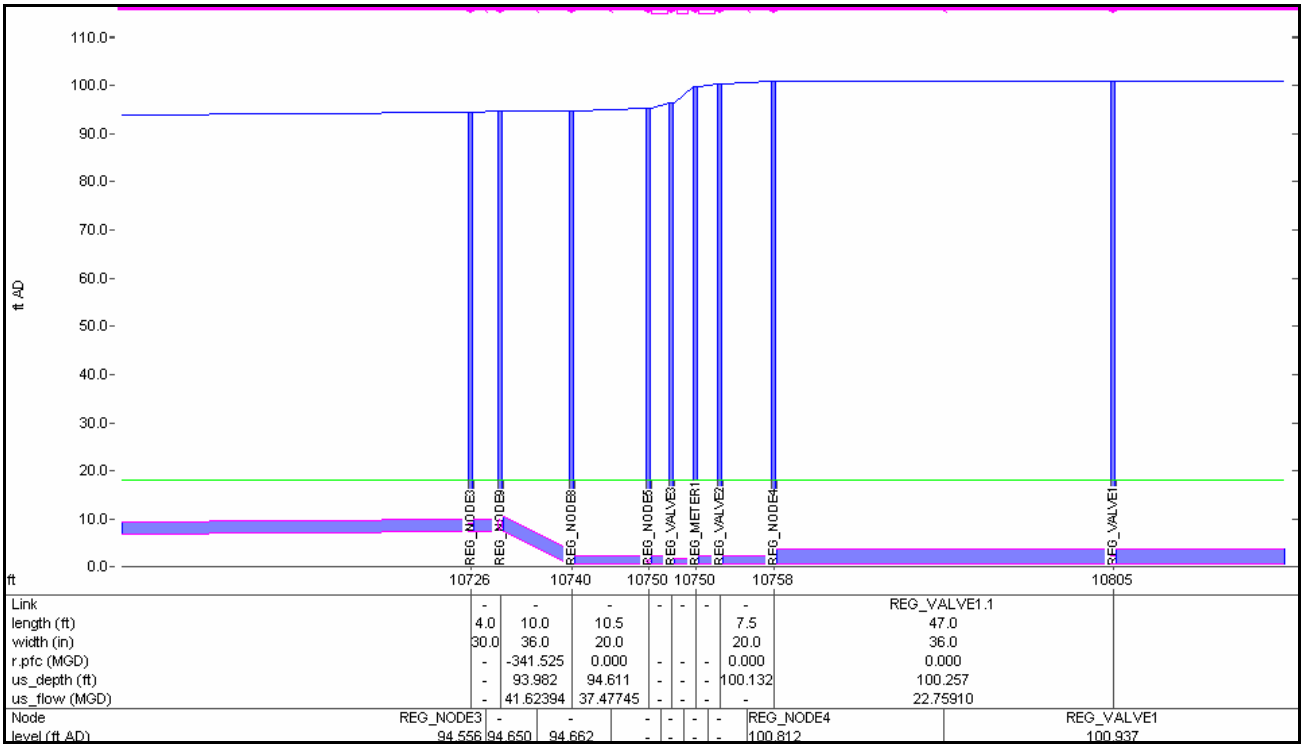


Figure 13. Hydraulic profile through the Regulator Structure.

Environmental Engineering And Consulting
9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

APPENDIX J

OCEAN OUTFALL CAPACITY TM

BROWN AND CALDWELL

TECHNICAL MEMORANDUM – FINAL

DATE: JULY 5, 2006

TO: ANGELA MORROW, CITY OF ESCONDIDO

FROM: VICTOR OCCIANO, BROWN AND CALDWELL

PREPARED BY: BILL FAISST, BROWN AND CALDWELL
TOM BIRMINGHAM, BROWN AND CALDWELL

SUBJECT: CITY OF ESCONDIDO
HALE AVENUE RESOURCE RECOVERY FACILITY –
SAN ELIJO OCEAN OUTFALL ASSESSMENT

SUMMARY

The existing San Elijo Ocean Outfall (SEOO) has a hydraulic capacity of about 25.5-25.8 million gallons per day (MGD) [*Tetra Tech, Inc. 2001; Brown and Caldwell, 2006*]. Per the current National Pollutant Discharge Elimination System (NPDES) permit (Order No. R9-2005-0101, NPDES No. CA0117981, June 8, 2005) and the Fact Sheet (Attachment F of the permit), the total monthly average effluent discharge from Hale Avenue Resource Recovery (HARRF) and from the San Elijo Water Reclamation Facility (SEWRF) cannot exceed 23.25 MGD - 18 MGD from HARRF and 5.25 MGD from SEWRF. Future growth in the area served by the HARRF will increase the average daily flow by about fifty percent to 27.5 mgd. Peak wet weather flow (PWWF) from the HARRF sewer service area is expected to be about 53.4 mgd. Combining this expected peak flow from HARRF with the PWWF assumed for the SEWRF service area of 10.5 (5.25 * 2) mgd yields a total potential peak ocean discharge of about 64 MGD (depending on the peaking factor for the SEWRF service area). Subtracting the intermittent live stream discharge flow allowed at HARRF of 9 mgd, the ocean outfall must be capable of conveying at least 55 mgd. If the outfall is to be expanded and the allowable disposal split between HARRF and SEWRF remains at 79 and 21 percent, respectively, the ocean outfall must be expanded to about 58.6 mgd (53.4/0.79 – 9).

For this study, two ultimate ocean outfall capacities were considered: 58 and 48 mgd. Each selected capacity is closely tied to various disposal alternatives reviewed for the City

of Escondido (City), which is described separately in the *Wastewater Treatment and Disposal Facilities Capacity Study Project Report*.

The probable initial dilution factor for the existing discharge is 237:1 per the Fact Sheet. If effluent discharge were to increase from 23.25 MGD to the maximum hydraulic capacity of the outfall (i.e., 25.8 MGD), the initial dilution would decrease by about five percent.

The City plans to increase the capacity of the HARRF in a manner that would maintain the current quality of effluent for ocean discharge. This effluent is suitable for discharge through the existing outfall in compliance with the Permit and California Ocean Plan (Ocean Plan) requirements. However, with increased flow, the mass emissions would increase proportionally to flow (given maintenance of current effluent quality and pretreatment controls) and, as noted above, initial dilution would decrease marginally. And if the City directs more effluent to reuse and/or live stream discharge, the annual mass emissions to the ocean would remain the same or possibly decrease slightly. Based on regulatory requirements and review of the Fact Sheet, it appears that the City should be able to obtain approval for increased discharge through the existing outfall up to its hydraulic capacity with minimal requirements for anti-degradation analysis. A thorough cleaning of the diffuser section (clearing debris from port and removing encrustation around the ports) would likely be required to ensure that the existing capacity and performance are maximized.

Increasing the SEOO capacity to allow disposal of the anticipated peak flow from HARRF at build out will require more improvements. A discharge from a properly designed diffuser in deeper water would achieve an initial dilution 5 to 10 percent greater than the existing discharge at its current location. The mass emissions of monitored constituents would increase substantially if the effluent quality is not upgraded, but could remain constant or even decrease as noted above. But, concentrations of the constituents outside of the zone of initial dilution would comply with current Ocean Plan limitations, ensuring that beneficial and recreational use areas along the shoreline are not impacted. Also, increased effluent recycling could limit mass emissions to current levels or even reduce them. Anti-degradation analyses likely would be required, but a "Simple Antidegradation Analysis" may be sufficient. Because of the scale of the project and the potential impacts, the City would need to prepare an Environmental Impact Report under the California Environmental Quality Act.

For this memorandum, the discharge system includes the onshore pipeline from the Regulator Structure to the beach and the offshore section from the beach through the diffuser. The City has two principal options for increasing the hydraulic capacity of the discharge system: 1) make incremental changes such as paralleling the existing, offshore and onshore sections; or 2) construct a new parallel outfall. The former approach could achieve a hydraulic capacity increase to about 35 MGD without constructing through the surf zone. With parallel construction through the surf zone, the overall capacity could be increased to the required capacity of 48 or 58 MGD (combined discharge from Escondido and San Elijo).

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

For the onshore section a review of site, access, and wetlands constraints point a likelihood of construction using trenchless technology - microtunneling being the best approach because of the limited laydown area. Any new crossings of the railroad and Highway 101 would be through steel casings. For the offshore portion a temporary trestle starting from a temporary staging area west of Highway would be required to traverse the surf zone, followed by sectional pipe placement from a laying barge offshore. Construction technology alternatives should be explored further during detailed design.

The best apparent alternative would be to construct a parallel outfall with a diffuser located in deeper water, but on the same compass heading as the existing outfall. This approach should minimize overall costs for design, construction, and construction management. It would also require facing regulatory and public review only once. Since portions of the SEOO system would be approaching the end of their useful life at the end of the planning period, construction of full parallel capacity would be the most prudent approach for planning purposes. To assess the condition of the existing SEOO and determine its remaining useful life, forensic investigation of the both the onshore and offshore sections, especially the asbestos cement pipe laid through the wetland, would be required. If the condition of the existing system is suitable for at least 50 years of additional service, then a phased approach or construction of the parallel system to carry the incremental flow above 25.8 MGD would cost less. Any new construction would require significant permitting approvals.

Table 1 presents order-of-magnitude capital cost opinions for various alternatives for the 48 and 58 MGD ultimate capacities. These costs are in current dollars for construction in Southern California and should be escalated to reflect projected construction dates. The Alternatives are briefly described below; a detailed discussion is provided in subsequent sections:

ALTERNATIVES FOR 48 MGD ULTIMATE CPACITY

Alternative A1 – Phased Expansion

- Phase I (Upgrades SEOO Capacity to 35 mgd)
  - Parallel the existing 30-inch diameter pipe beyond the surf zone (Station 15+00 to Station 40+00) with a 2,500 feet long, 54-inch-diameter pipe
  - Extend the diffuser section by about 500 feet into deeper water

- Phase II (Upgrades SEOO Capacity from 35 to 58 mgd)
  - Parallel or replace the land section of the SEOO (from the Regulator Structure to the beach)
  - Use a 42-inch diameter pipe in parallel
  - Extend the diffuser section by an additional 700 feet, into deeper water
  - If the condition and durability for the existing 30-inch-diameter pipe through the surf zone are a concern, then the replace this segment with a 48-inch diameter pipe

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Alternative A2 – Parallel Existing SEOO with New 42-inch Diameter System

Build a completely new 30- to 36-inch diameter parallel outfall and diffuser with a hydraulic capacity to accommodate City and San Elijo flows in excess of the existing outfall hydraulic capacity - about 16 to 24 MGD additional capacity.

Alternative A3 – Replace Existing SEOO with New 48-inch Diameter System

Build a completely new 48-inch diameter parallel outfall and diffuser with a hydraulic capacity of about 48 MGD.

ALTERNATIVES FOR 58 MGD ULTIMATE CPACITY

Alternative B1 – Phased Expansion

- Phase I
  - Parallel the existing 30-inch diameter pipe beyond the surf zone (Station 15+00 to Station 40+00) with a 2,500 feet long, 54-inch-diameter pipe
  - Extend the diffuser section by about 500 feet into deeper water

- Phase II
  - Parallel or replace the land section of the SEOO (from the Regulator Structure to the beach)
  - Use a 42-inch diameter pipe in parallel
  - Extend the diffuser section by an additional 1,200 feet, into deeper water
  - If the condition and durability for the existing 30-inch-diameter pipe through the surf zone are a concern, then the replace this segment with a 54-inch diameter pipe

Alternative B2 – Parallel 42-inch Diameter System

Build a completely new 42-inch diameter parallel outfall and diffuser with a hydraulic capacity to accommodate City and San Elijo flows in excess of the existing outfall hydraulic capacity - about 32 MGD.

Alternative B3 – Replace 54-inch Diameter System

Build a completely new 54-inch diameter parallel outfall and diffuser with a hydraulic capacity to accommodate the combined build-out flow - about 58 MGD.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

Table 1
Summary of Order-of-Magnitude Capital Cost Opinions for Outfall Alternatives
($million)

Alternative	48-MGD Ultimate Capacity On-shore	48-MGD Ultimate Capacity Off-shore	48-MGD Ultimate Capacity Total	58-MGD Ultimate Capacity On-shore	58-MGD Ultimate Capacity Off-shore	58-MGD Ultimate Capacity Total
A1/B1 – Phased Expansion						
▪ Phase I	0	14	14	0	16	16
▪ Phase II	7	21	28	8	26	34
▪ TOTAL	7	35	42	8	42	50
A2/B2 – Parallel Existing SEOO	7	48	55	8	61	69
A3/B3 – Replace Existing SEOO	9	62	71	10	70	80

NOTES:
1. All costs are current to Southern California, Spring 2006. They should be escalated to reflect escalation to the midpoint of construction after the construction schedule has been established
2. Capital costs include constructed costs, construction contingencies (40 percent), and allowances for engineering, legal, and administration (25 percent).
3. If full onshore pipe replacement is required, the onshore cost would increase to $10 million (54-inch diameter pipe versus 42-inch-diameter pipe).

Alternative A1 and B1 are the lowest cost alternative but includes the risks associated with the condition and longevity of the existing system. It would also require permitting and offshore construction on separate occasions. Alternative A3 and B would provide a new system with build out capacity for both communities, but it is the most costly. The overall ocean discharge permitting requirements and potential impacts are virtually the same for all alternatives since the dilution performance and construction impacts would be essentially the same.

INTRODUCTION

The City of Escondido has engaged Brown and Caldwell to determine the capacity of the HARRF. One of the tasks of the Capacity Study is to evaluate potential ocean discharge issues associated with increased effluent flows for ocean disposal. Discussions regarding the existing discharge and a two-phased approach to increasing the flow discharged through the SEOO are presented in this TM.

Future growth in the area served by the HARRF will increase the average daily flow treated by the HARRF from 15 to 27.5 mgd. Peak wet weather flow (PWWF) from the HARRF sewer service area is expected to be about 53.4 mgd. Combining this expected peak flow from HARRF with the PWWF assumed for the SEWRF service area of 10.5 (5.25 * 2) mgd yields a total potential peak ocean discharge of about 64 MGD (may be higher if the peaking factor for the SEWRF service area is higher than 2.0). Subtracting the intermittent live stream discharge flow allowed at HARRF of 9 mgd, the ocean outfall must be capable of conveying at least 55 mgd. If the outfall is to be expanded and the allowable disposal split between HARRF and SEWRF remains at 79 and 21 percent, respectively, the ocean outfall must be expanded to about 58.6 mgd (53.4/0.79 – 9).

For this study, two ultimate ocean outfall capacities were considered: 58 and 48 mgd. Each selected capacity is closely tied to various disposal alternatives reviewed for the City of Escondido (City), which is described separately in the *Wastewater Treatment and Disposal Facilities Capacity Study Project Report.*

EXISTING DISCHARGE

The existing SEOO includes about 4,250 feet of 30-inch diameter reinforced concrete pipe (RCP) constructed in 1965. A 48-inch-diameter, 5,200-foot RCP extension added in 1975 includes a 1,200-foot long diffuser section with discharge depths ranging from 116 feet to 148 feet. The diffuser has two hundred 2-inch diameter ports spaced six feet apart through the diffuser pipe side wall. All ports are open; however, based on the latest underwater video inspection (Thales GeoSolutions (Pacific), Inc., 2003), some of the ports are impaired by debris and possibly by encroaching marine growth. The ballast rock protection for the pipeline has suffered some deterioration caused by wave action and sand movement. Recent repairs have addressed ballast rock deficiencies.

Dilution analyses reported in the Fact Sheet estimate an initial dilution of 237:1 at a flow of 24 MGD. A key concern is the rising of the discharge plume to the surface without much dilution. However, the existing outfall produces a submerged effluent field much of the year. Excellent diffuser performance allows the City to avoid requirements for effluent disinfection while satisfying the 2005 California Ocean Plan bacterial standards for receiving waters.

PAST STUDIES

Several studies have reported on the condition of the SEOO and options for increasing the ocean outfall capacity. These studies and other pertinent references include:

- *Mixing in Inland and Coastal Waters*, H.B. Fischer, et al, Academic Press, 1979
- *City of Escondido San Elijo Joint Powers Authority San Elijo Ocean Outfall Improvements*, HYA Consulting Engineers, 1989
- *Phase II Treatment Process Upgrades and Enhancements – Facility Plan*, Water 3 Engineers, Inc. 1999
- *San Elijo Ocean Outfall Year 2002 Annual Inspection,* Thales GeoSolutions (Pacific), Inc.

- *San Elijo Ocean Outfall Inspection Year 2003 ROV Inspection,* Thales GeoSolutions (Pacific), Inc.
- *Hydraulic Capacity—San Elijo Ocean Outfall, Draft Technical Memorandum,* Tetra Tech, Inc., November 30, 2001.

The HYA and Water 3 Engineers reports focused on increasing discharge through the existing system by paralleling parts of the existing outfall and extending the outfall and diffuser into deeper water. These improvements reportedly would increase the hydraulic capacity of the SEOO to 32-40 MGD. Replacement of about 4,000 feet of the offshore section (Station 10+00 to Station 40+00) with a 54-inch diameter pipe and lengthening and extending the diffuser into deeper water were estimated to elevate the capacity to 35 MGD. The larger diameter pipe segment would reduce the headloss over the section by about 50 feet as compared to the existing 30-inch-diameter pipeline. This approach initially would avoid the more costly pipe replacement or paralleling near the surf zone, the area from the beach to water depths of 20 to 25 feet (depths at which barges and tug boats can safely work). None of the alternatives described in the reports can accommodate the peak build out flows now identified.

SYSTEM HYDRAULICS

Systems hydraulics is critical when assessing the capacity of the existing SEOO. The existing system is constrained by the pressure limitations of 50 feet of the asbestos cement pipe (Class 100) that extend from the regulator station to the shoreline and RCP pipe at the shoreline. It does not appear to be feasible to increase the existing system hydraulic capacity above 35 to 40 MGD without constructing larger diameter pipe through the surf zone. Analyses carried out by Brown and Caldwell as part of this study confirms the previously identified hydraulic limitations.

NEAR-TERM CAPACITY

The NPDES permit does not allow the full use of the SEOO hydraulic capacity (25.8 to 26.8 MGD). If all the ports in the diffuser section are left fully open, the SEOO should achieve the required initial dilution at higher flow rates. However, the estimated initial dilution under the current configuration would decrease slightly as flows increased above 24 MGD. The decrease can be estimated using the following equation (Equation 10.1, Fischer, et al):

$$S = 0.38\, g'^{1/3} d / q^{2/3}$$

where
- S = Centerline dilution
- g' = $g\,(\Delta \varrho / \varrho)$
- ϱ = the density of the discharge
- $\Delta \varrho$ = the density difference between the ambient fluid and the discharged fluid
- g = gravitational acceleration
- d = vertical distance above the source
- q = discharge per unit length

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

The decrease in initial dilution would be about five percent, to about 225:1. Preliminary calculations with the Visual Plumes program confirm this estimate.

The permitting for the minor capacity increase will likely follow similarly to the analysis presented in the Fact Sheet of the existing NPDES Permit. Minor increases in mass emissions, good dilution performance for the existing discharge, no evidence of deleterious effects from the existing discharge, and effluent constituent concentrations that rarely approach the permit limits are all key factors considered by the SDRWQCB when evaluating a proposed increase in discharge volume. In general, the existing HARRF operates very well. The pursuit to increase the allowable discharge through the SEOO should not be difficult.

LONG-TERM CAPACITY

Modifications to the existing outfall and diffuser would not achieve the required hydraulic capacity at buildout conditions. Based on the required flow capacity and the existing facilities, the following alternatives have been identified for increasing the capacity of the ocean outfall:
ALTERNATIVES FOR 48 MGD ULTIMATE SEOO CAPACITY

A1. Expand capacity in phases using existing facilities. Continue to use the existing land and ocean outfall in the permanent solution, assuming that the existing pipelines will last another 50 years. The phases would be as follows:

- Phase I – To provide an incremental increase in capacity, parallel the existing 30-inch diameter pipe beyond the surf zone (Station 15+00 to Station 40+00) with a 2,500 feet long, 54-inch-diameter pipe. Extend the diffuser section by about 500 feet into deeper water. This approach will ensure that the plume is well mixed and the bacteriological standards are met at offshore beneficial use areas.

- Phase II - Parallel or replace the land section of the SEOO (from the Regulator Structure to the beach). Use a 42-inch diameter pipe in parallel. Extend the diffuser section by an additional 700 feet, again into deeper water. (Note that if the condition and durability for the existing 30-inch-diameter pipe through the surf zone are suspect, then the replacement section should have a diameter of 48 inches.

A2. Build a completely new parallel outfall and diffuser with a hydraulic capacity to accommodate City and San Elijo flows in excess of the existing outfall hydraulic capacity - about 16 to 24 MGD. This alternative will include a land segment from the regulator structure across the wetlands and a new offshore outfall and diffuser. Based on reasonable pipe velocities - five to six feet per second (fps) - the preliminary estimate for pipe diameter is 30 to 36 inches. The offshore pipeline length upstream of the diffuser would be about 5,200 feet. The diffuser length would be at least 1,200 feet, with the diffuser located parallel to and offshore of the existing SEOO diffuser.

A3. Build a completely new parallel outfall and diffuser with a hydraulic capacity to accommodate the combined build-out flow--about 48 MGD. As in Alternative A2, this alternative would include a land segment from the regulator structure across the wetlands and a new offshore outfall and diffuser. Based on reasonable pipe velocities, the preliminary estimate for pipe diameter is 54 inches. The offshore pipeline length upstream of the diffuser would be about 8,000 feet. The diffuser length would be about 2,900 feet, with the diffuser located parallel to and offshore of the existing SEOO. Since this outfall would completely replace the existing system, the inshore end of the diffuser would start at the inshore end of the existing diffuser.

ALTERNATIVES FOR 58 MGD ULTIMATE SEOO CAPACITY

B1. Expand capacity in phases using existing facilities. Continue to use the existing land and ocean outfall in the permanent solution, assuming that the existing pipelines will last another 50 years. The phases would be as follows:

- Phase I – To provide an incremental increase in capacity, parallel the existing 30-inch diameter pipe beyond the surf zone (Station 15+00 to Station 40+00) with a 2,500 feet long, 54-inch-diameter pipe. Extend the diffuser section by about 500 feet into deeper water. This approach will ensure that the plume is well mixed and the bacteriological standards are met at offshore beneficial use areas.

- Phase II - Parallel or replace the land section of the SEOO (from the Regulator Structure to the beach). Use a 42-inch diameter pipe in parallel. Extend the diffuser section by an additional 1,200 feet, again into deeper water. (Note that if the condition and durability for the existing 30-inch-diameter pipe through the surf zone are suspect, then the replacement section should have a diameter of 54 inches.

B2. Build a completely new parallel outfall and diffuser with a hydraulic capacity to accommodate City and San Elijo flows in excess of the existing outfall hydraulic capacity - about 32 MGD. This alternative will include a land segment from the regulator structure across the wetlands and a new offshore outfall and diffuser. Based on reasonable pipe velocities - five to six feet per second (fps) - the preliminary estimate for pipe diameter is 42 inches. The offshore pipeline length upstream of the diffuser would be about 5,200 feet. The diffuser length would be at least 1,700 feet, with the diffuser located parallel to and offshore of the existing SEOO diffuser.

B3. Build a completely new parallel outfall and diffuser with a hydraulic capacity to accommodate the combined build-out flow--about 58 MGD. As in Alternative B2, this alternative would include a land segment from the regulator structure across the wetlands and a new offshore outfall and diffuser. Based on reasonable pipe velocities, the preliminary estimate for pipe diameter is 54 inches. The offshore pipeline length upstream of the diffuser would be about 8,000 feet. The diffuser length would be about 2,900 feet, with the diffuser located parallel to and offshore of the existing SEOO. Since this outfall would completely replace the existing system, the inshore end of the diffuser would start at the inshore end of the existing diffuser.

Figures 1 and 2 show conceptual alignments and segment locations for the alternatives.

Construction Options. For all alternatives, the following site constraints for the parallel or new system exist:

- The land outfall must cross under an active railroad track and Highway 101. Both the railroad and Caltrans will require that any crossing be in a casing to protect overlying infrastructure from a blow out.

- Construction space around the Regulator Structure appears to be highly constrained.

- A small amount of property not constrained by wetlands is located on either side of the railroad tracks or immediately east of the Highway 101.

- Other than a narrow parking strip, no land is available west of Highway 101.

- The land portion must traverse under the wetlands east and west of the railroad track.

- None of the areas adjacent of the crossings listed above and around the regulator structure appear to have good lay down areas or back site areas where steel or high density polyethylene (HDPE) could be pre-assembled prior to installation.

- Distances are too great for horizontal directional drilling (HDD) with some pipe materials such as ductile iron.

- Some soil types, especially sands, might be incompatible with larger diameter HHD options.

Construction of the new outfall and diffuser would have significant transitory impacts. A particular concern would be related to the construction through the reach from the Regulator Structure to the beach and through the surf zone (beach area to a depth of 20 to 30 feet). The location of the highway immediately adjacent to the ocean and the minimal staging area for construction would increase construction complexity and costs. Construction options for the onshore section include HDD, microtunneling, and conventional cut and cover. The larger pipe sizes required and limited back site distance would eliminate HDD. Conventional cut and cover would have significant impacts in the

9665 Chesapeake Drive, Suite 201, San Diego, California 92123
Tel: 858. 514.8822 Fax: 858.514.8833

wetlands area and was deemed infeasible to permit if other technology would be available. Hence, microtunneling was assumed for cost estimating purposes.

For the offshore section, options include constructing a temporary trestle to use as a platform for construction through the surf zone, using HDD trenchless technology, and conventional tunneling. Constraints listed above preclude HDD. Likewise, conventional tunneling would be infeasible. Hence the cost estimates presented herein assume a temporary trestle through the surf zone would be constructed, laying from a barge further offshore. For the nearshore area, the estimates include an allowance for a temporary coffer dam constructed with sheet piling and filled with sand to create a construction working area west of Highway 101.

Discharge Performance. For all alternatives, a discharge from a properly designed diffuser located in part in deeper water would achieve an initial dilution 5 to 10 percent greater than the existing discharge at its current location. The increased initial dilution would result from discharge into deeper water. Lengthening the diffuser or locating a new diffuser (Alternative 3) in deeper water could be used to increase initial dilution even further. Mass emissions would increase substantially if flow increased and the effluent quality remained the same (i.e., not upgraded to tertiary level), but concentrations outside of the zone of initial dilution would comply with the current ocean plan limitations. If the recycled water program increased in scope and some live-stream discharge were added, the annual average mass emissions could be comparable or even less than under current conditions. Impacts would be spread over a larger area so that potential for chronic effects would be minimal if such effects occurred at all. The offshore end for the discharge also would be deeper and located further from beneficial and recreational use areas along the shoreline. Anti-degradation analyses would likely be required, but environmental impacts would be minimal. A "Simple Antidegradation Analysis" may be sufficient. Because of the scale of the project and the potential impacts, the City would need to prepare an Environmental Impact Report under the California Environmental Quality Act.

Permits and Easements. Permitting agencies that would be involved in the expansion of the outfall include the San Diego Regional Water Quality Control Board, the California Coastal Commission, the California States Lands Commission, the United States Army Corps of Engineers, and Caltrans. A state permit for the microtunneling operation would be required. Other agencies such as State of California Department of Fish and Game (Streambed Alteration Agreement) and United States Fish and Wildlife Service would likely have issues with crossing of the lagoon just inland from South Coast Highway 101. An agreement/easement with the railroad would be needed for the new crossing under the railroad tracks.

Cost Opinion. To develop cost options for the alternatives, we used information from recent similar projects as well as cost curve for the marine construction. The accompanying spreadsheets provide details for the construction costs for various segments. These costs are current to coastal Southern California in the Spring 2006. They should be escalated to the construction midpoint once a schedule is established. The estimates include a contingency allowance of 40 percent to reflect major uncertainties especially associated with geotechnical conditions and permitting constraints. Each alternative's costs include an allowance of 25 percent for engineering, legal, and administrative costs, a higher than typical percentage being used to reflect the challenging regulatory issues. Table 1 shows the costs by alternative. Alternative 1 has the lowest cost because it takes the greatest advantage of the existing facilities. Cost for Alternative A2 and B2 is about 40 percent greater because more offshore pipeline is required. Alternative A3 and B3 has the highest cost since it includes a complete new system.

Comparison of Alternatives. Alternative A1 and B1 is the lowest cost alternative but includes the risks associated with the condition and longevity of the existing system. It would also require permitting and offshore construction on separate occasions. Alternative B3 would provide a new system with build out capacity for both communities, but it is the most costly. The overall ocean discharge permitting requirements and potential impacts are virtually the same for all alternatives since the dilution performance and construction impacts would be essentially the same.

9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
TEL: 858. 514.8822 FAX: 858.514.8833

APPENDIX K

STORM FLOW MODELING TM

BROWN AND CALDWELL

TECHNICAL MEMORANDUM – FINAL

DATE:	JULY 5, 2006
TO:	ANGELA MORROW, CITY OF ESCONDIDO
FROM:	VICTOR OCCIANO, BROWN AND CALDWELL
PREPARED BY:	MATT DAVIS, BROWN AND CALDWELL RION MERLO, BROWN AND CALDWELL RON APPLETON, BROWN AND CALDWELL
SUBJECT:	CITY OF ESCONDIDO HALE AVENUE RESOURCE RECOVERY FACILITY (HARRF) – STORM FLOW MODELING

SUMMARY

A hydrologic model was developed to simulate the infiltration and inflow (I/I) generated in service areas within the City of Escondido (City) and the City of San Diego's Rancho Bernardo (RB) community. Flows reaching HARRF from the City were modeled separately from those contributed by RB. This provided flexibility in evaluating future alternatives which might include the diversion of RB flows to another wastewater treatment facility. The model was calibrated and validated using rainfall data from a nearby rain gauge and flow data measured at the HARRF. Once the model was calibrated, a long-term record of historic rainfall was entered into the model and used to generate 38-years of simulated flows. From the long-term flow record, it was possible to determine the recurrence frequency of flow events such as a 10-year peak flow event. The flow events determined from the long-term simulation were used to support planning activities related to the design of alternatives for the HARRF. These planning activities and the development of alternatives are described elsewhere.

MODEL DESCRIPTION

BC developed the hydrologic model that was used to simulate I/I for this project. The model is implemented in a software system called the Capacity Assurance Planning Environment (CAPE). CAPE provides an integrated software environment for working with large sets of time-series data and developing I/I models.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

The main function of the model is to simulate I/I based on rainfall data. In addition to simulating I/I, the model also simulates sanitary flows. The model adds the sanitary flow to the I/I to calculate total flow.

The model uses an average daily sanitary flow value and a diurnal pattern to simulate sanitary flows. The diurnal pattern provides information about how the sanitary flow varies about an average value throughout the day. The formula used to calculate the sanitary flow is shown below.

$$Q_i^{Sanitary} = CD_{f(i)} \tag{1}$$

where,

$Q_i^{Sanitary}$	=	sanitary flow at timestep i (MGD)
C	=	average daily sanitary flow (MGD)
$D_{f(i)}$	=	diurnal variation corresponding to the i^{th} timestep's time of the day

I/I is calculated by assigning weights to rainfall that has fallen over different historic time intervals. For example, the I/I could be calculated by multiplying a weighting coefficient by the amount of rainfall that fell in the last hour then adding a weighting coefficient multiplied by the amount of rainfall that fell in the last day and then adding a weighting coefficient multiplied by the amount of rainfall that fell in the last month. The I/I generated by rainfall over each of these different time intervals is referred to as an I/I component. The I/I components which have a shorter time interval are referred to as the fast-response components, while the components which respond over a longer time frame are referred to as slow-response components. Generally, fast-response I/I is influenced by inflow-type mechanisms while slow-response I/I is influenced by infiltration-type mechanisms. In calculating I/I, the model takes into account the rainfall at the timestep and rainfall which has fallen in the past. The equation used for both I/I and sanitary flow is shown below

$$Q_i^{I/I} = \sum_{j=0}^{m} \frac{A_j}{B_j} \left(\sum_{k=0}^{B_j} R_{i-k} \right) \tag{2}$$

where,

Q_i	=	flow at timestep i (MGD)
A_j	=	coefficient for the j^{th} I/I component
m	=	number of I/I components
B_j	=	number of timesteps over which rainfall for the j^{th} component is summed
R_i	=	rainfall at the i^{th} timestep (in)

The only known quantity is the rainfall (R_i). All of the other parameters must be determined from the calibration.

MODELING PROCESS

The following steps were followed in the development and utilization of the City and RB models: calibration, validation, and long-term simulation. In the first step, the model was calibrated so that simulated flows adequately matched the measured flows during the calibration period. In the second

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

step, the model was used to simulate flows outside of the calibration period. The simulated flows were then compared to the measured flows during the time period. This process helps to validate the model and provide confidence that it can adequately predict flows outside of the time period over which it was calibrated. In the final step, a long-term rainfall record was input to the model and used to simulate a long-term flow record.

INPUT DATA

Rainfall from the Ramona Airport was used during the calibration and validation period for both the City and RB models. The Ramona Airport rain gauge was the closest gauge to the service areas which had reliable data readily available for the last few years. The input data used to develop and utilize the models is shown in Table 1.

Table 1. Input data for model.

<table>
<tr><th rowspan="2">Task</th><th rowspan="2">Rainfall Data Source</th><th colspan="2">Flow Data Source</th></tr>
<tr><th>Escondido</th><th>Rancho Bernardo</th></tr>
<tr><td>Calibration</td><td>Ramona Airport[1]</td><td rowspan="2">Rancho Bernardo Meter[2] subtracted from HARRF Parshall Flume[2]</td><td rowspan="2">Rancho Bernardo Meter[2]</td></tr>
<tr><td>Validation</td><td>Ramona Airport[1]</td></tr>
<tr><td>Long-term simulation</td><td>Lake Wohlford[1]</td><td>N/A</td><td>N/A</td></tr>
</table>

Notes:
1. Data measured at a 1 hour timestep.
2. Data measured at a 5 minute timestep.

The location of the Ramona Airport in relation to the HARRF is shown in Figure 1. The long-term simulation used the Lake Wohlford rain gauge which is also shown in Figure 1. The Lake Wohlford rain gauge was the closest rain gauge to the service areas with a long-term rainfall record. Both the Ramona Airport and the Lake Wohlford rain gauge collected data at an hourly timestep.

Flow data was used during the calibration and validation process to compare the simulated flows against measured flows. The way in which flows from the City and RB service areas enter the HARRF are shown in Figure 2. The RB flows are measured by a flow meter located along the pipeline, immediately upstream of the grit removal system. Flows recorded at this meter were used for the calibration and validation of the RB model. The Parshall flume in the headworks measures the combination of flow from the City and RB service areas. In order to estimate flows from the City service area, data from the RB flow meter were subtracted from the HARRF Parshall flume data. The estimated City flows were used for the City model. Flows were measured at the RB flow meter and the HARRF Parshall flume every 5 minutes.

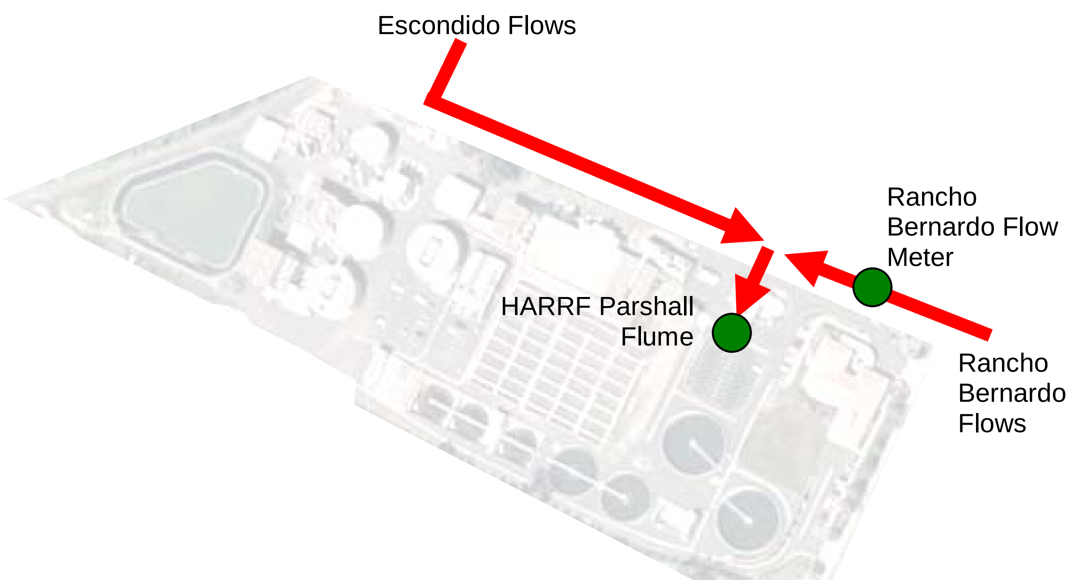


Figure 2. Measurement of flow streams.

Note that flows from RB are pumped to the HARRF by Pump Station 77 (PS77). RB has an agreement with City which governs the amount of flow that can be sent to the HARRF. There is a 5 million gallons (MG) off-line storage basin which can be used to temporarily store flows during periods of elevated flow. However, during wet weather events RB is allowed to pump at its maximum capacity of 9 million gallons per day (MGD).

MODEL CALIBRATION

The model was calibrated over the time period from April 1, 2003 through December 31, 2005 using the required flows and rainfall data shown in Table 1. The model was run using an hourly timestep. A shorter duration timestep was not possible since the model timestep is limited to the largest timestep reflected in input data. The hourly timestep is a result of using hourly rainfall data. The 5-minute flow data was averaged to an hourly timestep in order to be consistent with the rainfall data.

The following parameters shown in Equations 1 and 2 were calibrated: m, B_j, A_j, C, and $D_{f(i)}$. The average daily sanitary flow, C, and the diurnal pattern, $D_{f(i)}$, were determined from an analysis of flows during dry days. The diurnal pattern of the flow by the time it reaches the HARRF for City and RB is shown in Figures 3 and 4, respectively. The estimated average daily sanitary flow was 9.8 MGD and 3.5 MGD for City and RB, respectively.

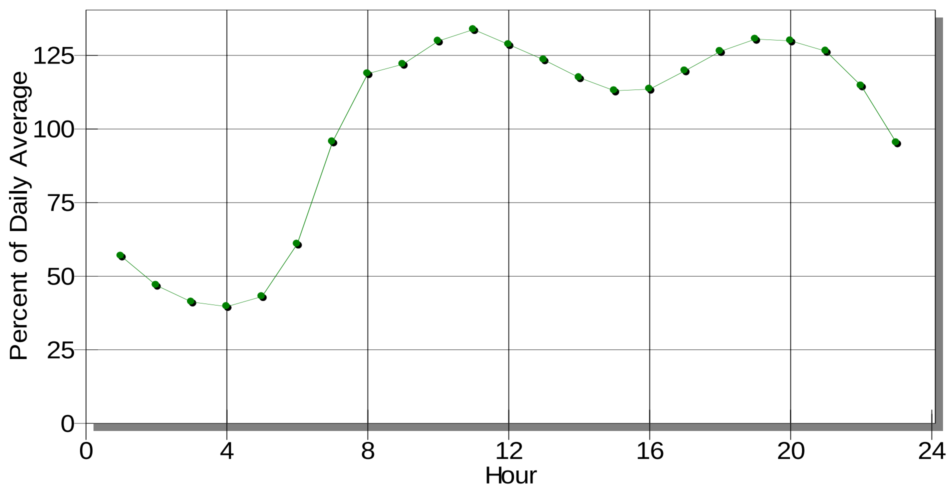


Figure 3. Diurnal Pattern for Escondido service area.

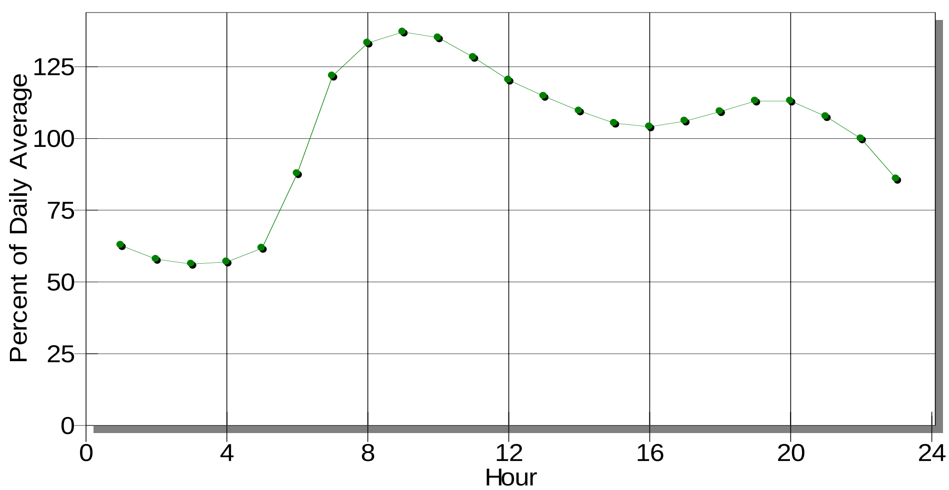


Figure 4. Diurnal pattern for Rancho Bernardo service area.

In calibrating the I/I, various combinations of I/I components and coefficients were evaluated. The optimal set of parameters is shown in Tables 2 and 3 for the City and RB models, respectively. As can be seen from the tables, the longer term I/I response components (e.g., 168 hours, 720 hours, and 2,160) generally have larger coefficients than the shorter term response components. This would seem to suggest that slow response I/I mechanisms, such as groundwater-induced infiltration, may be more prevalent in the system than fast response I/I mechanisms such as direct infiltration.

Table 2. Escondido I/I Parameters (*m*=6).

Timespan (hrs) B_j	Coefficient A_j
1	95
3	232
24	828
168	2,500
720	560
2,160	6,317

Table 3. Rancho Bernardo I/I Parameters (m=6).

Timespan (hrs) B_j	Coefficient A_j
1	50
12	100
72	166
168	1,197
720	1,697
2,160	95

The calibration of the City model to the largest and second largest peak flow events is shown in Figures 5 and 6, respectively. These events occurred in early 2005. The flows are plotted against the left-hand y-axis and rainfall is plotted in reverse order against the right-hand y-axis.

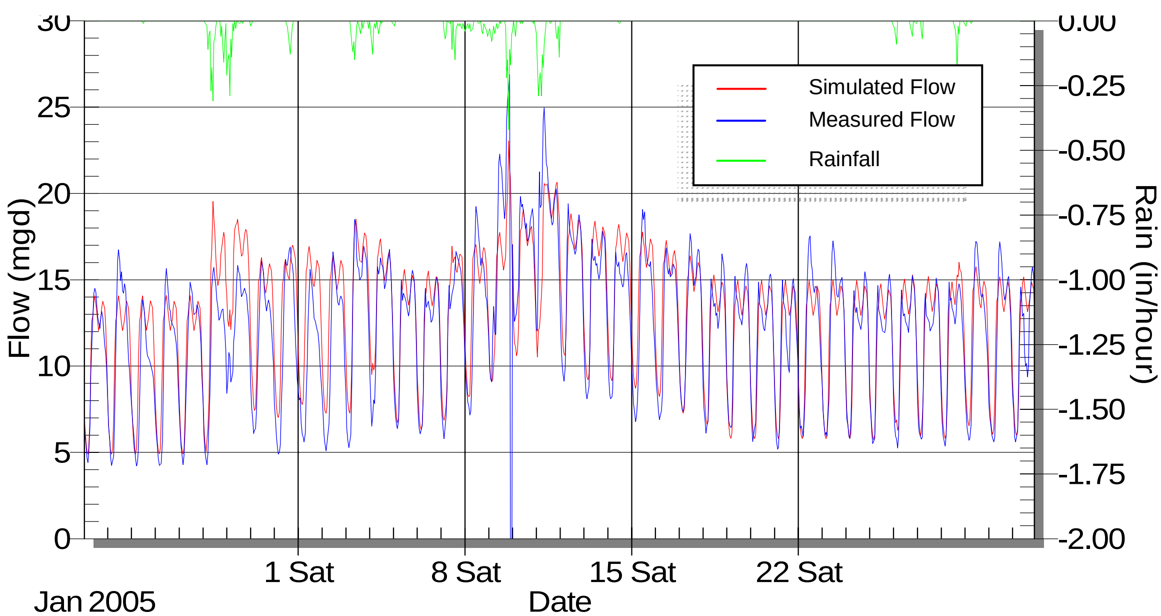


Figure 5. Escondido model calibration to largest event.

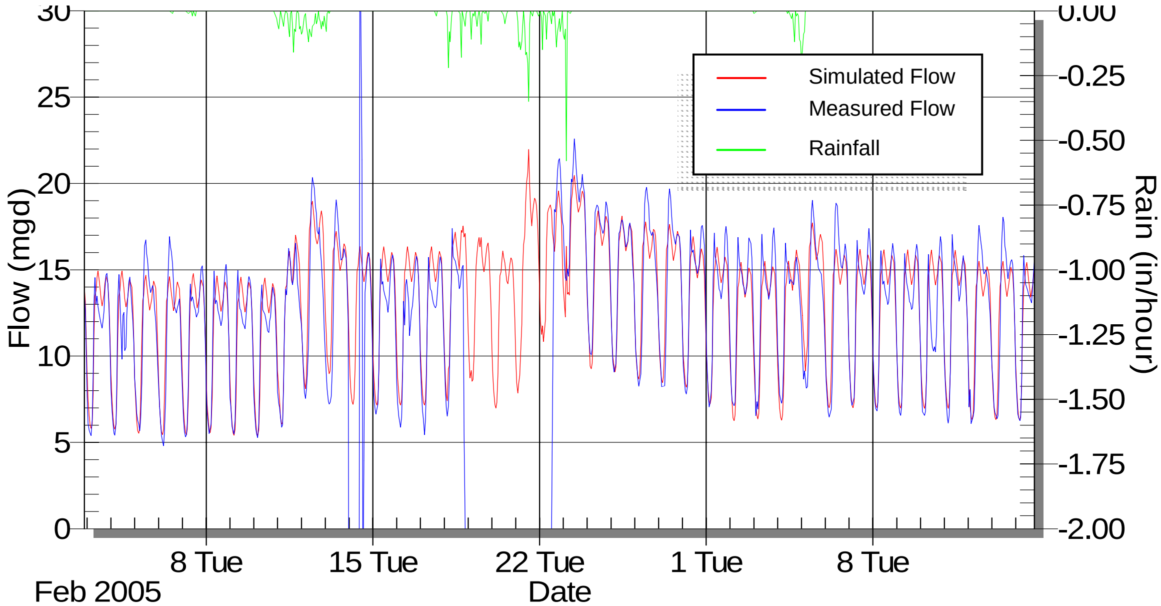


Figure 6. Escondido model calibration to second largest event.

The simulated peak flow for the largest event is 14% lower than the measured value. The simulated peak flow for the second largest event is 3% lower than the measured value. Note that the some of the measured flows are missing during the second largest event. A summary of the top twenty largest City peak flow events during the calibration period is presented in Table 4.

Table 4. Top 20 largest events during Escondido model calibration period.

Rain Event				Measured Flow		Simulated Flow			
Start Date	End Date	Rain Depth (in)	Duration (hr)	Peak Flow (MGD)	Vol (MG)	Peak Flow (MGD)	Volume (MG)	Peak Flow Error (%)	Volume Error (%)
1/7/2005 3:00	1/12/2005 12:00	4.8	129	26.88	84.76	23.05	83.05	-14.30%	-2.00%
2/20/2005 11:00	2/23/2005 20:00	3.54	81	22.59	N/A[1]	21.97	55.12	-2.80%	N/A[2]
10/27/2004 2:00	10/29/2004 3:00	2.12	49	20.89	25.95	19.95	28.62	-4.50%	10.30%
2/26/2004 1:00	2/26/2004 22:00	0.57	21	20.41	11.63	15.71	11.30	-23.00%	-2.80%
2/10/2005	2/13/2005 17:00	2.41	66	20.34	36.68	18.98	38.70	-6.70%	5.50%
3/4/2005 10:00	3/5/2005 16:00	1.05	30	19.03	17.90	17.73	18.70	-6.90%	4.50%
10/17/2004 6:00	10/21/2004 18:00	3.7	108	18.83	54.25	19.97	58.69	6.10%	8.20%
1/3/2005 5:00	1/5/2005 14:00	1	57	18.49	31.94	18.55	34.11	0.30%	6.80%
4/28/2005 5:00	4/29/2005 15:00	1.2	34	18.21	18.42	20.2	19.67	10.90%	6.80%
3/18/2005 16:00	3/20/2005 19:00	0.45	51	17.63	26.19	16.44	27.98	-6.70%	6.80%
4/8/2005 22:00	4/9/2005 15:00	0.04	17	17.41	8.21	14.8	8.07	-15.00%	-1.70%
2/17/2005 16:00	2/20/2005 9:00	1.22	65	17.39	N/A[1]	17.53	37.08	0.80%	N/A[2]
2/21/2004 17:00	2/24/2004	1.11	56	17.3	27.94	16.06	28.12	-7.20%	0.60%
11/24/2005 11:00	11/25/2005 0:00	0.01	13	17.3	7.15	13.33	6.82	-22.90%	-4.70%
2/27/2004 23:00	2/28/2004 12:00	0.01	13	17.29	5.71	14.71	5.85	-14.90%	2.50%
1/28/2005 14:00	1/29/2005 20:00	0.4	30	17.23	15.84	16.01	16.72	-7.00%	5.50%
9/20/2005 7:00	9/20/2005 22:00	0.02	15	17.09	9.05	13.22	8.00	-22.60%	-11.60%
5/3/2003 4:00	5/4/2003 18:00	0.92	38	17.08	18.88	17.59	21.01	3.00%	11.20%
3/3/2005 0:00	3/3/2005 15:00	0.03	15	17.06	7.92	15.5	7.52	-9.20%	-5.20%
12/31/2004 13:00	1/1/2005 7:00	0.34	18	16.9	8.86	17	10.37	0.60%	17.00%

Notes:

1. Volume could not be calculated due to missing flow data.
2. Error could not be calculated due to missing measured flow data.

The calibration of the RB model to the largest and second largest peak flow events is shown in Figures 7 and 8, respectively. These events coincide with the largest events for City. The simulated peak flow for the largest event is 3% lower than the measured value. The simulated peak flow for the second largest event is 10% lower than the measured value. As with the second largest City event, some of the measured flows are missing. A summary of the top twenty largest RB peak flow events during the calibration period is presented in Table 5.

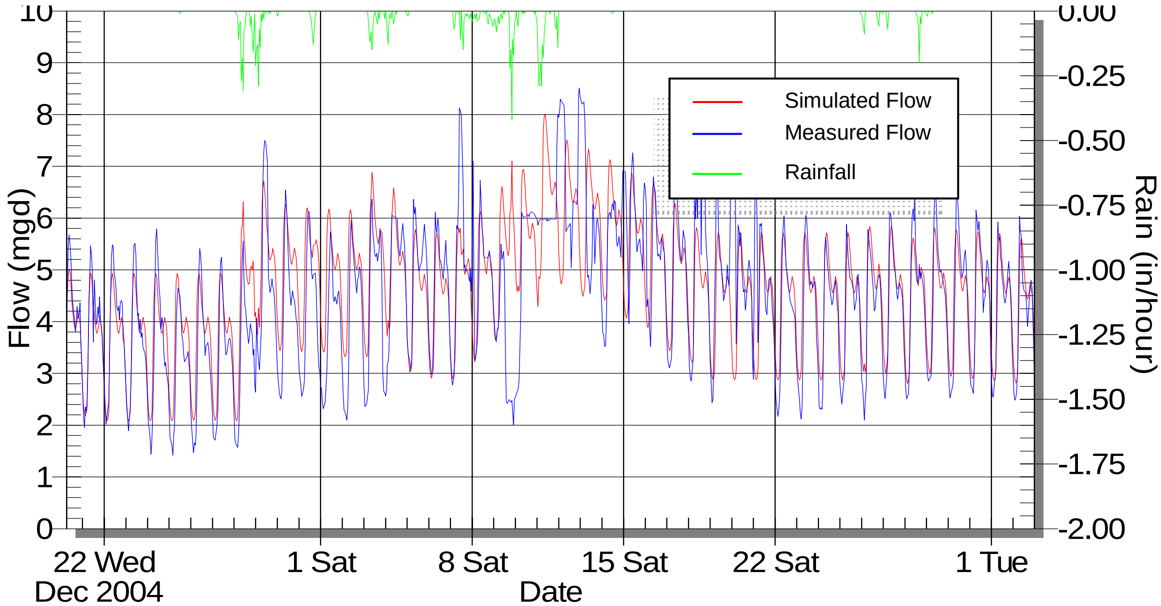


Figure 7. Rancho Bernardo model calibration to largest event.

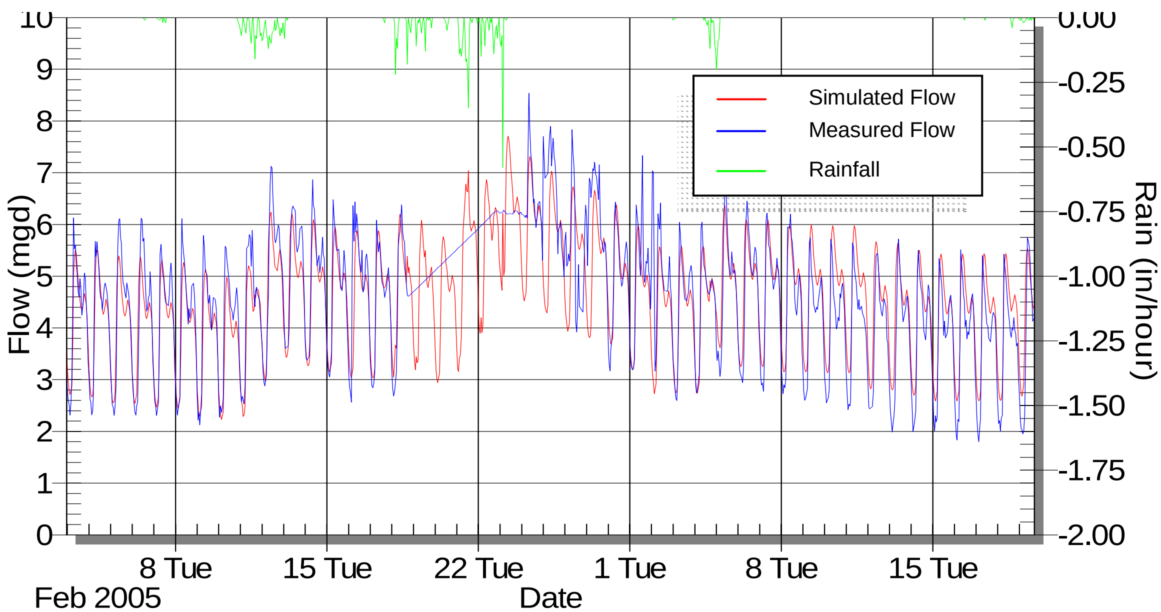


Figure 8. Rancho Bernardo model calibration to second largest event.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833

Table 5. Top 20 largest events during Rancho Bernardo model calibration period.

Rain Event				Measured Flow		Simulated Flow			
Start Date	End Date	Rain Depth (in)	Duration (hr)	Peak Flow (MGD)	Volume (MG)	Peak Flow (MGD)	Vol (MG)	Peak Flow Error (%)	Volume Error (%)
1/7/2005 3:00	1/12/2005 12:00	4.8	129	8.3	28.65	8.01	29.91	-3%	4%
12/28/2004 4:00	12/30/2004 14:00	2.72	58	7.5	10.57	6.72	12.24	-10%	16%
3/18/2005 16:00	3/20/2005 19:00	0.45	51	7.17	8.13	5.63	9.37	-22%	15%
2/10/2005 23:00	2/13/2005 17:00	2.41	66	7.13	13.47	6.24	12.73	-13%	-6%
1/14/2005 12:00	1/15/2005 1:00	0.01	13	6.94	3.54	6.54	3.36	-6%	-5%
3/4/2005 10:00	3/5/2005 16:00	1.05	30	6.79	6.20	6.38	6.37	-6%	3%
10/27/2004 2:00	10/29/2004 3:00	2.12	49	6.69	9.71	7.26	10.60	9%	9%
1/28/2005 14:00	1/29/2005 20:00	0.4	30	6.63	5.94	5.81	5.97	-12%	1%
2/17/2005 16:00	2/20/2005 9:00	1.22	65	6.38	N/A[1]	6.2	12.76	-3%	N/A[2]
1/3/2005 5:00	1/5/2005 14:00	1	57	6.37	11.78	6.88	12.35	8%	5%
11/12/2003 2:00	11/12/2003 20:00	0.11	18	6.34	3.59	4.93	2.96	-22%	-17%
5/3/2003 4:00	5/4/2003 18:00	0.92	38	6.3	6.51	5.83	7.10	-7%	9%
2/20/2005 11:00	2/23/2005 20:00	3.54	81	6.29	N/A[1]	7.71	19.25	23%	N/A[2]
11/16/2003 0:00	11/16/2003	0.02	19	6.19	3.34	4.89	2.98	-21%	-11%
2/26/2004 1:00	2/26/2004 22:00	0.57	21	6.19	3.61	5.98	4.14	-3%	15%
1/26/2005 19:00	1/27/2005 18:00	0.18	23	6.11	4.40	5.85	4.55	-4%	3%
3/3/2005 0:00	3/3/2005 15:00	0.03	15	6.04	2.82	5.59	2.80	-7%	-1%
11/4/2004 0:00	11/4/2004 13:00	0.01	13	6	2.29	5.42	2.34	-10%	2%
12/7/2003 19:00	12/8/2003 11:00	0.05	16	5.97	2.75	4.87	2.43	-18%	-12%
4/8/2005 22:00	4/9/2005 15:00	0.04	17	5.92	2.87	5	2.69	-16%	-6%

Generally speaking, I/I models are considered to be well-calibrated if they can provide an accuracy within 25% of the measured peak flows. The average absolute error in the peak flows for the 20 largest events for City and RB is 9% and 11%, respectively. None of the top twenty events for either model have an error equal to or greater than 25%. The calibration for both the City and RB models is considered to be very good.

It is important to note that usage of the off-line storage basin at PS77 shaves the peak flows before they are measured at the HARRF. It is known that the off-line storage basin has been used in the past, but detailed information is not available. For example is not known exactly when and for how long the storage was used. It would have been preferable to calibrate the RB flows upstream of PS77 so that the "true" flows coming from the service area could be modeled without the complicating effect of storage. However, since these up-system flows and information about the storage utilization were not available, the only available option was to calibrate to the data that was available at the HARRF.

MODEL VALIDATION

After calibration, the City and RB models were validated. The validation period was from January 1, 2006 to March 29, 2006. The models were validated using rainfall data from the Ramona Airport and flow data from the HARRF Parshall flume and the RB meter.

Six rainfall events occurred during the validation period. Table 6 provides a summary of the City flows during these events. The table confirms that the model continues to reliably predict flows outside of the validation period. The average absolute error was 13%, which is slightly higher than the error during the calibration period, but note that the sample size is much smaller.

Table 6. Events during Escondido model validation period.

Rain Event				Measured Flow		Simulated Flow			
Start Date	End Date	Rain Depth (in)	Duration (hr)	Peak Flow (MGD)	Volume (MG)	Peak Flow (MGD)	Volume (MG)	Peak Flow Error (%)	Volume Error (%)
1/1/2006 19:00	1/3/2006 12:00	1.07	41	19.85	19.94	16.67	20.26	-16.00%	1.60%
1/14/2006 16:00	1/15/2006	0.2	24	16.92	11.01	13.81	10.95	-18.40%	-0.50%
2/17/2006 22:00	2/18/2006 18:00	0.1	20	16.34	9.14	13.48	8.59	-17.50%	-6.00%
2/19/2006 2:00	2/19/2006 18:00	0.26	16	16.13	7.59	13.91	7.54	-13.80%	-0.70%
2/27/2006 14:00	3/1/2006 0:00	1.44	34	15.96	17.67	16.88	19.00	5.70%	7.60%
2/14/2006 12:00	2/15/2006 1:00	0.01	13	13.85	6.52	12.95	6.51	-6.50%	-0.10%

Table 7 provides a summary of the RB peak flows during the storm events. The average absolute error is 6%, which is better than the absolute error during the calibration period (though it is noted again that the validation period sample size is small). It is concluded that the RB model can also provide reliable flow predictions outside of the calibration period.

Table 7. Events during Rancho Bernardo model validation period.

Rain Event				Measured Flow		Simulated Flow			
Start Date	End Date	Rain Depth (in)	Duration (hr)	Peak Flow (MGD)	Volume (MG)	Peak Flow (MGD)	Volume (MG)	Peak Flow Error (%)	Volume Error (%)
1/1/2006 19:00	1/3/2006 12:00	1.07	41	5.51	5.96	5.46	7.15	-1%	20%
1/14/2006 16:00	1/15/2006 16:00	0.2	24	5.13	3.33	5.12	3.93	0%	18%
2/14/2006 12:00	2/15/2006 1:00	0.01	13	4.62	2.21	4.27	2.09	-8%	-5%
2/17/2006 22:00	2/18/2006 18:00	0.1	20	5.34	3.04	4.92	3.10	-8%	2%
2/19/2006 2:00	2/19/2006 18:00	0.26	16	5.29	2.57	5.1	2.75	-4%	7%
2/27/2006 14:00	3/1/2006	1.44	34	5.43	5.67	6.16	6.28	14%	11%

LONG-TERM MODEL SIMULATION

After completing the validation, the models were run in long-term simulation mode. The Lake Wohlford rain gauge was used to supply a long-term rainfall record to the model. Thirty-eight years of data were available from the Lake Wohlford rain gauge beginning in the year 1949 and ending in 1986. The result of running the models with the Lake Wohlford data was a 38-year flow record for both City and RB. These flows represent how the system is expected to respond over a wide range of meteorological conditions.

As mentioned previously, the capacity of PS77 is 9 MGD. During the long-term simulation the RB model simulated events which exceeded this capacity. In order to account for capacity of PS77, the simulated flows for RB were revised so that they did not exceed 9 MGD. Analysis of the flow record indicated that RB flows exceeded 9 MGD for approximately 11 days during the 38 years of simulation.

The simulated flows from City and RB were added together to develop a table of flow events at the HARRF. Flows from the calibration, validation, and long-term simulation were combined to produce a 41+ year record. The top twenty simulated peak flow events at HARRF are summarized in Table 8. The largest event resulted from the historical storm in December of 1966 which resulted in more than 11 inches of rainfall. This storm resulted in a simulated peak flow of 44.3 MGD at the HARRF.

Table 8. Largest simulated events for combined flows from Escondido and Rancho Bernardo.

Rain Event				Simulated Flow			
Start Date	End Date	Rain Depth (in)	Duration (hr)	Peak Flow (MGD)	Volume (MG)	Rank Peak Flow	Rank Volume
12/4/1966 20:00	12/7/1966 5:00	11.2	57	44.33	82.16	1	1
2/20/1980 15:00	2/21/1980 18:00	2.3	27	41.29	41.22	2	26
2/19/1980	2/20/1980 13:00	3.8	35	41.13	49.60	3	16
1/24/1969 3:00	1/26/1969 15:00	5.6	60	39.6	68.29	4	5
3/4/1978 3:00	3/6/1978 0:00	3.4	45	37.97	59.67	5	9
3/21/1969 20:00	3/22/1969 4:00	1.3	8	36.37	7.72	6	785
2/23/1969 10:00	2/26/1969 12:00	5.3	74	35.13	81.83	7	2
2/17/1980 19:00	2/18/1980 16:00	2.3	21	34.98	26.55	8	108
1/16/1978 15:00	1/17/1978 10:00	2.1	19	34.84	21.39	9	166
1/27/1980 19:00	1/30/1980 1:00	5.5	54	34.23	54.83	10	13
2/16/1980 10:00	2/17/1980 11:00	2.2	25	34.19	28.88	11	81
1/10/1980 21:00	1/11/1980 23:00	3.7	26	34.09	29.78	12	66
2/9/1963 13:00	2/11/1963	6.04	50	33.6	51.65	13	14
2/28/1978 13:00	3/3/1978	5	66	33.38	72.35	14	4
2/13/1980 13:00	2/15/1980 6:00	2.6	41	32.89	36.44	15	34
4/1/1965 8:00	4/2/1965 8:00	2.2	24	32.78	21.51	16	163
12/24/1983 18:00	12/27/1983 16:00	3.6	70	32.4	59.94	17	8
11/30/1982 2:00	11/30/1982	2.1	17	32.23	16.60	18	262
1/26/1969 16:00	1/27/1969 3:00	0.5	11	31.91	13.21	19	389
2/15/1980 7:00	2/16/1980 1:00	1.2	18	31.86	20.11	20	188

It should be noted that aside from the capacity limitation of 9 MGD at PS77, the results from the hydrologic model do not take into account capacity limitations in the collection system. The predicted flow of 44.3 MGD during the December 1966 event may be large enough to overwhelm the capacity of some portions of the collection system. In this kind of a circumstance, the amount of flow delivered to the HARRF would actually be less than expected due to upstream sanitary sewer overflows. The hydrologic model, however, does not take this into account. It merely predicts what would arrive at the HARRF if the collection system had enough capacity to deliver it.

STORAGE VOLUME ANALYSIS

The most-limiting hydraulic component of the HARRF is the ocean outfall which has a permitted capacity of 23.75 MGD (see Section 4.1.3). During wet weather events, on-site storage can be used to temporarily store flows which exceed the capacity of the outfall. When the capacity of the on-site storage is exceeded, flows are released from the HARRF as an intermittent live stream discharge. As a result, a relationship exists between the amount of on-site storage available at the HARRF and the frequency with which the intermittent live stream discharge will occur: more storage equates to a less frequent intermittent live stream discharge, and vice versa. An analysis was performed to quantify the relationship between storage and stream discharge frequency.

In order to perform the analysis, a simple mass-balance model was developed. The model is referred to as the Simple Storage/Outfall Model (SSOM) and is shown schematically in Figure 9.

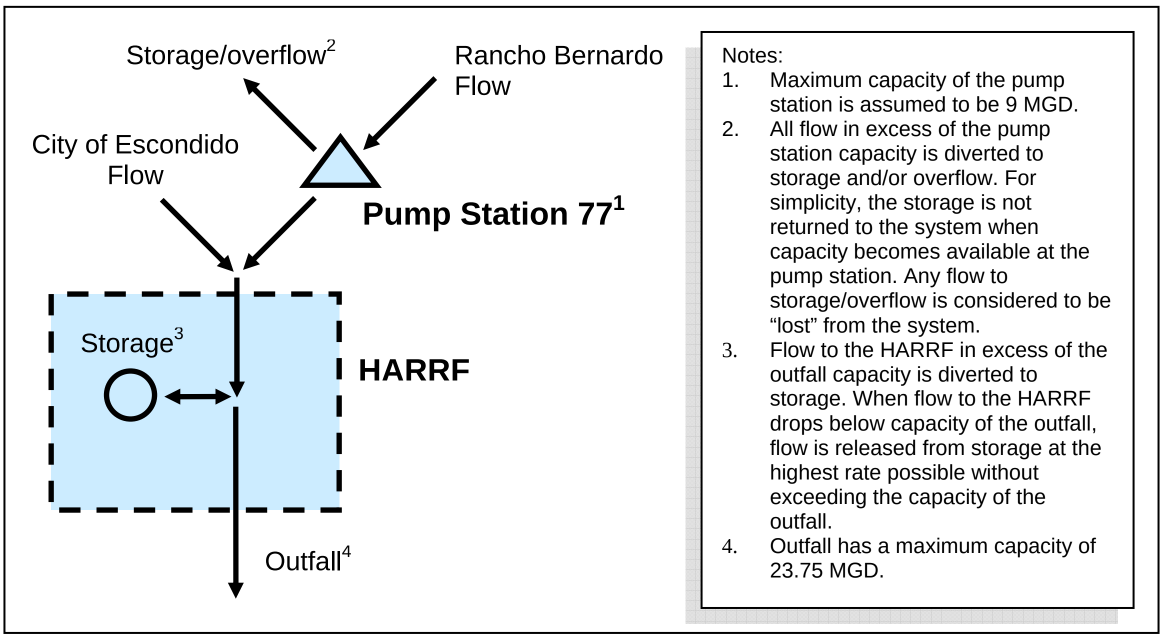


Figure 9. Schematic of Simple Storage/Outfall Model

The basic operation of the model is described below:

- Flows from the City of Escondido and Rancho Bernardo provided input to the model. The flows were obtained from the long-term simulations discussed in the previous sub-section. The SSOM used an hourly timestep in order to be consistent with the timestep of the input flow data. All together, 41 years of flows were simulated using the SSOM.
- The SSOM routed flow from Rancho Bernardo through Pump Station 77. The maximum hydraulic capacity of Pump Station 77 was assumed to be 9 MGD. Any flow in excess of the pump station's capacity was assumed to be diverted to storage or, if necessary, to an overflow location. For simplicity, the storage was not returned to the system when

capacity becomes available at the pump station. As a result, any flow to storage/overflow was considered to be “lost” from the system.

- The Rancho Bernardo flows (after Pump Station 77) were combined with the City of Escondido flows to estimate influent flow to the HARRF. Any flows at the HARRF in excess of the outfall capacity were routed to storage. When flows dropped below the capacity of the outfall, the storage was emptied at the highest rate possible without exceeding the capacity of the outfall.
- The SSOM is a simple mass-balance model and did not perform any sophisticated hydraulic routing. For example, the time of travel from Pump Station 77 to the HARRF and the time of travel through the HARRF are not taken into account.

The SSOM output the storage at the HARRF for each hour during the 41-year simulation period. The SSOM analyzed the data and summarized the maximum amount of storage during each of the “storage events”. A “storage event” is defined as a continuous period of time during which storage is utilized. The event begins when flow is first diverted to the storage facility and ends when the storage facility is completely empty. The next event begins when the storage facility begins to fill again.

A summary of the largest storage events from the 41 years of simulation are shown Table 9. The rank of each of the events is shown in the table. The largest simulated storage event occurred from 2/16/1980 through 3/14/1980. The storage facility was in continuous use for almost 27 days during this event. The maximum amount of storage required during this period of time was more than 58 MG. The results indicate that the HARRF would need more than 58 MG of storage to avoid an intermittent stream discharge during an event of this size.

Table 9. Storage vs. Return Period Results

Start Date	End Date	Max. Storage Volume (gallons)	Ranking	Return Period (years)
2/16/1980 8:00	3/14/1980 5:00	58,221,470	1	68.7
12/5/1966 5:00	12/21/1966 6:00	54,422,420	2	25.8
3/1/1978 7:00	3/19/1978 6:00	33,047,700	3	15.8
1/25/1969 7:00	2/2/1969 22:00	14,104,540	4	11.4
2/24/1969 7:00	3/4/1969 17:00	10,100,720	5	9.0
1/28/1980 18:00	2/4/1980 2:00	6,736,510	6	7.4
2/10/1963 7:00	2/13/1963 23:00	5,842,833	7	6.2
1/11/1980 6:00	1/15/1980 4:00	4,914,360	8	5.4
1/14/1978 18:00	1/20/1978 5:00	4,753,080	9	4.8
12/30/1951 7:00	1/1/1952 2:00	3,416,445	10	4.3
1/18/1952 7:00	1/20/1952 3:00	2,983,339	11	3.9
2/15/1986 7:00	2/16/1986 18:00	2,874,967	12	3.6
3/17/1982 9:00	3/20/1982 1:00	2,791,424	13	3.3
1/26/1956 17:00	1/28/1956 6:00	2,570,427	14	3.0
1/11/2005 7:00	1/13/2005 1:00	2,103,213	15	2.8
4/7/1958 7:00	4/8/1958 7:00	2,091,613	16	2.6
3/16/1952 7:00	3/17/1952 5:00	2,061,890	17	2.5
2/4/1958 8:00	2/5/1958 5:00	2,056,105	18	2.3
2/14/1980 8:00	2/15/1980 6:00	1,985,841	19	2.2
2/9/1976 7:00	2/10/1976 5:00	1,936,869	20	2.1

Table 9. Storage vs. Return Period Results

Start Date	End Date	Max. Storage Volume (gallons)	Ranking	Return Period (years)
2/15/1980 8:00	2/16/1980 6:00	1,930,374	21	2.0
11/29/1985 9:00	11/30/1985 5:00	1,637,275	22	1.9
3/22/1958 7:00	3/23/1958 4:00	1,611,129	23	1.8
4/1/1958 8:00	4/2/1958 5:00	1,547,232	24	1.7
2/23/2005 7:00	2/24/2005 4:00	1,407,276	25	1.7
4/3/1958 8:00	4/4/1958 4:00	1,405,603	26	1.6
12/25/1983 9:00	12/26/1983 3:00	1,383,670	27	1.5
2/13/1978 8:00	2/14/1978 3:00	1,378,959	28	1.5
3/16/1986 8:00	3/17/1986 3:00	1,303,154	29	1.4
3/22/1954 9:00	3/23/1954 3:00	1,289,063	30	1.4
11/30/1982 7:00	12/1/1982 2:00	1,188,726	31	1.3
1/24/1969 9:00	1/25/1969 5:00	1,155,345	32	1.3
2/6/1969 8:00	2/7/1969 3:00	1,127,732	33	1.3
12/27/1971 10:00	12/28/1971 2:00	921,787	34	1.2
2/21/2005 10:00	2/22/2005 2:00	920,264	35	1.2
12/28/1971 8:00	12/29/1971 2:00	899,058	36	1.2
3/21/1979 7:00	3/22/1979 2:00	843,385	37	1.1
3/21/1969 20:00	3/22/1969 2:00	826,283	38	1.1
12/3/1966 12:00	12/4/1966 2:00	753,670	39	1.1
1/9/2005 18:00	1/10/2005 2:00	731,382	40	1.0
3/1/1970 8:00	3/2/1970 1:00	726,356	41	1.0

The return period of each of the storage events is estimated in Table 9. The return period is the frequency with which the storage volume is expected to be equaled or exceeded on average. It is expressed in terms of years. The largest event during the 41 simulated years is estimated to have a return period of 68.7 years. The second largest is estimated to have a return period of 28.8 years.

The return period estimates shown in Table 9 were calculated using a technique called plotting position. Plotting position formulae are frequently used in the fields of climatology and hydrology to estimate the return period associated with specific events (e.g., rainfall, flood, etc.). Please refer to Chapter 3 of *Hydrology and Floodplain Analysis* (Bedient and Huber, 1988) for a discussion of plotting position.

The Cunnane plotting position formula was used to calculate the return periods. The Cunnane plotting position was used because it is unbiased and is a relatively distribution-free plotting position, implying that it is appropriate when the underlying distribution of the data is unknown. The Cunnane formula is show below:

Formula 2

$$T = \frac{n + 0.2}{m - 0.4}$$

where T is the return period, n is the number of periods, and m is the rank of the event (m = 1 is the largest event).

A plot of the return period versus the maximum storage volume is shown in Figures 10 and 11. Figure 10 shows return periods ranging from 1 to 100 years, while Figure 11 shows return period ranging from 1 to 10 years. The return period is shown using a logarithmic scale.

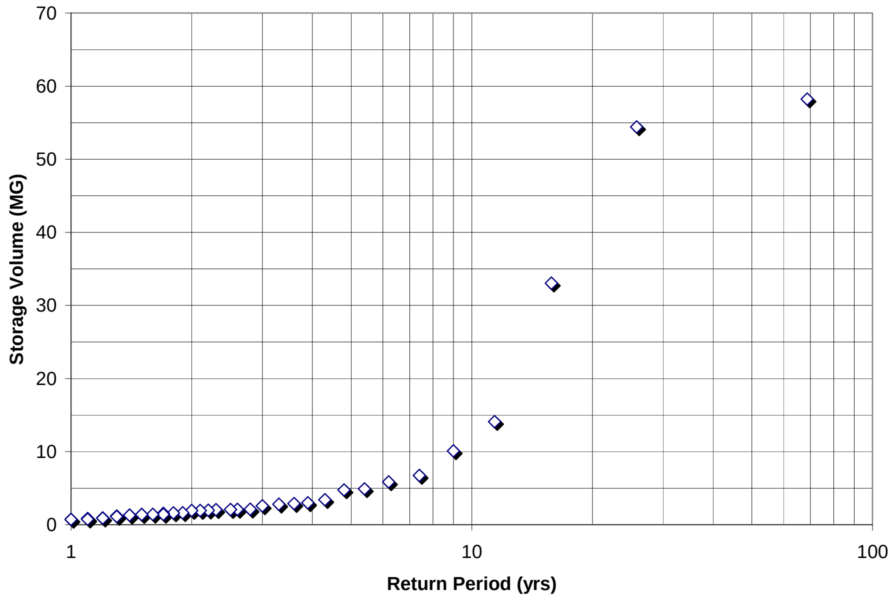


Figure 10. Storage Volume Exceedance Frequencies (Return Period 1-100 Years)

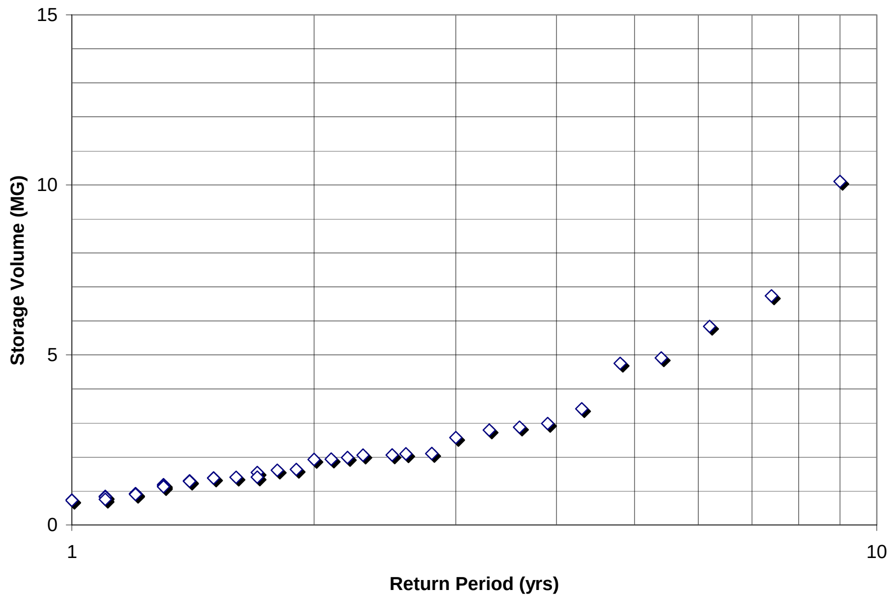


Figure 11. Storage Volume Exceedance Frequencies (Return Period 1-10 Years)

The estimated relationship between storage volume and the frequency of intermittent stream discharge is presented in Table 10. The storage volume requirements were estimated using the data in Table 9 and Figures 10-11. The storage volumes were rounded up to the nearest 0.5 MG.

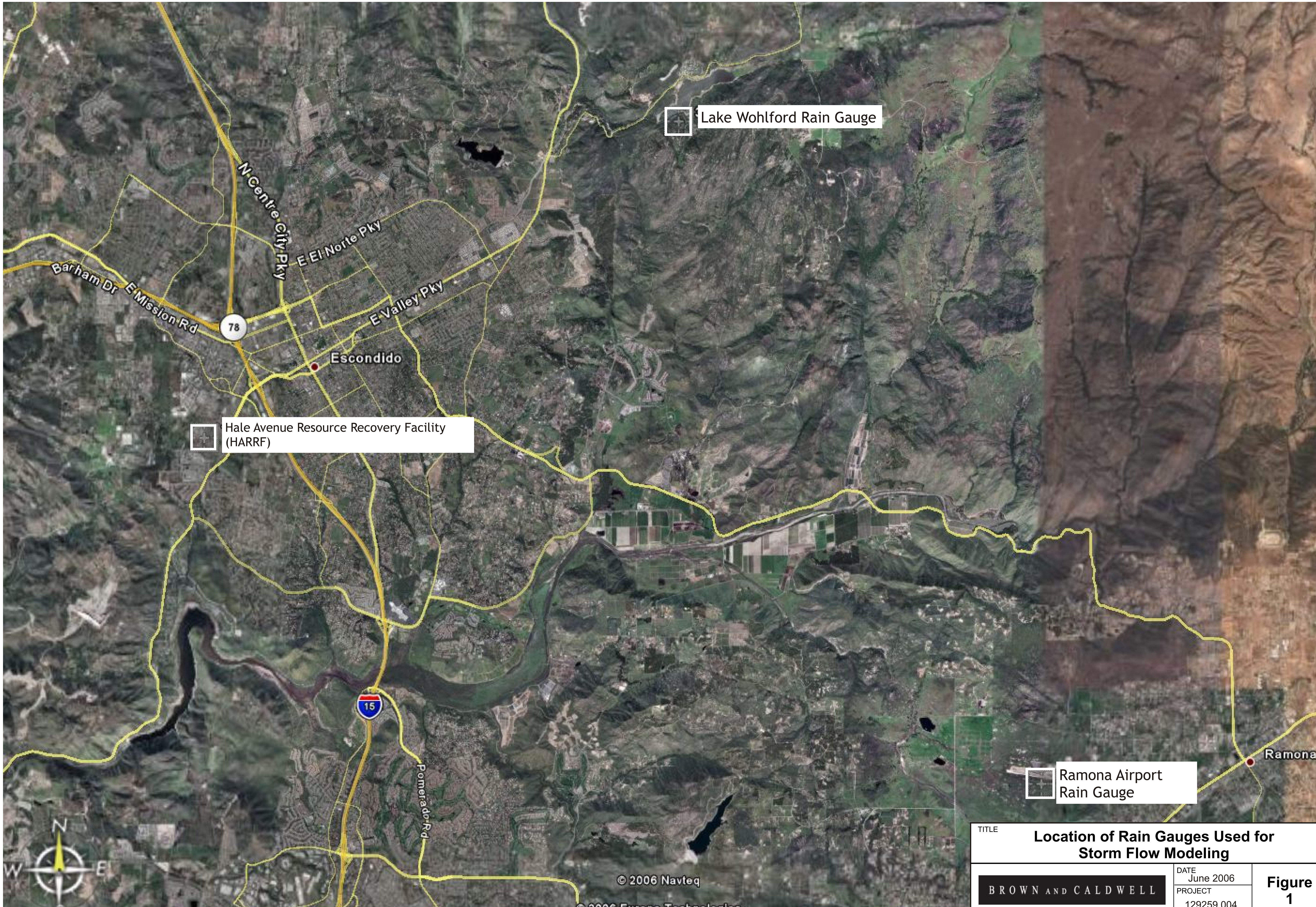
Table 10. Relationship between Storage Volume and Frequency of Intermittent Stream Discharge

Intermittent Stream Discharge Frequency (yrs)	Storage Volume[1] (MG)
2	2.0
3	2.5
4	3.0
5	5.0
10	12.0
20	43.5

Notes:
1. Rounded up to the nearest 0.5 MG.

The HARRF has a 2 MG equalization basin which can be used to store flow during wet weather events. Based on the results of this analysis, an intermittent stream discharge is expected to occur about once every 2 years on average.

Environmental Engineering And Consulting
9665 CHESAPEAKE DRIVE, SUITE 201, SAN DIEGO, CALIFORNIA 92123
Tel: 858. 514.8822 Fax: 858.514.8833



TITLE
Location of Rain Gauges Used for Storm Flow Modeling

BROWN AND CALDWELL

DATE June 2006		**Figure 1**
PROJECT 129259.004		

APPENDIX L

DETAILED COST ESTIMATE WORKSHEETS

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Near-Term ELO Improvements

	ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST	NOTES
	Structural					
	CCTV Entire ELO Line	71896	LF	$4	$287,584	
	Seal Manhole 74	1	EA	$95,000	$95,000	
	Seal inlet and outlet manholes local to siphons	12	EA	$95,000	$1,140,000	
	CONSTRUCTION COST			**Subtotal**	**$1,522,584**	
	Contractor General Conditions (7% of Construction Cost)				$106,581	
	Contractor OH/P (10% of Construction Cost)				$152,258	
	Sales Tax (7.75% of Construction Cost)				$118,000	
	Material Shipping and Handling (4% of Construction Cost)				$60,903	
	Worker's Travel Subsistance (0.01% of Construction Cost)				$152	
	Earthquake Insurance (0.1% of Construction Cost)				$1,523	
	Construction Contingency (50% of Construction Cost)				$761,292	
	Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$30,452	
	Escalation to Midpoint (8% of Construction Cost)				$121,807	
	Performance & Payment Bonds (2% of Construction Cost)				$30,452	
	TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$2,906,004**	
	Engineering & EIR (15% of Total Capital)				$435,901	
	Environmental Monitoring and Mitigation				$2,000,000	
	Construction Management (6% of Total Capital)				$174,360	
	Legal, And Administration (10% of Total Capital)				$290,600	
	TOTAL PROJECT COST				**$5,806,865**	

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for
Rehabilitation of Existing ELO

	ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST	NOTES
	Pipe Rehabilitation					
	CCTV Entire ELO Line	71896	LF	$4	$287,584	
	Insitu Form for 5 Miles of Existing ELO	26400	LF	$350	$9,240,000	
	Bypass Pumping	70	DAY	$12,000	$840,000	
	CCTV for QA/QC	26400	LF	$3	$66,000	
	Manhole Rehab	53	EA	$5,500	$184,800	
				Pipe Rehabilitation Subtotal	$10,618,384	
	CONSTRUCTION COST			**Subtotal**	**$10,618,384**	
	Contractor General Conditions (7% of Construction Cost)				$743,287	
	Contractor OH/P (10% of Construction Cost)				$1,061,838	
	Sales Tax (7.75% of Construction Cost)				$822,925	
	Material Shipping and Handling (4% of Construction Cost)				$424,735	
	Worker's Travel Subsistance (0.01% of Construction Cost)				$1,062	
	Earthquake Insurance (0.1% of Construction Cost)				$10,618	
	Construction Contingency (50% of Construction Cost)				$5,309,192	
	Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$212,368	
	Escalation to Midpoint (8% of Construction Cost)				$849,471	
	Performance & Payment Bonds (2% of Construction Cost)				$212,368	
	TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$20,266,248**	
	Engineering & EIR (15% of Total Capital)				$3,039,937	
	Environmental Monitoring and Mitigation				$500,000	
	Construction Management (6% of Total Capital)				$1,215,975	
	Legal, And Administration (10% of Total Capital)				$2,026,625	
	TOTAL PROJECT COST				**$27,048,784**	

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for
Abandonment of Existing ELO

	ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST	NOTES
	Pipe Abandonment					
	Sand Fill Existing Line	18271	CY	$50	$913,550	
	Demolish and Slurry Cap Manholes	74	EA	$1,500	$111,000	
				Pipe Abandonment Subtotal	$1,024,550	
	CONSTRUCTION COST			**Subtotal**	**$1,024,550**	
	Contractor General Conditions (7% of Construction Cost)				$71,719	
	Contractor OH/P (10% of Construction Cost)				$102,455	
	Sales Tax (7.75% of Construction Cost)				$79,403	
	Material Shipping and Handling (4% of Construction Cost)				$40,982	
	Worker's Travel Subsistance (0.01% of Construction Cost)				$102	
	Earthquake Insurance (0.1% of Construction Cost)				$1,025	
	Construction Contingency (50% of Construction Cost)				$512,275	
	Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$20,491	
	Escalation to Midpoint (8% of Construction Cost)				$81,964	
	Performance & Payment Bonds (2% of Construction Cost)				$20,491	
	TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$1,955,456**	
	Engineering & EIR (15% of Total Capital)				$293,318	
	Environmental Monitoring and Mitigation				$500,000	
	Construction Management (6% of Total Capital)				$117,327	
	Legal, And Administration (10% of Total Capital)				$195,546	
	TOTAL PROJECT COST				**$3,061,648**	

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for
New 42" land Outfall

	ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST	NOTES
	Structural					
	Excavation	325282 (1)	CY	$14	$4,602,747	
	Backfilling and compaction	255133 (1)	CY	$27	$6,786,547	
	Hauling	70149 (1)	CY	$36	$2,551,324	
				Structural Subtotal	$13,940,617	
	Piping					
	42 inch Standard Concrete Pipe, 100 psi (8 to 15)	79463 (1)	LF	$490	$38,936,723	Unit Cost includes Shoring and Sheeting
	42 inch Standard Concrete Pipe, 100 psi (>15' up to 18')	3217 (1)	LF	$590	$1,898,172	Unit Cost includes Shoring and Sheeting
	New 6 ft Ø manhole	100	EA	$10,000	$1,000,000	
				Piping Subtotal	$41,834,895	
	CONSTRUCTION COST			**Subtotal**	**$55,775,512**	
	Contractor General Conditions (7% of Construction Cost)				$3,904,286	
	Contractor OH/P (10% of Construction Cost)				$5,577,551	
	Sales Tax (7.75% of Construction Cost)				$4,322,602	
	Material Shipping and Handling (4% of Construction Cost)				$2,231,020	
	Worker's Travel Subsistance (0.01% of Construction Cost)				$5,578	
	Earthquake Insurance (0.1% of Construction Cost)				$55,776	
	Construction Contingency (50% of Construction Cost)				$27,887,756	
	Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,115,510	
	Escalation to Midpoint (8% of Construction Cost)				$4,462,041	
	Performance & Payment Bonds (2% of Construction Cost)				$1,115,510	
	TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$106,453,142**	
	Engineering & EIR (15% of Total Capital)				$15,967,971	
	Environmental Monitoring and Mitigation				$2,000,000	
	Construction Management (6% of Total Capital)				$6,387,189	
	Legal, And Administration (10% of Total Capital)				$10,645,314	
	TOTAL PROJECT COST				**$141,453,616**	

(1) Includes a 15% increase in length for deviations from existing alignment
(2) Includes costs for traffic control, property acquisition, etc.

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for
New 54" land Outfall

	ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST	NOTES
	Structural					
	Excavation	325864 (1)	CY	$14	$4,610,979	
	Backfilling and compaction	238325 (1)	CY	$27	$6,339,434	
	Hauling	87540 (1)	CY	$36	$3,183,817	
				Structural Subtotal	$14,134,230	
	Piping					
	54 inch Standard Concrete Pipe, 100 psi (8 to 15)	79463 (1)	LF	$625	$49,664,188	Unit Cost includes Shoring and Sheeting
	54 inch Standard Concrete Pipe, 100 psi (>15' up to 18')	3217 (1)	LF	$750	$2,412,930	Unit Cost includes Shoring and Sheeting
	New 6 ft Ø manhole	100	EA	$10,000	$1,000,000	
				Piping Subtotal	$53,077,118	
	CONSTRUCTION COST			**Subtotal**	**$67,211,347**	
	Contractor General Conditions (7% of Construction Cost)				$4,704,794	
	Contractor OH/P (10% of Construction Cost)				$6,721,135	
	Sales Tax (7.75% of Construction Cost)				$5,208,879	
	Material Shipping and Handling (4% of Construction Cost)				$2,688,454	
	Worker's Travel Subsistance (0.01% of Construction Cost)				$6,721	
	Earthquake Insurance (0.1% of Construction Cost)				$67,211	
	Construction Contingency (50% of Construction Cost)				$33,605,674	
	Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,344,227	
	Escalation to Midpoint (8% of Construction Cost)				$5,376,908	
	Performance & Payment Bonds (2% of Construction Cost)				$1,344,227	
	TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$128,279,577**	
	Engineering & EIR (15% of Total Capital)				$19,241,937	
	Environmental Monitoring and Mitigation				$2,000,000	
	Construction Management (6% of Total Capital)				$7,696,775	
	Legal, And Administration (10% of Total Capital)				$12,827,958	
	TOTAL PROJECT COST				**$170,046,246**	

(1) Includes a 15% increase in length for deviations from existing alignment
(2) Includes costs for traffic control, property acquisition, etc.

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for
New 72" land Outfall

	ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST	NOTES
	Structural					
	Excavation	326737 (1)	CY	$14	$4,623,327	
	Backfilling and compaction	205273 (1)	CY	$27	$5,460,258	
	Hauling	121464 (1)	CY	$36	$4,417,647	
				Structural Subtotal	$14,501,232	
	Piping					
	72 inch Standard Concrete Pipe, 100 psi (8 to 15)	79463 (1)	LF	$925	$73,502,998	Unit Cost includes Shoring and Sheeting
	72 inch Standard Concrete Pipe, 100 psi (>15' up to 18')	3217 (1)	LF	$1,075	$3,458,533	Unit Cost includes Shoring and Sheeting
	New 6 ft Ø manhole	100	EA	$10,000	$1,000,000	
				Piping Subtotal	$77,961,531	
	CONSTRUCTION COST			**Subtotal**	**$92,462,762**	
	Contractor General Conditions (7% of Construction Cost)				$6,472,393	
	Contractor OH/P (10% of Construction Cost)				$9,246,276	
	Sales Tax (7.75% of Construction Cost)				$7,165,864	
	Material Shipping and Handling (4% of Construction Cost)				$3,698,510	
	Worker's Travel Subsistance (0.01% of Construction Cost)				$9,246	
	Earthquake Insurance (0.1% of Construction Cost)				$92,463	
	Construction Contingency (50% of Construction Cost)				$46,231,381	
	Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,849,255	
	Escalation to Midpoint (8% of Construction Cost)				$7,397,021	
	Performance & Payment Bonds (2% of Construction Cost)				$1,849,255	
	TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$176,474,428**	
	Engineering & EIR (15% of Total Capital)				$26,471,164	
	Environmental Monitoring and Mitigation				$2,000,000	
	Construction Management (6% of Total Capital)				$10,588,466	
	Legal, And Administration (10% of Total Capital)				$17,647,443	
	TOTAL PROJECT COST				**$233,181,501**	

(1) Includes a 15% increase in length for deviations from existing alignment
(2) Includes costs for traffic control, property acquisition, etc.

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for

Near Term HARRF Improvements to Ensure 18.0 MGD Average Daily Flow Capacity

ITEM DESCRIPTION	UNIT	QUANTITY	TOTAL UNIT COST	TOTAL COST
CEPT				
CHEMICAL PUMPS FOR FERRIC	EA	2	$5,000	$10,000
FERRIC TANK. 9500 gal	EA	1	$13,625	$13,300
CHEMICAL PUMPS FOR POLYMER	EA	2	$2,500	$5,000
POLYMER DAY TANK,5000 gal	EA	2	$6,050	$12,100
Polyblend system	EA	1	$9,900	$9,900
CHEMICAL STORAGE STATION	SF	800	$147	$58,000
MIXER for Polymer Day Tank	EA	1	$1,546	$1,546
Additonal Air Injection to RAS Line				
Compressor and Pressurization System				$200,000
SVI Reduction				
Facilities currently exist to provide polymer injection at the splitter box, and chlorinate the RAS				$0
Piping				
Misc. piping and valves	15%	% of Mechanical		$46,477
Electrical and Instrumentation	22%	% of Mech, Piping, Conc.		$60,043
CONSTRUCTION RAW COST			**Subtotal**	$416,366
Contractor General Conditions (10% of Construction Cost)				$41,637
Contractor OH/P (12% of Construction Cost)				$49,964
Sales Tax (8.25% of Construction Cost)				$32,268
Material Shipping and Handling (2% of Construction Cost)				$8,327
Worker's Travel Subsistence (0.01% of Construction Cost)				$42
Start-up, Training & Contr. O & M (2% of Construction Cost)				$8,327
Earthquake Insurance (0.1% of Construction Cost)				$416
Construction Contingency (40% of Construction Cost)				$166,546
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$8,327
Escalation to Midpoint (8% of Construction Cost)				$33,309
Performance & Payment Bonds (2% of Construction Cost)				$8,327
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$773,858**
SCADA, E&I (10% of Total Capital)				$77,386
ENGINEERING, (20% of Total Capital)				$154,772
Construction Management (12% of Total Capital)				$77,386
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$77,386
Dewatering Centrifuge				
200 gpm centrifure from Trimax, monthly rental (two units- three times until Phase 1 expantion)	EA	6		$150,000
TOTAL PROJECT COST				**$1,310,787**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3A - High Rate CAS with BAF and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
37-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	600	CY	$14	$8,490
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	CY	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
BAF				
SITE CLEARING	0.5	ACRE	$9,276	$4,638
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Adding Aeration Basin-6				
HAULING	6500	CY	$36	$236,405
EXCAVATION	7540	CY	$14	$106,691
BACKFILL AND COMPACTION	1040	CY	$27	$27,664
CONCRETE	1100	CY	$895	$984,500
GEOTEXTILE FILTER FABRIC	10200	SF	$2	$20,400
SOLDIER PILES	9112	SF	$100	$911,200
ANOXIC SELECTOR WALL for 6 basin	75	CY	$895	$67,125
37-ft Diameter DAFTs				
HAULING	1350	CY	$36	$49,100
EXCAVATION	2730	CY	$14	$38,630
BACKFILL AND COMPACTION	1425	CY	$27	$37,905
CONCRETE	368	CY	$895	$328,913
SOLDIER PILES	3150	SF	$100	$315,000
RETAINING WALL	2400	SF	$130	$312,000
GRADING	195	SY	$2	$293
Paving	195	SY	$55	$10,696
Slurrycup/Grit Snail Unit				
CONCRETE	10	CY	$895	$8,950
HAULING	10	CY	$36	$364
EXCAVATION	20	CY	$14	$283
BACKFILL AND COMPACTION	10	CY	$27	$266
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
BAF				
HAULING	11100	CY	$36	$403,707
EXCAVATION	13500	CY	$14	$191,025
BACKFILL AND COMPACTION	2390	CY	$27	$63,574
CONCRETE	2800	CY	$895	$2,506,000
SOLDIER PILES	13200	SF	$100	$1,320,000
Fine Screens-BAF PS				
HAULING	325	CY	$36	$11,820
EXCAVATION	675	CY	$14	$9,551
BACKFILL AND COMPACTION	325	CY	$27	$8,645
CONCRETE	150	CY	$895	$134,250
SOLDIER PILES	2340	SF	$100	$234,000
109-ft Anaerobic Digester	1	LS		$1,632,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Converting to CEPT				
CHEMICAL PUMPS FOR FERRIC	2	EA	$5,000	$10,000
FERRIC TANK. 9500 gal	1	EA	$13,625	$13,625
CHEMICAL PUMPS FOR POLYMER	2	EA	$2,500	$5,000
POLYMER DAY TANK,5000 gal	2	EA	$6,050	$12,100
Polyblend system	1	EA	$9,900	$9,900
MIXER for Polymer Day Tank	1	EA	$1,546	$1,546
CHEMICAL STORAGE STATION	800	SF	$147	$58,000
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Adding Aeration Basin-6				
New Fine-Bubble Diffusers	6	Basin	$895,800	$1,410,297
Turblex Blower	1	EA	$889,000	$889,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
42 " High Guard Rail	1320	LF	$44	$58,648
Aluminum Grating	150	SF	$20	$3,060
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
Submersible Mixer	7	EA	$21,000	$147,000
37-ft Diameter DAFTs				
DAFT-37 ft diameter	2	LS	$132,425	$264,850
Thickened Sludge Pumps	4	EA	$35,714	$142,857
Pressurization System	4	EA	$104,286	$417,143
Slurrycup/Grit Snail Unit				
Equipment Cost	1	LS	$479,571	$479,571
Grit pump	2	ES	$16,667	$33,333

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3A - High Rate CAS with BAF and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS	$125,000	$125,000
New Carbon System	1	LS	$750,000	$750,000
BAF				
BIOFOR-N	1	LS	$4,142,857	$4,142,857
FINE SCREENS	1	LS	$463,680	$463,680
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
BAF Influent Pumps	2	EA	$145,714	$291,429
Backwash Pumps	2	EA	$21,429	$42,857
109-ft Anaerobic Digester				$6,528,610
Centrifuge	1	EA	$500,000	$500,000
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Screening Building	300	SF	$147	$45,165
BAF Influent Pump Station	600	SF	$147	$90,630
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	900	SF	$142	$127,170
New WAS Pump Station	900	SF	$142	$127,170
Piping				
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 36" IPS force main	300	LF	$345	$103,500
New 12"WAS pipe	LF	1000	$228	$227,500
Misc. piping and valves	15%	% of Mechanical		$2,973,804.93
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$5,092,708
CONSTRUCTION RAW COST			**Subtotal**	**$42,698,252**
Contractor General Conditions (10% of Construction Cost)				$4,269,825
Contractor OH/P (12% of Construction Cost)				$5,123,790
Sales Tax (7.75% of Construction Cost)				$3,309,115
Material Shipping and Handling (2% of Construction Cost)				$853,965
Worker's Travel Subsistence (0.01% of Construction Cost)				$4,270
Start-up, Training & Contr. O & M (2% of Construction Cost)				$853,965
Earthquake Insurance (0.1% of Construction Cost)				$42,698
Construction Contingency (40% of Construction Cost)				$17,079,301
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$853,965
Escalation to Midpoint (8% of Construction Cost)				$3,415,860
Performance & Payment Bonds (2% of Construction Cost)				$853,965
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$79,358,972**
SCADA, E&I (10% of Total Capital)				$7,935,897
ENGINEERING, (20% of Total Capital)				$15,871,794
Construction Management (10% of Total Capital)				$7,935,897
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$7,935,897
TOTAL PROJECT COST				**$119,038,457**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3A - High Rate CAS with BAF and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
36-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	600	CY	$14	$8,490
Grit Storage Building				
Storage Building	900	SF	$85	$76,500
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	LF	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
BAF				
SITE CLEARING	0.5	ACRE	$9,276	$4,638
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Grit Removal System				
HAULING	800	CY	$36	$29,096
EXCAVATION	900	CY	$14	$12,735
BACKFILL AND COMPACTION	100	CY	$27	$2,660
CONCRETE	230	CY	$895	$205,850
SOLDIER PILES	800	SF	$100	$80,000
Modifications to Grit Inlet Channel	1	LS		$10,000
Adding Aeration Basin-6				
HAULING	6500	CY	$36	$236,405
EXCAVATION	7540	CY	$14	$106,691
BACKFILL AND COMPACTION	1040	CY	$27	$27,664
CONCRETE	1100	CY	$895	$984,500
GEOTEXTILE FILTER FABRIC	10200	SF	$2	$20,400
SOLDIER PILES	9112	SF	$100	$911,200
ANOXIC SELECTOR WALL for 6 basin	75	CY	$895	$67,125
36-ft Diameter DAFT				
HAULING	395	CY	$36	$14,366
EXCAVATION	930	CY	$14	$13,160
BACKFILL AND COMPACTION	420	CY	$27	$11,172
CONCRETE	130	CY	$895	$116,350
SOLDIER PILES	1500	SF	$100	$150,000
RETAINING WALL	1600	SF	$130	$208,000
Paving	130	SY	$55	$7,131
GRADING	130	SY	$2	$195
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
BAF				
HAULING	11100	CY	$36	$403,707
EXCAVATION	13500	CY	$14	$191,025
BACKFILL AND COMPACTION	2390	CY	$27	$63,574
CONCRETE	2800	CY	$895	$2,506,000
SOLDIER PILES	13200	SF	$100	$1,320,000
Fine Screens-BAF PS				
HAULING	325	CY	$36	$11,820
EXCAVATION	675	CY	$14	$9,551
BACKFILL AND COMPACTION	325	CY	$27	$8,645
CONCRETE	150	CY	$895	$134,250
SOLDIER PILES	2340	SF	$100	$234,000
141-ft Anaerobic Digester				$2,731,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Grit Removal				
24-ft dia. Grit cyclone	1	EA	$28,571	$28,571
Grit Pump	2	EA	$16,667	$33,333
Grit classifier	1	EA	$58,333	$58,333
Parshall Flume				$5,000
Converting to CEPT				
CHEMICAL PUMPS FOR FERRIC	2	EA	$5,000	$10,000
FERRIC TANK. 9500 gal	1	EA	$13,625	$13,625
CHEMICAL PUMPS FOR POLYMER	2	EA	$2,500	$5,000
POLYMER DAY TANK,5000 gal	2	EA	$6,050	$12,100
Polyblend system	1	EA	$9,900	$9,900
MIXER for Polymer Day Tank	1	EA	$1,546	$1,546
CHEMICAL STORAGE STATION	800	SF	$147	$58,000
Adding Aeration Basin-6				
New Fine-Bubble Diffusers	6	Basin		$1,410,297
Turblex Blower	1	EA		$889,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
42 " High Guard Rail	1320	LF	$44	$58,648
Aluminum Grating	150	SF	$20	$3,060
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3A - High Rate CAS with BAF and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Submersible Mixer	7	EA	$21,000	$147,000
36-ft Diameter DAFT				
DAFT-36 ft diameter	1	LS	$131,165	$131,165
Thickened Sludge Pumps	2	EA	$28,250	$56,500
Pressurization System	1	EA	$85,000	$85,000
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
BAF				
BIOFOR-N	1	LS	$4,142,857	$4,142,857
FINE SCREENS	1	LS	$463,680	$463,680
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
BAF Influent Pumps	2	EA	$145,714	$291,429
Backwash Pumps	2	EA	$21,429	$42,857
141-ft Anaerobic Digester				$10,924,610
Centrifuge	2	EA	$500,000	$1,000,000
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Screening Building	300	SF	$147	$45,165
BAF Influent Pump Station	600	SF	$147	$90,630
Grit Storage Building				
Storage Building	900	SF	$142	$127,170
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	2000	SF	$142	$282,600
New WAS Pump Station	900	SF	$142	$127,170
Piping				
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 36" IPS force main	300	LF	$345	$103,500
New 12"WAS pipe	LF	1000	$228	$227,500
Misc. piping and valves	15%	% of Mechanical		$3,609,617
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$6,165,111
CONSTRUCTION RAW COST			**Subtotal**	**$49,579,033**
Contractor General Conditions (10% of Construction Cost)				$4,957,903
Contractor OH/P (12% of Construction Cost)				$5,949,484
Sales Tax (7.75% of Construction Cost)				$3,842,375
Material Shipping and Handling (2% of Construction Cost)				$991,581
Worker's Travel Subsistence (0.01% of Construction Cost)				$4,958
Start-up, Training & Contr. O & M (2% of Construction Cost)				$991,581
Earthquake Insurance (0.1% of Construction Cost)				$49,579
Construction Contingency (40% of Construction Cost)				$19,831,613
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$991,581
Escalation to Midpoint (8% of Construction Cost)				$3,966,323
Performance & Payment Bonds (2% of Construction Cost)				$991,581
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$92,147,591**
SCADA, E&I (10% of Total Capital)				$9,214,759
ENGINEERING, (20% of Total Capital)				$18,429,518
Construction Management (10% of Total Capital)				$9,214,759
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$9,214,759
TOTAL PROJECT COST				**$138,221,386**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3B - High Rate CAS with Microfiltration and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
37-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	600	CY	$14	$8,490
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	LF	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
MICROFILTER				
Taking out Dynasand filters and flocculation mixers	1.0	LS		$394,000
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
Adding Aeration Basin-6				
HAULING	6500	CY	$36	$236,405
EXCAVATION	7540	CY	$14	$106,691
BACKFILL AND COMPACTION	1040	CY	$27	$27,664
CONCRETE	1100	CY	$895	$984,500
GEOTEXTILE FILTER FABRIC	10200	SF	$2	$20,400
SOLDIER PILES	9112	SF	$100	$911,200
ANOXIC SELECTOR WALL for 6 basin	75	CY	$895	$67,125
37-ft Diameter DAFTs				
HAULING	1350	CY	$36	$49,100
EXCAVATION	2730	CY	$14	$38,630
BACKFILL AND COMPACTION	1425	CY	$27	$37,905
CONCRETE	368	CY	$895	$328,913
SOLDIER PILES	3150	SF	$100	$315,000
RETAINING WALL	2400	SF	$130	$312,000
GRADING	195	SY	$2	$293
Paving	195	SY	$55	$10,696
Slurrycup/Grit Snail Unit				
CONCRETE	10	CY	$895	$8,950
HAULING	10	CY	$36	$364
EXCAVATION	20	CY	$14	$283
BACKFILL AND COMPACTION	10	CY	$27	$266
109-ft Anaerobic Digester				$1,632,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Converting to CEPT				
CHEMICAL PUMPS FOR FERRIC	2	EA	$5,000	$10,000
FERRIC TANK. 9500 gal	1	EA	$13,625	$13,625
CHEMICAL PUMPS FOR POLYMER	2	EA	$2,500	$5,000
POLYMER DAY TANK,5000 gal	2	EA	$6,050	$12,100
Polyblend system	1	EA	$9,900	$9,900
MIXER for Polymer Day Tank	1	EA	$1,546	$1,546
CHEMICAL STORAGE STATION	800	SF	$147	$58,000
Adding Aeration Basin-6				
New Fine Bubble Diffusers	6	Basin		$1,410,297
Turblex Blower	1	EA		$889,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
42 " High Guard Rail	1320	LF	$44	$58,648
Aluminum Grating	150	SF	$20	$3,060
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
Submersible Mixer	7	EA	$21,000	$147,000
37-ft Diameter DAFTs				
DAFT-37 ft diameter	2	LS	$132,425	$264,850
Thickened Sludge Pumps	4	EA	$35,714	$142,857
Pressurization System	4	EA	$104,286	$417,143
Slurrycup/Grit Snail Unit				
Equipment Cost	1	LS	$479,571	$479,571
Grit pump	2	ES	$16,667	$33,333
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
MICROFILTER				
Microfilter	1	LS	$12,857,143	$12,857,143
109-ft Anaerobic Digester				$6,528,610
Centrifuge	1	EA	$500,000	$500,000

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3B - High Rate CAS with Microfiltration and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	900	SF	$142	$127,170
New WAS Pump Station	900	SF	$142	$127,170
Piping				
Misc. piping and valves	15%	% of Mechanical		$4,139,567
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 12"WAS pipe	LF	1000	$228	$227,500
New 36" IPS force main	300	LF	$345	$103,500
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$7,058,960
CONSTRUCTION RAW COST			**Subtotal**	**$49,108,806**
Contractor General Conditions (10% of Construction Cost)				$4,910,881
Contractor OH/P (12% of Construction Cost)				$5,893,057
Sales Tax (7.75% of Construction Cost)				$3,805,932
Material Shipping and Handling (2% of Construction Cost)				$982,176
Worker's Travel Subsistence (0.01% of Construction Cost)				$4,911
Start-up, Training & Contr. O & M (2% of Construction Cost)				$982,176
Earthquake Insurance (0.1% of Construction Cost)				$49,109
Construction Contingency (40% of Construction Cost)				$19,643,522
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$982,176
Escalation to Midpoint (8% of Construction Cost)				$3,928,704
Performance & Payment Bonds (2% of Construction Cost)				$982,176
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$91,273,626**
SCADA, E&I (10% of Total Capital)				$9,127,363
ENGINEERING, (20% of Total Capital)				$18,254,725
Construction Management (10% of Total Capital)				$9,127,363
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$9,127,363
TOTAL PROJECT COST				**$136,910,439**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3B - High Rate CAS with Microfiltration and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
36-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	600	CY	$14	$8,490
Grit Storage Building				
Storage Building	900	SF	$85	$76,500
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	LF	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
MICROFILTER				
Taking out Dynasand filters and flocculation mixers	1.0	LS		$394,000
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Grit Removal System				
HAULING	800	CY	$36	$29,096
EXCAVATION	900	CY	$14	$12,735
BACKFILL AND COMPACTION	100	CY	$27	$2,660
CONCRETE	230	CY	$895	$205,850
SOLDIER PILES	800	SF	$100	$80,000
Modifications to Grit Inlet Channel	1	LS		$10,000
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
Adding Aeration Basin-6				
HAULING	6500	CY	$36	$236,405
EXCAVATION	7540	CY	$14	$106,691
BACKFILL AND COMPACTION	1040	CY	$27	$27,664
CONCRETE	1100	CY	$895	$984,500
GEOTEXTILE FILTER FABRIC	10200	SF	$2	$20,400
SOLDIER PILES	9112	SF	$100	$911,200
ANOXIC SELECTOR WALL for 6 basin	75	CY	$895	$67,125
36-ft Diameter DAFT				
HAULING	395	CY	$36	$14,366
EXCAVATION	930	CY	$14	$13,160
BACKFILL AND COMPACTION	420	CY	$27	$11,172
CONCRETE	130	CY	$895	$116,350
SOLDIER PILES	1500	SF	$100	$150,000
RETAINING WALL	1600	SF	$130	$208,000
Paving	130	SY	$55	$7,131
GRADING	130	SY	$2	$195
141-ft Anaerobic Digester				$2,731,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Grit Removal				
24-ft dia. Grit cyclone	1	EA	$28,571	$28,571
Grit Pump	2	EA	$16,667	$33,333
Grit classifier	1	EA	$58,333	$58,333
Parshall Flume				$5,000
Converting to CEPT				
CHEMICAL PUMPS FOR FERRIC	2	EA	$5,000	$10,000
FERRIC TANK. 9500 gal	1	EA	$13,625	$13,625
CHEMICAL PUMPS FOR POLYMER	2	EA	$2,500	$5,000
POLYMER DAY TANK,5000 gal	2	EA	$6,050	$12,100
Polyblend system	1	EA	$9,900	$9,900
MIXER for Polymer Day Tank	1	EA	$1,546	$1,546
CHEMICAL STORAGE STATION	800	SF	$147	$58,000
Adding Aeration Basin-6				
New Fine Bubble Diffusers	6	Basin		$1,410,297
Turblex Blower	1	EA		$889,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
42 " High Guard Rail	1320	LF	$44	$58,648
Aluminum Grating	150	SF	$20	$3,060
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
Submersible Mixer	7	EA	$21,000	$147,000
36-ft Diameter DAFT				
DAFT-36 ft diameter	1	LS	$131,165	$131,165
Thickened Sludge Pumps	2	EA	$28,250	$56,500
Pressurization System	1	EA	$85,000	$85,000
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
MICROFILTER				
Microfilter	1	LS	$12,857,143	$12,857,143
141-ft Anaerobic Digester				$10,924,610
Centrifuge	2	EA	$500,000	$1,000,000

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3B - High Rate CAS with Microfiltration and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Grit Storage Building				
Storage Building	900	SF	$142	$127,170
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	2000	SF	$142	$282,600
New WAS Pump Station	900	SF	$142	$127,170
Piping				
Misc. piping and valves	15%	% of Mechanical		$4,775,380
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 12"WAS pipe	LF	1000	$228	$227,500
New 36" IPS force main	300	LF	$345	$103,500
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$8,131,363
CONSTRUCTION RAW COST			**Subtotal**	**$55,989,586**
Contractor General Conditions (10% of Construction Cost)				$5,598,959
Contractor OH/P (12% of Construction Cost)				$6,718,750
Sales Tax (7.75% of Construction Cost)				$4,339,193
Material Shipping and Handling (2% of Construction Cost)				$1,119,792
Worker's Travel Subsistence (0.01% of Construction Cost)				$5,599
Start-up, Training & Contr. O & M (2% of Construction Cost)				$1,119,792
Earthquake Insurance (0.1% of Construction Cost)				$55,990
Construction Contingency (40% of Construction Cost)				$22,395,834
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,119,792
Escalation to Midpoint (8% of Construction Cost)				$4,479,167
Performance & Payment Bonds (2% of Construction Cost)				$1,119,792
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$104,062,245**
SCADA, E&I (10% of Total Capital)				$10,406,224
ENGINEERING, (20% of Total Capital)				$20,812,449
Construction Management (10% of Total Capital)				$10,406,224
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$10,406,224
TOTAL PROJECT COST				**$156,093,367**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3C - High Rate CAS with MBR and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
37-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	600	CY	$14	$8,490
Odor Control				
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	CY	$50	$130,556
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
Adding MBR Tank				
HAULING	6500	CY	$36	$236,405
EXCAVATION	7540	CY	$14	$106,691
BACKFILL AND COMPACTION	1040	CY	$27	$27,664
CONCRETE	1100	CY	$895	$984,500
GEOTEXTILE FILTER FABRIC	10200	SF	$2	$20,400
SOLDIER PILES	9112	SF	$100	$911,200
ANOXIC SELECTOR WALL for 6 basin	74	CY	$895	$66,296
Fine Screens				
HAULING	100	CY	$36	$3,637
EXCAVATION	260	CY	$14	$3,679
BACKFILL AND COMPACTION	160	CY	$27	$4,256
CONCRETE	150	CY	$895	$134,250
SOLDIER PILES	960	SF	$100	$96,000
37-ft Diameter DAFTs				
HAULING	1350	CY	$36	$49,100
EXCAVATION	2730	CY	$14	$38,630
BACKFILL AND COMPACTION	1425	CY	$27	$37,905
CONCRETE	368	CY	$895	$328,913
SOLDIER PILES	3150	SF	$100	$315,000
RETAINING WALL	2400	1600	$130	$208,000
GRADING	195	SY	$2	$293
Paving	195	SY	$55	$10,696
Slurrycup/Grit Snail Unit				
CONCRETE	10	CY	$895	$8,950
HAULING	10	CY	$36	$364
EXCAVATION	20	CY	$14	$283
BACKFILL AND COMPACTION	10	CY	$27	$266
109-ft Anaerobic Digester				$1,632,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Converting to CEPT				
CHEMICAL PUMPS FOR FERRIC	2	EA	$5,000	$10,000
FERRIC TANK. 9500 gal	1	EA	$13,625	$13,625
CHEMICAL PUMPS FOR POLYMER	2	EA	$2,500	$5,000
POLYMER DAY TANK,5000 gal	2	EA	$6,050	$12,100
Polyblend system	1	EA	$9,900	$9,900
MIXER for Polymer Day Tank	1	EA	$1,546	$1,546
CHEMICAL STORAGE STATION	800	SF	$147	$58,000
Adding MBR Tank				
New Fine Bubble Diffusers	6	Basin		$1,410,297
42 " High Guard Rail	1320	LF	$44	$58,648
Aluminum Grating	150	SF	$20	$3,060
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
Submersible Mixer	6	EA	$21,000	$126,000
MBR	1	LS	$12,857,143	$12,857,143
FINE SCREENS	1	LS	$463,680	$463,680
Turblex Blower	1	EA		$889,000
MBR-RAS Pumps	4	EA	$166,667	$666,667
WAS Pumps	2	EA	$59,417	$118,833
Slurrycup/Grit Snail Unit				
Equipment Cost	1	LS	$479,571	$479,571
Grit pump	2	ES	$16,667	$33,333
37-ft Diameter DAFTs				
DAFT-37 ft diameter	2	LS	$132,425	$264,850
Thickened Sludge Pumps	4	EA	$35,714	$142,857
Pressurization System	4	EA	$104,286	$417,143
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
109-ft Anaerobic Digester				$6,528,610
Centrifuge	1	EA	$500,000	$500,000

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3C - High Rate CAS with MBR and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Building				
Screening Building	600	SF	$147	$90,330
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	900	SF	$142	$127,170
New WAS Pump Station	900	SF	$142	$127,170
Piping				
Misc. piping and valves	15%	% of Mechanical		$4,230,845
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 12"WAS pipe	LF	1000	$228	$227,500
New 36" IPS force main	300	LF	$345	$103,500
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$7,212,915
CONSTRUCTION RAW COST			**Subtotal**	**$49,705,548**
Contractor General Conditions (10% of Construction Cost)				$4,970,555
Contractor OH/P (12% of Construction Cost)				$5,964,666
Sales Tax (7.75% of Construction Cost)				$3,852,180
Material Shipping and Handling (2% of Construction Cost)				$994,111
Worker's Travel Subsistence (0.01% of Construction Cost)				$4,971
Start-up, Training & Contr. O & M (2% of Construction Cost)				$994,111
Earthquake Insurance (0.1% of Construction Cost)				$49,706
Construction Contingency (40% of Construction Cost)				$19,882,219
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$994,111
Escalation to Midpoint (8% of Construction Cost)				$3,976,444
Performance & Payment Bonds (2% of Construction Cost)				$994,111
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$92,382,731**
SCADA, E&I (10% of Total Capital)				$9,238,273
ENGINEERING, (20% of Total Capital)				$18,476,546
Construction Management (10% of Total Capital)				$9,238,273
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$9,238,273
TOTAL PROJECT COST				**$138,574,097**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3C - High Rate CAS with MBR and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
36-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	500	CY	$14	$7,075
Grit Storage Building				
Storage Building	900	SF	$85	$76,500
Odor Control				
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	CY	$50	$130,556
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Grit Removal System				
HAULING	800	CY	$36	$29,096
EXCAVATION	900	CY	$14	$12,735
BACKFILL AND COMPACTION	100	CY	$27	$2,660
CONCRETE	230	CY	$895	$205,850
SOLDIER PILES	800	SF	$100	$80,000
Modifications to Grit Inlet Channel	1	LS		$10,000
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
Adding MBR Tank				
HAULING	6500	CY	$36	$236,405
EXCAVATION	7540	CY	$14	$106,691
BACKFILL AND COMPACTION	1040	CY	$27	$27,664
CONCRETE	1100	CY	$895	$984,500
GEOTEXTILE FILTER FABRIC	10200	SF	$2	$20,400
SOLDIER PILES	9112	SF	$100	$911,200
ANOXIC SELECTOR WALL for 6 basin	74	CY	$895	$66,296
Fine Screens				
HAULING	100	CY	$36	$3,637
EXCAVATION	260	CY	$14	$3,679
BACKFILL AND COMPACTION	160	CY	$27	$4,256
CONCRETE	150	CY	$895	$134,250
SOLDIER PILES	960	SF	$100	$96,000
36-ft Diameter DAFT				
HAULING	395	CY	$36	$14,366
EXCAVATION	930	CY	$14	$13,160
BACKFILL AND COMPACTION	420	CY	$27	$11,172
CONCRETE	130	CY	$895	$116,350
SOLDIER PILES	1500	SF	$100	$150,000
RETAINING WALL	1600	SF	$130	$208,000
Paving	130	SY	$55	$7,131
GRADING	130	SY	$2	$195
141-ft Anaerobic Digester				$2,731,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Grit Removal				
24-ft dia. Grit cyclone	1	EA	$28,571	$28,571
Grit Pump	2	EA	$16,667	$33,333
Grit classifier	1	EA	$58,333	$58,333
Parshall Flume				$5,000
Converting to CEPT				
CHEMICAL PUMPS FOR FERRIC	2	EA	$5,000	$10,000
FERRIC TANK. 9500 gal	1	EA	$13,625	$13,625
CHEMICAL PUMPS FOR POLYMER	2	EA	$2,500	$5,000
POLYMER DAY TANK,5000 gal	2	EA	$6,050	$12,100
Polyblend system	1	EA	$9,900	$9,900
MIXER for Polymer Day Tank	1	EA	$1,546	$1,546
CHEMICAL STORAGE STATION	800	SF	$147	$58,000
Adding MBR Tank				
New Fine Bubble Diffusers	6	Basin		$1,410,297
42 " High Guard Rail	1320	LF	$44	$58,648
Aluminum Grating	150	SF	$20	$3,060
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
Submersible Mixer	6	EA	$21,000	$126,000
MBR	1	LS	$12,857,143	$12,857,143
FINE SCREENS	1	LS	$463,680	$463,680
Turblex Blower	1	EA		$889,000
MBR-RAS Pumps	4	EA	$166,667	$666,667
WAS Pumps	2	EA	$59,417	$118,833
36-ft Diameter DAFT				
DAFT-36 ft diameter	1	LS	$131,165	$131,165
Thickened Sludge Pumps	2	EA	$28,250	$56,500
Pressurization System	2	EA	$85,000	$170,000

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 3C - High Rate CAS with MBR and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Thickening Centrifuge				
Centrifuge	2	EA	$875,000	$1,750,000
Dewatered Sludge Pumps	4	EA	$28,571	$114,286
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
141-ft Anaerobic Digester				$10,924,610
Centrifuge	2	EA	$500,000	$1,000,000
Building				
Screening Building	600	SF	$147	$90,330
Centrifuge Building	900	SF	$147	$135,495
Grit Storage Building				
Storage Building	900	SF	$142	$127,170
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	2000	SF	$142	$282,600
New WAS Pump Station	900	SF	$142	$127,170
Piping				
Misc. piping and valves	15%	% of Mechanical		$5,179,374
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 12"WAS pipe	LF	1000	$228	$227,500
New 36" IPS force main	300	LF	$345	$103,500
Electrical and Instrumentation	22%	% of Mech, Piping		$8,812,768
CONSTRUCTION RAW COST			**Subtotal**	**$59,613,861**
Contractor General Conditions (10% of Construction Cost)				$5,961,386
Contractor OH/P (12% of Construction Cost)				$7,153,663
Sales Tax (7.75% of Construction Cost)				$4,620,074
Material Shipping and Handling (2% of Construction Cost)				$1,192,277
Worker's Travel Subsistence (0.01% of Construction Cost)				$5,961
Start-up, Training & Contr. O & M (2% of Construction Cost)				$1,192,277
Earthquake Insurance (0.1% of Construction Cost)				$59,614
Construction Contingency (40% of Construction Cost)				$23,845,544
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,192,277
Escalation to Midpoint (8% of Construction Cost)				$4,769,109
Performance & Payment Bonds (2% of Construction Cost)				$1,192,277
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$110,798,322**
SCADA, E&I (10% of Total Capital)				$11,079,832
ENGINEERING, (20% of Total Capital)				$22,159,664
Construction Management (10% of Total Capital)				$11,079,832
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$11,079,832
TOTAL PROJECT COST				**$166,197,482**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6A - MBBR with BAF and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2	$104,000
37-ft Diameter DAFT				
Paving	126	SY	$15	$1,915
EXCAVATION	600	CY	$14	$8,490
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	CY	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
BAF				
SITE CLEARING	0.5	ACRE	$9,276	$4,638
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Aeration Basin Improvements				
Seperation Wall for 5 basin	85	CY	$895	$76,075
37-ft Diameter DAFTs				
HAULING	1350	CY	$36	$49,100
EXCAVATION	2730	CY	$14	$38,630
BACKFILL AND COMPACTION	1425	CY	$27	$37,905
CONCRETE	368	CY	$895	$328,913
SOLDIER PILES	3150	SF	$100	$315,000
RETAINING WALL	2400	SF	$130	$312,000
GRADING	195	SY	$2	$293
Paving	195	SY	$55	$10,696
Slurrycup/Grit Snail Unit				
CONCRETE	10	CY	$895	$8,950
HAULING	10	CY	$36	$364
EXCAVATION	20	CY	$14	$283
BACKFILL AND COMPACTION	10	CY	$27	$266
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
BAF				
HAULING	11100	CY	$36	$403,707
EXCAVATION	13500	CY	$14	$191,025
BACKFILL AND COMPACTION	2390	CY	$27	$63,574
CONCRETE	2800	CY	$895	$2,506,000
SOLDIER PILES	13200	SF	$100	$1,320,000
Fine Screens-BAF PS				
HAULING	325	CY	$36	$11,820
EXCAVATION	675	CY	$14	$9,551
BACKFILL AND COMPACTION	325	CY	$27	$8,645
CONCRETE	150	CY	$895	$134,250
SOLDIER PILES	2340	SF	$100	$234,000
109-ft Anaerobic Digester	1	LS		$1,632,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Aeration Basin Improvements				
MBBR system	1	LS	$4,107,143	$4,107,143
New Fine-Bubble Diffusers	5	Basin		$973,776
Turblex Blower	1	EA	$889,000	$889,000
Modifications to RAS pipe	1	LS		$100,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
37-ft Diameter DAFTs				
DAFT-37 ft diameter	2	LS	$132,425	$264,850
Thickened Sludge Pumps	4	EA	$35,714	$142,857
Pressurization System	4	EA	$104,286	$417,143
Slurrycup/Grit Snail Unit				
Equipment Cost	1	LS	$479,571	$479,571
Grit pump	2	ES	$16,667	$33,333
Odor Control				
ALUMINUM COVERS	28305	SF	$45	$1,273,725
New Blowers for PSBs, ventilators	1	LS	$125,000	$125,000
New Carbon System	1	LS	$750,000	$750,000
BAF				
BIOFOR-N	1	LS	$4,142,857	$4,142,857
FINE SCREENS	1	LS	$463,680	$463,680
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
BAF Influent Pumps	2	EA	$145,714	$291,429
Backwash Pumps	2	EA	$21,429	$42,857
109-ft Anaerobic Digester				$6,528,610
Centrifuge	1	EA	$500,000	$500,000

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6A - MBBR with BAF and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Screening Building	300	SF	$147	$45,165
BAF Influent Pump Station	600	SF	$147	$90,630
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	900	SF	$142	$127,170
New WAS Pump Station	900	SF	$142	$127,170
Piping				
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 36" IPS force main	300	LF	$345	$103,500
New 12"WAS pipe	LF	1000	$228	$227,500
Misc. piping and valves	15%	% of Mechanical		$3,491,566.50
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$5,965,999
CONSTRUCTION RAW COST			**Subtotal**	**$45,263,139**
Contractor General Conditions (10% of Construction Cost)				$4,526,314
Contractor OH/P (12% of Construction Cost)				$5,431,577
Sales Tax (7.75% of Construction Cost)				$3,507,893
Material Shipping and Handling (2% of Construction Cost)				$905,263
Worker's Travel Subsistence (0.01% of Construction Cost)				$4,526
Start-up, Training & Contr. O & M (2% of Construction Cost)				$905,263
Earthquake Insurance (0.1% of Construction Cost)				$45,263
Construction Contingency (40% of Construction Cost)				$18,105,256
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$905,263
Escalation to Midpoint (8% of Construction Cost)				$3,621,051
Performance & Payment Bonds (2% of Construction Cost)				$905,263
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$84,126,070**
SCADA, E&I (10% of Total Capital)				$8,412,607
ENGINEERING, (20% of Total Capital)				$16,825,214
Construction Management (10% of Total Capital)				$8,412,607
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$8,412,607
TOTAL PROJECT COST				**$126,189,105**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6A - MBBR with BAF and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2.08	$104,000
36-ft Diameter DAFT				
Paving	126	SY	$15.25	$1,915
EXCAVATION	600	CY	$14.15	$8,490
Grit Storage Building				
Storage Building	900	SF	$85	$76,500
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	LF	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
BAF				
SITE CLEARING	0.5	ACRE	$9,276	$4,638
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14	$4,670
BACKFILL AND COMPACTION	230	CY	$27	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36	$125,477
EXCAVATION	3650	CY	$14	$51,648
BACKFILL AND COMPACTION	200	CY	$27	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2.00	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Grit Removal System				
HAULING	800	CY	$36	$29,096
EXCAVATION	900	CY	$14	$12,735
BACKFILL AND COMPACTION	100	CY	$27	$2,660
CONCRETE	230	CY	$895	$205,850
SOLDIER PILES	800	SF	$100	$80,000
Modifications to Grit Inlet Channel	1	LS		$10,000
Adding Aeration Basin-6				
Seperation Wall for 5 basin	85	CY	$895	$76,075
36-ft Diameter DAFT				
HAULING	395	CY	$36	$14,366
EXCAVATION	930	CY	$14	$13,160
BACKFILL AND COMPACTION	420	CY	$27	$11,172
CONCRETE	130	CY	$895	$116,350
SOLDIER PILES	1500	SF	$100	$150,000
RETAINING WALL	1600	SF	$130	$208,000
Paving	130	SY	$55	$7,131
GRADING	130	SY	$1.50	$195
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
BAF				
HAULING	11100	CY	$36	$403,707
EXCAVATION	13500	CY	$14	$191,025
BACKFILL AND COMPACTION	2390	CY	$27	$63,574
CONCRETE	2800	CY	$895	$2,506,000
SOLDIER PILES	13200	SF	$100	$1,320,000
Fine Screens-BAF PS				
HAULING	325	CY	$36.37	$11,820
EXCAVATION	675	CY	$14.15	$9,551
BACKFILL AND COMPACTION	325	CY	$26.60	$8,645
CONCRETE	150	CY	$895	$134,250
SOLDIER PILES	2340	SF	$100	$234,000
141-ft Anaerobic Digester				$2,731,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44.43	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Grit Removal				
24-ft dia. Grit cyclone	1	EA	$28,571	$28,571
Grit Pump	2	EA	$16,667	$33,333
Grit classifier	1	EA	$58,333	$58,333
Parshall Flume				$5,000
Aeration Basin Improvements				
MBBR system	1	LS	$4,107,143	$4,107,143
New Fine Bubble Diffusers	Basin	5		$973,776
Turblex Blower	1	EA		$889,000
Modifications to RAS pipe	1	LS		$100,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
36-ft Diameter DAFT				
DAFT-36 ft diameter	1	LS	$131,165	$131,165
Thickened Sludge Pumps	2	EA	$28,250	$56,500
Pressurization System	1	EA	$85,000	$85,000
Odor Control				
ALUMINUM COVERS	28305	SF	$45.00	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
BAF				
BIOFOR-N	1	LS	$4,142,857	$4,142,857
FINE SCREENS	1	LS	$463,680	$463,680
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
BAF Influent Pumps	2	EA	$145,714	$291,429
Backwash Pumps	2	EA	$21,429	$42,857

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6A - MBBR with BAF and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
141-ft Anaerobic Digester				$10,924,610
Centrifuge	2	EA	$500,000	$1,000,000
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Screening Building	300	SF	$147	$45,165
BAF Influent Pump Station	600	SF	$147	$90,630
Grit Storage Building				
Storage Building	900	SF	$142	$127,170
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	2000	SF	$142	$282,600
New WAS Pump Station	900	SF	$142	$127,170
Piping				
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 36" IPS force main	300	LF	$345	$103,500
New 12"WAS pipe	LF	1000	$228	$227,500
Misc. piping and valves	15%	% of Mechanical		$4,127,379
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$7,038,402
CONSTRUCTION RAW COST			**Subtotal**	**$52,143,919**
Contractor General Conditions (10% of Construction Cost)				$5,214,392
Contractor OH/P (12% of Construction Cost)				$6,257,270
Sales Tax (7.75% of Construction Cost)				$4,041,154
Material Shipping and Handling (2% of Construction Cost)				$1,042,878
Worker's Travel Subsistence (0.01% of Construction Cost)				$5,214
Start-up, Training & Contr. O & M (2% of Construction Cost)				$1,042,878
Earthquake Insurance (0.1% of Construction Cost)				$52,144
Construction Contingency (40% of Construction Cost)				$20,857,568
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,042,878
Escalation to Midpoint (8% of Construction Cost)				$4,171,514
Performance & Payment Bonds (2% of Construction Cost)				$1,042,878
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$96,914,689**
SCADA, E&I (10% of Total Capital)				$9,691,469
ENGINEERING, (20% of Total Capital)				$19,382,938
Construction Management (10% of Total Capital)				$9,691,469
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$9,691,469
TOTAL PROJECT COST				**$145,372,033**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6B - MBBR with Microfiltration and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2.08	$104,000
37-ft Diameter DAFT				
Paving	126	SY	$15.25	$1,915
EXCAVATION	600	CY	$14.15	$8,490
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85	$1,997,500
HAULING DEBRIS	2611	CY	$20	$51,569
TIPPING FEE	2611	LF	$50	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
MICROFILTER				
Taking out Dynasand filters and flocculation mixers	1.0	LS		$394,000
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14.15	$4,670
BACKFILL AND COMPACTION	230	CY	$26.60	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36.37	$125,477
EXCAVATION	3650	CY	$14.15	$51,648
BACKFILL AND COMPACTION	200	CY	$26.60	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2.00	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
Aeration Basin Improvements				
Seperation Wall for 5 basin	85	CY	$895	$76,075
37-ft Diameter DAFTs				
HAULING	1350	CY	$36.37	$49,100
EXCAVATION	2730	CY	$14.15	$38,630
BACKFILL AND COMPACTION	1425	CY	$26.60	$37,905
CONCRETE	368	CY	$895	$328,913
SOLDIER PILES	3150	SF	$100	$315,000
RETAINING WALL	2400	SF	$130	$312,000
GRADING	195	SY	$1.50	$293
Paving	195	SY	$55	$10,696
Slurrycup/Grit Snail Unit				
CONCRETE	10	CY	$895	$8,950
HAULING	10	CY	$36	$364
EXCAVATION	20	CY	$14	$283
BACKFILL AND COMPACTION	10	CY	$27	$266
109-ft Anaerobic Digester				$1,632,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44.43	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Aeration Basin Improvements				
MBBR system	1	LS	$4,107,143	$4,107,143
New Fine Bubble Diffusers	5	Basin		$973,776
Turblex Blower	1	EA		$889,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
37-ft Diameter DAFTs				
DAFT-37 ft diameter	2	LS	$132,425	$264,850
Thickened Sludge Pumps	4	EA	$35,714	$142,857
Pressurization System	4	EA	$104,286	$417,143
Slurrycup/Grit Snail Unit				
Equipment Cost	1	LS	$479,571	$479,571
Grit pump	2	ES	$16,667	$33,333
Odor Control				
ALUMINUM COVERS	28305	SF	$45.00	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
MICROFILTER				
Microfilter	1	LS	$12,857,143	$12,857,143
109-ft Anaerobic Digester				$6,528,610
Centrifuge	1	EA	$500,000	$500,000
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	900	SF	$142	$127,170
New WAS Pump Station	900	SF	$142	$127,170
Piping				
Misc. piping and valves	15%	% of Mechanical		$4,642,329
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 12"WAS pipe	LF	1000	$228	$227,500
New 36" IPS force main	300	LF	$345	$103,500
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$7,906,951

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6B - MBBR with Microfiltration and Sludge Co-thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
CONSTRUCTION RAW COST			**Subtotal**	**$51,533,392**
Contractor General Conditions (10% of Construction Cost)				$5,153,339
Contractor OH/P (12% of Construction Cost)				$6,184,007
Sales Tax (7.75% of Construction Cost)				$3,993,838
Material Shipping and Handling (2% of Construction Cost)				$1,030,668
Worker's Travel Subsistence (0.01% of Construction Cost)				$5,153
Start-up, Training & Contr. O & M (2% of Construction Cost)				$1,030,668
Earthquake Insurance (0.1% of Construction Cost)				$51,533
Construction Contingency (40% of Construction Cost)				$20,613,357
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,030,668
Escalation to Midpoint (8% of Construction Cost)				$4,122,671
Performance & Payment Bonds (2% of Construction Cost)				$1,030,668
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$95,779,963**
SCADA, E&I (10% of Total Capital)				$9,577,996
ENGINEERING, (20% of Total Capital)				$19,155,993
Construction Management (10% of Total Capital)				$9,577,996
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$9,577,996
TOTAL PROJECT COST				**$143,669,944**

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6B - MBBR with Microfiltration and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Demolition				
MOB AND DEMOB	1	LS		$36,000
Influent Pump Station				
Remove welded 30 inch header pipe	1	LS		$12,250
Remove Manual Bar Screen	1	EA	$10,000	$10,000
Removing 30" IPS force main	300	LF	$103	$30,900
Anaerobic Selector Zone				
REMOVING AERATION PANELS	50000	SF	$2.08	$104,000
36-ft Diameter DAFT				
Paving	126	SY	$15.25	$1,915
EXCAVATION	600	CY	$14.15	$8,490
Grit Storage Building				
Storage Building	900	SF	$85.00	$76,500
Odor Control				
PRIMARY CLARIFIER BLDG.	23500	SF	$85.00	$1,997,500
HAULING DEBRIS	2611	CY	$19.75	$51,569
TIPPING FEE	2611	LF	$50.00	$130,556
60" FRP DUCT	160	LF	$220	$35,200
42 " FRP DUCT	250	LF	$200	$50,000
30" FRP DUCT	75	LF	$170	$12,750
CAUSTIC SCRUBBERS	2	EA	$13,750	$27,500
CAUSTIC PUMPS	3	EA	$1,500	$4,500
FANS	2	EA	$11,000	$22,000
BIOTOWERS	1	LS		$113,234
Salvage Existing Feed Pumps and Sump Pumps for Biotowers	1	LS		$17,500
MICROFILTER				
Taking out Dynasand filters and flocculation mixers	1.0	LS		$394,000
Structural				
Influent Pump Station				
Concrete filling to screen channel	5	CY	$895	$4,833
Structural improvements for new screen installation		LS		$2,000
Excavation to remove 30" pipe	330	CY	$14.15	$4,670
BACKFILL AND COMPACTION	230	CY	$26.60	$6,118
Adding Primary Clarifier-5				
HAULING	3450	CY	$36.37	$125,477
EXCAVATION	3650	CY	$14.15	$51,648
BACKFILL AND COMPACTION	200	CY	$26.60	$5,320
CONCRETE	620	CY	$895	$554,900
GEOTEXTILE FILTER FABRIC	8250	SF	$2.00	$16,500
SOLDIER PILES	8200	SF	$100	$820,000
Hydraulic Improvements				
Structural openings for new AB gates	48	EA	$1,850	$88,800
Grit Removal System				
HAULING	800	CY	$36.37	$29,096
EXCAVATION	900	CY	$14.15	$12,735
BACKFILL AND COMPACTION	100	CY	$26.60	$2,660
CONCRETE	230	CY	$895	$205,850
SOLDIER PILES	800	SF	$100	$80,000
Modifications to Grit Inlet Channel	1	LS		$10,000
New WAS PS				
SOLDIER PILES	1350	SF	$100	$135,000
Aeration Basin Improvements				
Seperation Wall for 5 basin	85	CY	$895	$76,075
36-ft Diameter DAFT				
HAULING	395	CY	$36.37	$14,366
EXCAVATION	930	CY	$14.15	$13,160
BACKFILL AND COMPACTION	420	CY	$26.60	$11,172
CONCRETE	130	CY	$895	$116,350
SOLDIER PILES	1500	SF	$100	$150,000
RETAINING WALL	1600	SF	$130	$208,000
Paving	130	SY	$55	$7,131
GRADING	130	SY	$2	$195
141-ft Anaerobic Digester				$2,731,152
Mechanical				
Influent Pump Station				
Mechanical screen, conveyor, compactor	1	LS	$300,000	$300,000
Adding Primary Clarifier-5				
42 " High Guard Rail	250	LF	$44.43	$11,108
CHAIN AND FLIGHT SYSTEM	1	LS	$147,060	$147,060
PRIMARY SLUDGE PUMPS	2	EA	$31,429	$69,143
Hydraulic Improvements				
1ftX1ft Gate Valve	48	EA	$1,794	$86,112
Grit Removal				
24-ft dia. Grit cyclone	1	EA	$28,571	$28,571
Grit Pump	2	EA	$16,667	$33,333
Grit classifier	1	EA	$58,333	$58,333
Parshall Flume				$5,000
Aeration Basin Improvements				
MBBR system	1	LS	$4,107,143	$4,107,143
New Fine Bubble Diffusers	5	Basin		$973,776
Turblex Blower	1	EA		$889,000
RAS Pumps	3	EA	$75,000	$225,000
WAS Pumps	2	EA	$59,417	$118,833
1ftX1ft Gate Valve	4	EA	$1,794	$7,176
1.5ft X 1.5ft Gate Valve	4	EA	$2,194	$8,776
36-ft Diameter DAFT				
DAFT-36 ft diameter	1	LS	$131,165	$131,165
Thickened Sludge Pumps	2	EA	$28,250	$56,500
Pressurization System	1	EA	$85,000	$85,000
Odor Control				
ALUMINUM COVERS	28305	SF	$45.00	$1,273,725
New Blowers for PSBs, ventilators	1	LS		$125,000
New Carbon System	1	LS		$750,000
MICROFILTER				
Microfilter	1	LS	$12,857,143	$12,857,143
141-ft Anaerobic Digester				$10,924,610
Centrifuge	2	EA	$500,000	$1,000,000

Wastewater Treatment and Disposal Facilities Capacity Study

Planning-Level Cost Estimate for Long Term Improvements

Alternative 6B - MBBR with Microfiltration and Sludge Separate Thickening

ITEM DESCRIPTION	QUANTITY	UNIT	TOTAL UNIT COST	TOTAL COST
Building				
Thickened Sludge PS and Pressurization Bldg.	1600	SF	$279	$366,160
Grit Storage Building				
Storage Building	900	SF	$142	$127,170
Influent Pump Station				
10'x10'x10' Fiberglass enclosure	1	LS		$10,000
7' high x 6' wide double leaf door	1	LS		$3,000
Centrifuge Building				
Building Modifications	2000	SF	$142	$282,600
New WAS Pump Station	900	SF	$142	$127,170
Piping				
Misc. piping and valves	15%	% of Mechanical		$5,278,141
New 36" steel welded header pipe	1	LS	$18,500	$18,500
New 12"WAS pipe	LF	1000	$228	$227,500
New 36" IPS force main	300	LF	$345	$103,500
Electrical and Instrumentation	22%	% of Mech,Build,Piping		$8,979,355
CONSTRUCTION RAW COST			**Subtotal**	**$58,414,173**
Contractor General Conditions (10% of Construction Cost)				$5,841,417
Contractor OH/P (12% of Construction Cost)				$7,009,701
Sales Tax (7.75% of Construction Cost)				$4,527,098
Material Shipping and Handling (2% of Construction Cost)				$1,168,283
Worker's Travel Subsistence (0.01% of Construction Cost)				$5,841
Start-up, Training & Contr. O & M (2% of Construction Cost)				$1,168,283
Earthquake Insurance (0.1% of Construction Cost)				$58,414
Construction Contingency (40% of Construction Cost)				$23,365,669
Builders Risk, Liability & Auto Ins. (2% of Construction Cost)				$1,168,283
Escalation to Midpoint (8% of Construction Cost)				$4,673,134
Performance & Payment Bonds (2% of Construction Cost)				$1,168,283
TOTAL CONSTRUCTION CAPITAL			**Subtotal**	**$108,568,582**
SCADA, E&I (10% of Total Capital)				$10,856,858
ENGINEERING, (20% of Total Capital)				$21,713,716
Construction Management (10% of Total Capital)				$10,856,858
LEGAL, AND ADMINISTRATION (10% of Total Capital)				$10,856,858
TOTAL PROJECT COST				**$162,852,872**

APPENDIX M

OFF-GAS TESTING REPORT

REDMON
ENGINEERING
COMPANY

Consulting Engineers

PO Box 044258

Racine, Wisconsin 53404-7005

(262) 681-0100
FAX: (262) 681-0303

July 24, 2006

Brown & Caldwell
Attn: Ron Appleton
201 North Civic Drive
Walnut Creek, CA 94596

Re: Hale Avenue Resource Recovery Facility - Report of Full Scale Offgas Analysis

Dear Ron,

Attached please find my report of the full-scale offgas tests conducted April 25 to 27, 2006 on the Parkson fine pore membrane panel aeration system at the Hale Avenue Resource Recovery Facility (HARRF), in Escondido, California.

Following your review, should you have any questions, please let me know.

Best regards,

REDMON ENGINEERING COMPANY

David Redmon

David T. Redmon, P.E.

Cc: Victor Occiano

FULL SCALE OFFGAS ANALYSIS
OF
AERATION BASINS 1, 3, AND 4
AT THE
HALE AVENUE RESOURCE RECOVERY FACILITY
ESCONDIDO, CALIFORNIA

PERFORMED ON BEHALF OF:

BROWN & CALDWELL

CONDUCTED

April 2006

PERFORMED BY:

REDMON ENGINEERING COMPANY

PO Box 044258
Racine, Wisconsin 53404

(262) 681-0100

REDMON
ENGINEERING COMPANY

FULL SCALE OFFGAS ANALYSIS
OF THE
PARKSON FINE PORE MEMBRANE PANEL AERATION SYSTEM
AT
HALE AVENUE RESOURCE RECOVERY FACILITY

April 2006

INTRODUCTION

Redmon Engineering Company was engaged by Brown and Caldwell to conduct a series of full-scale offgas analyses on the Parkson fine pore membrane panel aeration system at the Hale Avenue Resource Recovery Facility in Escondido, California. The objective of the offgas evaluation was to observe the oxygen transfer performance of the system in an effort to assess the capacity of the aeration system.

David Redmon of Redmon Engineering Company conducted the on-site tests from April 25 to 27, 2006.

BACKGROUND

The full-scale test involves placing a floating offgas collection device on the liquid surface of the basin(s) in question at various locations and to analyze the exiting gas for the partial pressure of oxygen compared to that of ambient air. In addition, the rate of offgas evolution is typically measured for each offgas collection hood sampling position employed and each test condition. These data are analyzed according to the procedures described in the paper, "Oxygen Transfer Efficiency Measurements in Mixed Liquor Using Offgas Techniques," by Redmon, et al. (WPCF November, 1983) and the ASCE "Standard Guidelines for In-Process Oxygen Transfer Testing".

Aeration System

The aeration system at the Hale Avenue Resource Recovery Facility (HARRF) consists of five aeration basins. All five basins were retrofitted with Parkson Panel ultra fine bubble panels in 2000. Each aeration basin is 50 feet wide by 200 feet in plan view, and operates at a side water depth of 16.5 feet. The primary effluent enters into an anoxic selector zone, which spans across the width of the basins and occupies 12.5 feet of basin length and is separated from the rest of the basin by a baffle. A single coarse bubble header mixes the anoxic zone. The remainder of each basin is operated as an aerobic reactor and is aerated by Parkson Panel diffusion units.

Figures 1 and 2 show the panel layout for each of the five aeration basins. At the time the offgas tests were conducted all the basins were operating in the plug-flow mode.

Test Parameters

The full-scale tests were conducted to measure oxygen transfer efficiency, alpha factor, and oxygen transfer rates under actual operating conditions.

Manufacturers of aeration systems typically quote performance based on clean water oxygen transfer test results. To compare data, the tests should best be conducted in large-scale tanks in accordance with standard procedures (ASCE Clean Water Test Standard, 1992). For a given basin geometry, diffuser type and layout, aeration equipment manufacturers can provide acceptable estimates of clean water standard oxygen transfer efficiency (SOTE) and equilibrium dissolved oxygen (DO) concentration at standard conditions as time approaches infinity ($C^*_{\infty 20}$). Standard conditions of temperature and pressure are 20°C and 1.0 atmosphere of pressure (29.92 in Hg or 760 mm Hg), respectively.

To estimate the oxygen transfer efficiency in the process water under actual operating conditions, the following equation is used (ASCE, 1992):

$$OTE_F = \alpha(SOTE)(\Theta^{T-20})(\Upsilon\Omega\beta C^*_{\infty 20} - C) / C^*_{\infty 20}$$

Where:

OTE_F = Process water oxygen transfer efficiency, mass fraction of oxygen transferred per unit of oxygen supplied, decimal fraction.

PO Box 044258 Racine, Wisconsin 53404-7005 (262) 681-0100

α = Alpha, the ratio of mass transfer coefficients, process water to clean water, decimal fraction.

Θ = Mass transfer coefficient temperature correction factor, generally taken to be 1.024, dimensionless.

T = Temperature of the process water, °C.

Y = Temperature correction factor (C^*_{bST}/C^*_{b20}) of the steady state DO saturation concentration, dimensionless.

Where:

C^*_{bST} = Tabulated DO surface saturation value at temperature T, taken from Standard Methods, mg/l.

C^*_{b20} = Tabulated DO surface saturation value at 20°C taken from Standard Methods, mg/l.

β = Ratio of steady state DO saturation concentration in process and clean water, dimensionless (basis total dissolved solids).

Ω = Pressure correction factor (P_b/P_s) for the steady state DO saturation concentration, dimensionless.

Where:

P_b = Local barometric pressure for the site, in Hg.

P_s = Standard barometric pressure, 29.92 in Hg (101.3 k Pa).

C = Dissolved oxygen concentration averaged over process water volume being evaluated, mg/l.

All of the factors involved in the conversion from clean water to process water, except alpha, can be specified or reasonably estimated from published or assumed values. The field studies were conducted at the HARRF in an effort to document the performance of the aeration system and assist in determining the capacity of the aeration portion of the plant.

RESULTS

General

The results of the full-scale offgas tests are summarized as Tables 1 to 3. The field data sheets from which the summary tables were developed are contained in Appendix I. Figure 3 is a plan view of one of the aeration basins showing the offgas hood locations employed in the evaluation. The offgas collection hood used was two feet wide by eight feet in length, and having a total capture area of 16 square feet. In identifying the hood location (station) in Tables 1 to 3, the first position is the basin number, the next position is the zone number (e.g. 1 to 7), and the last position following the period refers to the hood position within a given zone. By way of example, sample location 1.4.3 refers to Tank 1, zone 4, and location 3 within the zone.

Table 1 summarizes the results of the first day of offgas testing, conducted on April 25, 2006. The first several columns of this table, including time, sampling station designation, mixed liquor temperature, gas-phase sensor output (Mog and Mr), DO concentrations (C), and offgas flow rate are obtained from the field data sheets. Knowing the DO saturation value from clean water testing of the equipment in question (C^*_{20}), the field saturation value (C^*_f) can be estimated by applying corrections for local atmospheric pressure, mixed liquor temperature and total dissolved solids, which are reflected in the beta factor. The column headed C^*_f-DO (Column 7) represents the DO driving force (saturation minus the DO concentration) at that sampling station.

Columns 8 and 9 are the float heights (in millimeters) for the two rotameters measuring offgas flow rate. Column 10 is the collection area of the offgas collection hood. Column 11 lists the measured offgas flux for the sample location in question. The offgas flux is determined by dividing the offgas flow rate by the offgas collection hood area (Column 10).

Column 12 is the panel aerator flux and is computed by dividing the offgas flux by the panel area per square foot of floor area. The airflow to the zone (Column 13) in question is computed as the product of the offgas flux times the area of the zone in question.

The gas-phase oxygen transfer efficiency under process conditions is given by the columns headed OTE_F (Column 14), OTE_{SP20} (Column 15) and SOTEpw (Column 16). The field oxygen transfer efficiency (OTE_F) is the actual gas-phase transfer, as a decimal, under existing field conditions of DO concentration, barometric pressure, total dissolved solids, mixed liquor temperature and prevailing operating mode. OTE_{SP20} is the transfer efficiency per each mg/l of driving force, corrected to a 20°C mixed liquor temperature. SOTEpw is the oxygen transfer efficiency in process water corrected to standard conditions of one atmosphere of pressure, zero DO concentration and 20°C. Column 17, SOTEcw, is an estimate of the clean water oxygen transfer efficiency based on vendor data.

Column 18 is the ratio of SOTEpw to SOTEcw. When diffusers are not new, this ratio is alpha(F). This is the product of alpha and the fouling factor, F. F accounts for a loss of efficiency due to diffuser fouling and changes in the diffuser media. Column 19 is the computed oxygen uptake rate (OUR) for each hood location based on a gas-phase mass balance. The mass balance calculation procedure is presented in Appendix II.

The tables are divided into seven data blocks, one for each zone along the length of the tank. Listed at the bottom of the tables are the overall average values of DO concentration, offgas flux, diffuser air flow, alpha(F), and oxygen uptake rate along with the total air flow and the mean weighted average OTE_F and SOTEpw values for the entire basin.

Full Scale Results

The aeration basins tested in this evaluation were Basins 1, 3, and 4. Table 1 is a summary of the offgas data for the first day of testing (Tuesday, April 25). Offgas testing was conducted on Basin 1 from about 10 am to 6 pm.

A review of the first line of data in Table 1 for sampling station 1.1.1 (1012 hours) shows that the mixed liquor temperature was 23.3°C, the dissolved oxygen (DO) concentration was 3.15 mg/l, and that the offgas flux was observed to be 0.683 scfm per square foot. This translates to a diffuser flux of 1.49 scfm per square foot of media, and an estimated flow to the zone as a whole of 784 scfm. The field oxygen transfer efficiency (OTEf) for this location is 5.59% and when adjusted to standard conditions (20°C, zero DO, and one atmosphere) is 8.35%. The estimated clean water standard oxygen transfer efficiency (SOTEcw) at this operating condition is 35.0% based on the vendor data. The ratio of SOTEpw to SOTEcw (alpha(F)) is 0.24. The gas-phase mass balance for this location yields a calculated oxygen uptake rate (OUR) of 38.4 mg/l/hr.

Table 1 is broken up into seven sections. As shown in Figure 3, the sampling plan was broken down into seven zones along the length of the basin. Below the data for each zone is the average performance for the zone in question. At the bottom of the table is the overall performance for the basin as a whole. On April 25, 2006 the overall average performance for Basin 1 indicates a mean DO concentration of 3.14 mg/l, an average diffuser flux of 1.35 scfm per square foot, and a field oxygen transfer efficiency of 8.63%. The overall alpha(F) for Basin 1 is 0.39 and the average OUR is 39.2 mg/l/hr.

Tables 2 and 3 contain the offgas data for Basins 3 and 4 on Wednesday April 26th and Thursday April 27th, respectively. The same offgas sampling plan was employed in all cases. A review of the overall offgas results for Basins 3 and 4 show that the three basins tested all have similar performance characteristics. The results for

Basin 3 indicate an average DO concentration of 3.10 mg/l, an average diffuser flux of 1.82 scfm per square foot, and a field transfer efficiency of 6.82%. The overall alpha(F) for Basin 3 is 0.33 and the average OUR is 40.9 mg/l/hr. Basin 4 was observed to have an overall average DO concentration of 2.66 mg/l, a diffuser flux of 1.51 scfm per square foot and an overall field oxygen transfer efficiency of 8.49%. The overall alpha(F) for Basin 4 is 0.37 and the average OUR is 42.1 mg/l/hr.

SUMMARY

Three days of offgas testing were conducted at the Hale Avenue Resource Recovery Facility, April 25 to 27, 2006. The main objective was to observe aeration performance of the Parkson Panel Aeration System in an effort to assess the capacity of the system. The results indicate that the HARRF aeration system is operating at or near is capacity. The overall diffuser flux or the three basins tested is 1.56 scfm per square foot of diffuser media. The overall average field transfer efficiency was observed to be 8.50% at an overall average DO concentration of 2.97 mg/l. The overall standard oxygen transfer efficiency under process water conditions (SOTEpw) is 11.65%. On this basis the overall alpha(F) for the system is 0.36. The aeration system was operating with a mixed liquor suspended solids concentration of about 1400 mg/l and a solids retention time of 3 to 5 days at the time the system was evaluated. A review of the data shows that alpha(F) immediately downstream from the anoxic selector is approximately 0.25, and increases gradually to about 0.40 by the end of the basin. If the system could be operated at an average DO of 2.0 mg/l instead of nearly 3.0 mg/l, the overall efficiency of the system would increase by about 15%. Said another way, if the load to the system increased by about 15% the system DO would drop to an average DO of 2.0 mg/l from the current level of approximately 3.0 mg/l.

There was no effort to drain any of the basins to examine the condition of the membranes following approximately six years of continuous service. As a result, we do not know if the membranes have been significantly fouled nor do we know the condition of the panel media (whether they have stretched or deformed significantly) and what impact these changes, if any, have on the performance of the system. At some convenient point in time it is suggested that one of the aeration basins be drained so inspection of the panel aerators can be made.

FIGURE 1 ESCONDIDO, CA - HARRF
PARKSON PANEL GENERAL LAYOUT - APRIL 2006

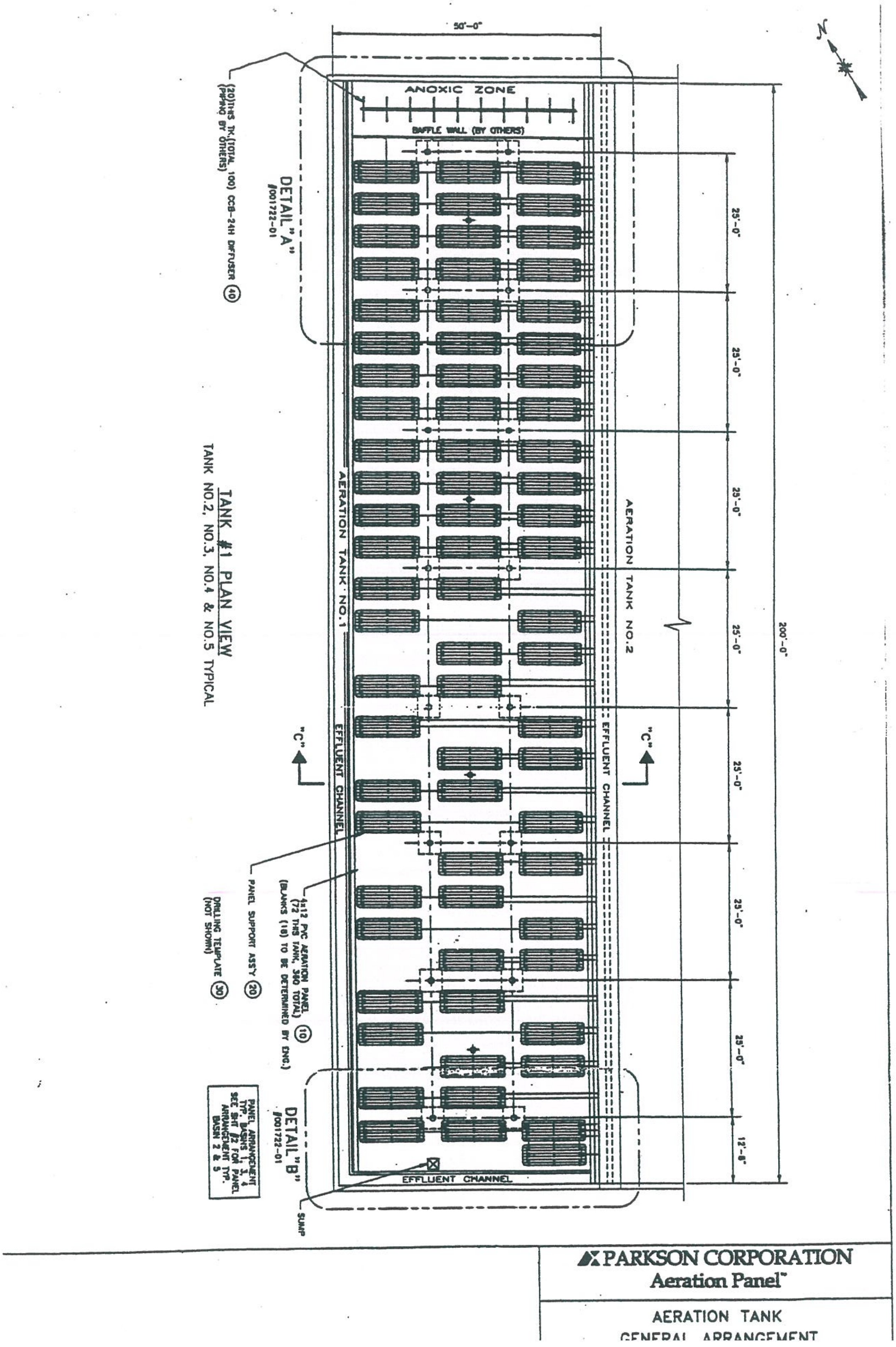
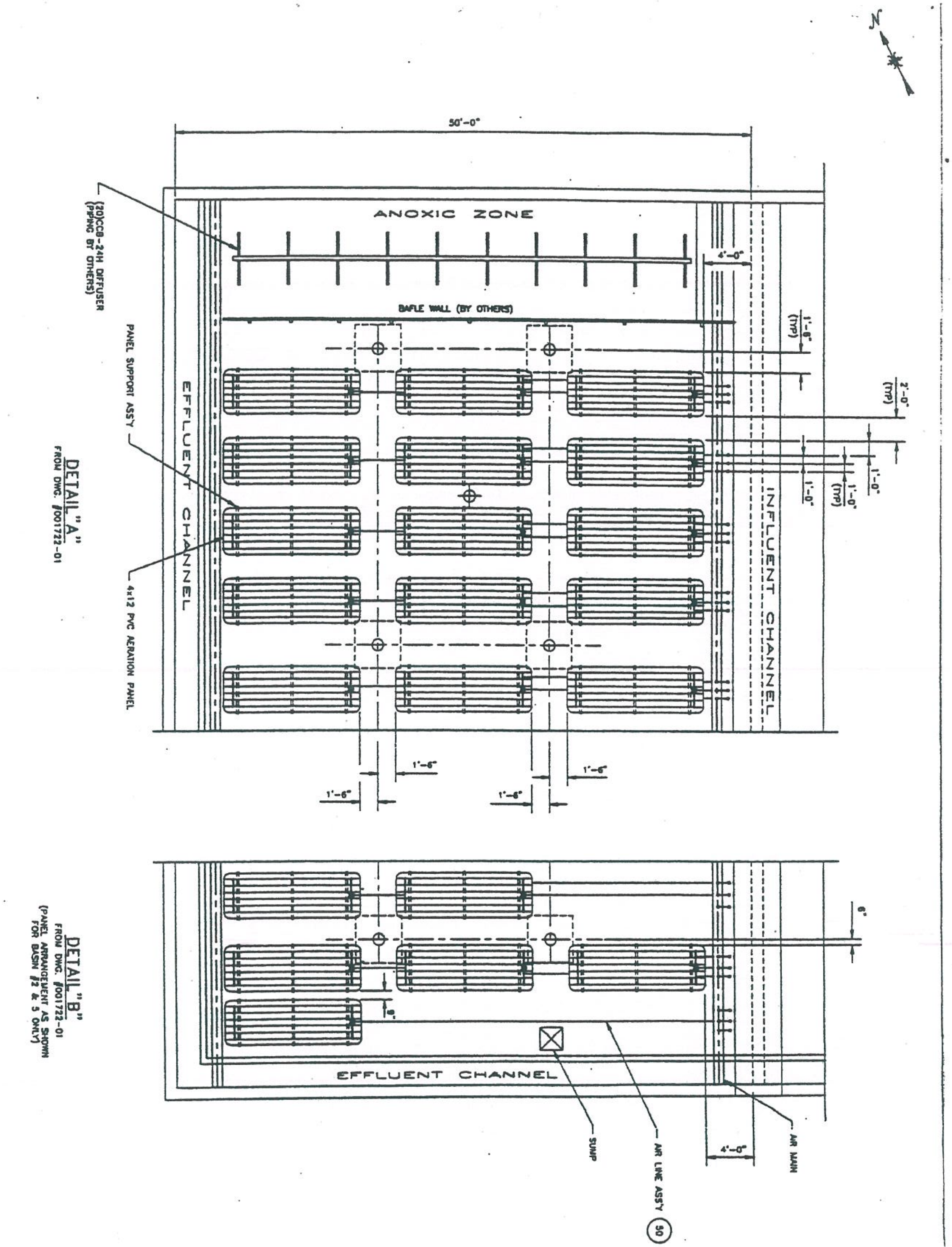


FIGURE 2 ESCONDIDO, CA - HARRF
PARKSON PANEL DETAILED LAYOUT - ARPIL 2006



PARKSON CORPORATION
Aeration Panel™

AERATION TANK
G.A. DETAIL "A" & "B"

FIGURE 3 ESCONDIDO, CA - HARRRF
OFFGAS SAMPLING PLAN - APRIL 2006

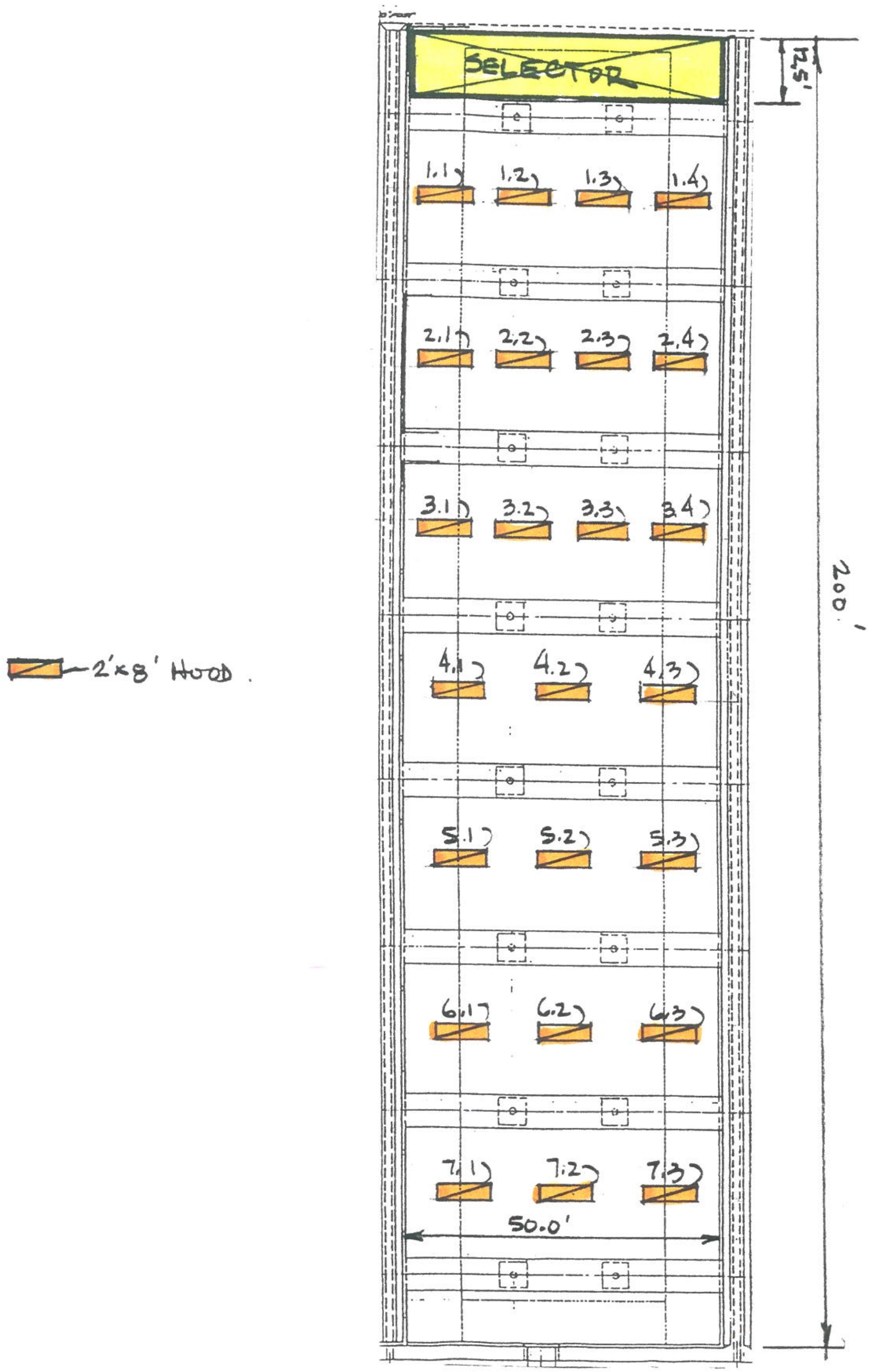


TABLE 1
SUMMARY DATA SHEET - FULL SCALE OFFGAS TESTS

DATE: April 25, 2006

SITE: ESCONDIDO HARRF
SYSTEM: PARKSON PANELS
SUBMERGENCE: 16.25 FT.
SWD: 16.50 FT.
DIFFUSERS/BASIN: 68.00

MLSS: 1,450 MG/L
RAS MG/L
TDS: 1,000 MG/L (ASSUMED)
SRT: 3-5 DAYS

LOCAL BAROMETER: 29.42 in. Hg.
BETA: 0.97
C^*_{20}: 10.75 MG/L
C^*_F: 9.60 MG/L

H_R: 0.00
H_{OG}: 0.00 LB H_2O PER LB B.D. AIR
CO_2: 0.00

AERATION BASIN 1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
TIME	STATION	ML TEMP °C	M_{OG} (mv)	M_R (mv)	DO (mg/l)	C^*_F-DO (mg/l)	Rmm 1	Rmm 2	HOOD AREA (sq ft)	OFFGAS FLUX (scfm/sq ft)	DIFFUSER FLUX (scfm/sq ft)	TOTAL AIR FLOW (cfm)	OTE_F (decimal)	OTE_{SP20} (decimal)	SOTEpw	SOTEcw	ALPHA(F)	OUR (mg/l/hr)
1012	1.1.1	23.3	959	1004	3.15	6.45	0	103	16	0.683	1.49	234.8	0.0559	0.00801	0.0835	0.350	0.24	38.4
1028	1.1.2	23.3	938	1005	2.85	6.75	0	74	16	0.524	1.49	180.1	0.0830	0.01137	0.1186	0.350	0.34	43.8
1116	1.1.3	23.3	951	1001	2.55	7.05	0	86	16	0.589	1.49	202.5	0.0626	0.00822	0.0857	0.350	0.24	37.1
1128	1.1.4	23.3	948	1001	2.70	6.90	0	67	16	0.485	1.49	166.7	0.0660	0.00885	0.0923	0.350	0.26	32.2
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.81					0.570	1.49	784.1	0.0660		0.0940	0.350	0.27	37.89
1226	1.2.1	23.3	950	1005	3.40	6.20	0	105	16	0.694	1.55	216.9	0.0683	0.01019	0.1062	0.350	0.30	47.7
1217	1.2.2	23.3	945	1005	2.65	6.95	0	103	16	0.683	1.55	213.4	0.0743	0.00989	0.1031	0.350	0.29	51.1
1206	1.2.3	23.3	916	1004	2.45	7.15	0	85	16	0.584	1.55	182.5	0.1087	0.01406	0.1466	0.350	0.42	63.9
1156	1.2.4	23.4	930	1005	3.45	6.15	0	98	16	0.655	1.55	204.7	0.0925	0.01387	0.1446	0.350	0.41	61.0
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.99					0.654	1.55	817.5	0.0849		0.1240	0.350	0.35	55.92
1251	1.3.1	23.4	940	1007	3.85	5.75	0	103	16	0.683	1.31	213.4	0.0823	0.01320	0.1376	0.370	0.37	56.6
1301	1.3.2	23.4	934	1008	3.70	5.90	0	73	16	0.518	1.31	161.9	0.0913	0.01428	0.1489	0.370	0.40	47.6
1311	1.3.3	23.4	936	1008	3.35	6.25	0	66	16	0.480	1.31	150.0	0.0887	0.01309	0.1365	0.370	0.37	42.9
1321	1.3.4	23.4	927	1008	3.15	6.45	0	77	16	0.540	1.31	168.8	0.0993	0.01420	0.1480	0.370	0.40	54.0
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.51					0.555	1.32	694.1	0.0899		0.1425	0.370	0.39	50.24
1440	1.4.1	23.4	929	1007	3.50	6.10	200	0	16	0.316	1.27	131.9	0.0959	0.01450	0.1512	0.320	0.47	30.5
1429	1.4.2	23.4	934	1008	3.65	5.95	250	0	16	0.391	1.27	163.2	0.0913	0.01416	0.1476	0.320	0.46	35.9
1416	1.4.3	23.4	932	1010	3.60	6.00	234	0	16	0.366	1.27	152.8	0.0959	0.01474	0.1537	0.320	0.48	35.3
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.58					0.358	1.27	447.9	0.0942		0.1507	0.320	0.47	33.92
1544	1.5.1	23.4	939	1008	3.30	6.30	0	53	16	0.410	1.27	171.2	0.0849	0.01243	0.1297	0.320	0.41	35.0
1602	1.5.2	23.4	944	1009	3.35	6.25	170	0	16	0.269	1.27	112.3	0.0800	0.01181	0.1231	0.320	0.38	21.7
1613	1.5.3	23.4	937	1008	2.95	6.65	250	0	16	0.391	1.27	163.2	0.0876	0.01215	0.1267	0.320	0.40	34.5
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.20					0.357	1.27	446.7	0.0846		0.1269	0.320	0.40	30.39
1701	1.6.1	23.3	941	1006	2.95	6.65	0	73	16	0.518	1.28	216.3	0.0800	0.01112	0.1160	0.320	0.36	41.7
1644	1.6.2	23.4	913	1007	3.00	6.60	225	0	16	0.354	1.28	147.8	0.1155	0.01614	0.1683	0.320	0.53	41.1
1633	1.6.3	23.4	925	1007	3.05	6.55	130	0	16	0.211	1.28	88.1	0.1011	0.01424	0.1485	0.320	0.46	21.5
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.00					0.361	1.29	452.2	0.0957		0.1394	0.320	0.44	34.78
1725	1.7.1	23.4	933	1007	2.85	6.75	240	0	16	0.375	1.25	156.6	0.0913	0.01248	0.1301	0.320	0.41	34.5
1734	1.7.2	23.4	930	1006	3.05	6.55	185	0	16	0.292	1.25	121.9	0.0936	0.01318	0.1374	0.320	0.43	27.5
1753	1.7.3	23.3	937	1004	2.75	6.85	250	0	16	0.391	1.25	163.2	0.0830	0.01121	0.1169	0.320	0.37	32.7
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.88					0.353	1.26	441.7	0.0889		0.1272	0.320	0.40	31.55
			OVERALL		3.14					0.458	1.35	4084.2	0.0863		0.1293	0.336	0.39	39.24

TABLE 2
SUMMARY DATA SHEET - FULL SCALE OFFGAS TESTS

DATE: April 26, 2006

SITE: ESCONDIDO HARRF
SYSTEM: PARKSON PANELS
SUBMERGENCE: 16.25 FT.
SWD: 16.50 FT.
DIFFUSERS/BASIN: 68.00

MLSS: 1,450 MG/L
RAS MG/L
TDS: 1,000 MG/L (ASSUMED)
SRT: 3-5 DAYS

LOCAL BAROMETER: 29.35 in. Hg.
BETA: 0.97
C^*_{20}: 10.75 MG/L
C^*_F: 9.60 MG/L

H_R: 0.00
H_{OG}: 0.00 LB H_2O PER LB B.D. AIR
CO_2: 0.00

AERATION BASIN 3

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
TIME	STATION	ML TEMP °C	M_{OG} (mv)	M_R (mv)	DO (mg/l)	C^*_F-DO (mg/l)	Rmm 1	Rmm 2	HOOD AREA (sq ft)	OFFGAS FLUX (scfm/sq ft)	DIFFUSER FLUX (scfm/sq ft)	TOTAL AIR FLOW (cfm)	OTE_F (decimal)	OTE_{SP20} (decimal)	SOTEpw	SOTEcw	ALPHA(F)	OUR (mg/l/hr)
0937	3.1.1	23.3	968	1007	3.15	6.45	0	116	16	0.756	1.60	259.9	0.0483	0.00692	0.0722	0.340	0.21	36.8
0947	3.1.2	23.2	948	1011	2.85	6.75	0	78	16	0.546	1.60	187.7	0.0777	0.01068	0.1113	0.340	0.33	42.7
0958	3.1.3	23.2	962	1009	2.50	7.10	0	93	16	0.628	1.60	215.9	0.0585	0.00764	0.0796	0.340	0.23	37.0
1009	3.1.4	23.3	957	1008	2.90	6.70	0	74	16	0.524	1.60	180.1	0.0630	0.00870	0.0907	0.340	0.27	33.2
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.85					0.614	1.60	843.6	0.0606		0.0868	0.340	0.26	37.43
1223	3.2.1	23.3	967	1008	2.75	6.85	0	133	16	0.855	1.50	267.2	0.0506	0.00683	0.0712	0.350	0.20	43.5
1213	3.2.2	23.3	955	1007	3.05	6.55	0	103	16	0.683	1.50	213.4	0.0642	0.00906	0.0944	0.350	0.27	44.1
1202	3.2.3	23.3	929	1009	2.60	7.00	0	59	16	0.442	1.50	138.1	0.0981	0.01296	0.1352	0.350	0.39	43.7
1153	3.2.4	23.4	946	1009	2.70	6.90	0	80	16	0.556	1.50	173.8	0.0777	0.01039	0.1084	0.350	0.31	43.5
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.78					0.634	1.50	792.5	0.0685		0.0968	0.350	0.28	43.70
1248	3.3.1	23.4	957	1002	3.85	5.75	0	135	16	0.867	1.57	270.9	0.0559	0.00896	0.0934	0.350	0.27	48.7
1257	3.3.2	23.4	944	1004	4.05	5.55	0	105	16	0.694	1.57	216.9	0.0743	0.01236	0.1289	0.350	0.37	51.9
1308	3.3.3	23.4	942	1001	3.60	6.00	0	78	16	0.546	1.57	170.6	0.0732	0.01126	0.1174	0.350	0.34	40.2
1318	3.3.4	23.4	945	1003	3.40	6.20	0	78	16	0.546	1.57	170.6	0.0717	0.01067	0.1113	0.350	0.32	39.4
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.73					0.663	1.57	829	0.0675		0.1113	0.350	0.32	45.08
1451	3.4.1	23.5	935	1002	3.65	5.95	0	93	16	0.628	1.90	262.2	0.0830	0.01284	0.1339	0.350	0.38	52.5
1441	3.4.2	23.5	937	1003	3.65	5.95	0	69	16	0.496	1.90	207.1	0.0819	0.01267	0.1321	0.350	0.38	40.9
1430	3.4.3	23.5	928	1001	3.45	6.15	0	65	16	0.474	1.90	197.9	0.0906	0.01355	0.1413	0.350	0.40	43.2
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.58					0.533	1.90	667.2	0.0849		0.1355	0.350	0.39	45.53
1515	3.5.1	23.5	946	1003	3.35	6.25	0	93	16	0.628	2.21	262.2	0.0706	0.01039	0.1084	0.260	0.42	44.6
1526	3.5.2	23.5	960	1004	3.35	6.25	0	96	16	0.644	2.21	268.9	0.0547	0.00806	0.0840	0.260	0.32	35.5
1537	3.5.3	23.5	956	1004	3.00	6.60	0	86	16	0.589	2.21	245.9	0.0596	0.00831	0.0867	0.260	0.33	35.4
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					3.23					0.620	2.21	777	0.0616		0.0931	0.260	0.36	38.48
1622	3.6.1	23.5	961	1006	2.90	6.70	0	94	16	0.633	1.92	264.3	0.0559	0.00767	0.0800	0.270	0.30	35.6
1610	3.6.2	23.5	943	1003	2.80	6.80	0	68	16	0.491	1.92	205.0	0.0743	0.01006	0.1049	0.270	0.39	36.7
1600	3.6.3	23.5	948	1004	2.80	6.80	0	68	16	0.491	1.92	205.0	0.0694	0.00940	0.0980	0.270	0.36	34.3
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.83					0.538	1.92	674.3	0.0656		0.0930	0.270	0.34	35.55
1644	3.7.1	23.5	951	1007	2.85	6.75	0	83	16	0.573	2.07	239.2	0.0694	0.00947	0.0987	0.260	0.38	40.0
1654	3.7.2	23.5	950	1007	2.65	6.95	0	74	16	0.524	2.07	218.8	0.0706	0.00935	0.0975	0.260	0.37	37.2
1706	3.7.3	23.5	950	1004	2.55	7.05	0	96	16	0.644	2.07	268.9	0.0672	0.00877	0.0914	0.260	0.35	43.5
					$\bar{X}$ =					$\bar{X}$ =	$\bar{X}$ =	Σ =	MWA =		MWA =	$\bar{X}$ =		$\bar{X}$ =
					2.68					0.580	2.07	726.9	0.0689		0.0956	0.260	0.37	40.27
			OVERALL		3.10					0.597	1.82	5310.5	0.0682		0.1017	0.311	0.33	40.86

TABLE 3
SUMMARY DATA SHEET - FULL SCALE OFFGAS TESTS

DATE: April 27, 2006

SITE: ESCONDIDO HARRF		MLSS: 1,430 MG/L		LOCAL BAROMETER: 29.37 in. Hg.		H_R: 0.00	
SYSTEM: PARKSON PANELS		RAS MG/L		BETA: 0.97		H_{OG}: 0.00 LB H_20	
SUBMERGENCE: 16.25 FT.		TDS: 1,000 MG/L (ASSUMED)		C^*_{20}: 10.75 MG/L		PER LB B.D. AIR	
SWD: 16.50 FT.		SRT: 3-5 DAYS		C^*_F: 9.65 MG/L		CO_2: 0.00	
DIFFUSERS/BASIN: 68.00							

AERATION BASIN 4

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
TIME	STATION	ML TEMP °C	M_{OG} (mv)	M_R (mv)	DO (mg/l)	C^*_F-DO (mg/l)	Rmm 1	Rmm 2	HOOD AREA (sq ft)	OFFGAS FLUX (scfm/sq ft)	DIFFUSER FLUX (scfm/sq ft)	TOTAL AIR FLOW (cfm)	OTE_F (decimal)	OTE_{SP20} (decimal)	SOTEpw	SOTEcw	ALPHA(F)	OUR (mg/l/hr)
0901	4.1.1.	23.1	950	1005	1.65	8.00	0	54	16	0.415	1.28	142.7	0.0683	0.00793	0.0827	0.370	0.22	28.5
0910	4.1.2	23.1	946	1004	1.65	8.00	0	70	16	0.502	1.28	172.6	0.0717	0.00833	0.0868	0.370	0.23	36.2
0921	4.1.3	23.1	945	1004	2.60	7.05	0	68	16	0.491	1.28	168.8	0.0732	0.00965	0.1006	0.370	0.27	36.2
0930	4.1.4	23.1	957	1003	2.40	7.25	0	80	16	0.556	1.28	191.1	0.0574	0.00735	0.0767	0.370	0.21	32.1
					$\bar{X}$ = 2.08					$\bar{X}$ = 0.491	$\bar{X}$ = 1.28	Σ = 675.2	MWA = 0.0673		MWA = 0.0865	$\bar{X}$ = 0.370	0.23	$\bar{X}$ = 33.26
1020	4.2.1	23.1	927	1003	1.80	7.85	0	50	16	0.394	1.26	123.1	0.0940	0.01112	0.1160	0.370	0.31	37.3
1010	4.2.2	23.1	930	1003	2.40	7.25	0	89	16	0.606	1.26	189.4	0.0898	0.01151	0.1200	0.370	0.32	54.8
0958	4.2.3	23.1	923	1002	2.55	7.10	0	78	16	0.546	1.26	170.6	0.0974	0.01274	0.1329	0.370	0.36	53.5
0948	4.2.4	23.1	946	1003	2.85	6.80	0	85	16	0.584	1.26	182.5	0.0706	0.00964	0.1006	0.370	0.27	41.5
					$\bar{X}$ = 2.40					$\bar{X}$ = 0.533	$\bar{X}$ = 1.26	Σ = 665.6	MWA = 0.0872		MWA = 0.1172	$\bar{X}$ = 0.370	0.32	$\bar{X}$ = 46.76
1042	4.3.1	23.1	929	1006	2.90	6.75	0	61	16	0.453	1.27	141.6	0.0947	0.01304	0.1360	0.370	0.37	43.2
1052	4.3.2	23.1	918	1008	3.10	6.55	0	73	16	0.518	1.27	161.9	0.1102	0.01563	0.1630	0.370	0.44	57.5
1102	4.3.3	23.1	944	1011	3.10	6.55	0	85	16	0.584	1.27	182.5	0.0823	0.01167	0.1217	0.370	0.33	48.4
1111	4.3.4	23.2	952	1007	2.85	6.80	0	85	16	0.584	1.27	182.5	0.0679	0.00926	0.0965	0.370	0.26	39.9
					$\bar{X}$ = 2.99					$\bar{X}$ = 0.535	$\bar{X}$ = 1.27	Σ = 668.5	MWA = 0.0877		MWA = 0.1278	$\bar{X}$ = 0.370	0.35	$\bar{X}$ = 47.24
1148	4.4.1	23.2	918	1007	3.00	6.65	184	0	16	0.291	1.48	121.5	0.1091	0.01520	0.1585	0.310	0.51	31.9
1138	4.4.2	23.2	924	1008	3.10	6.55	0	58	16	0.436	1.48	182.0	0.1034	0.01463	0.1526	0.310	0.49	45.4
1127	4.4.3	23.2	948	1009	3.00	6.65	0	74	16	0.524	1.48	218.8	0.0755	0.01052	0.1097	0.310	0.35	39.8
					$\bar{X}$ = 3.03					$\bar{X}$ = 0.417	$\bar{X}$ = 1.48	Σ = 522.3	MWA = 0.0930		MWA = 0.1360	$\bar{X}$ = 0.310	0.44	$\bar{X}$ = 39.05
1215	4.5.1	23.2	943	1006	2.85	6.80	0	57	16	0.431	1.79	179.9	0.0777	0.01060	0.1105	0.290	0.38	33.7
1228	4.5.2	23.5	947	1005	2.85	6.80	0	74	16	0.524	1.79	218.8	0.0717	0.00970	0.1012	0.290	0.35	37.8
1239	4.5.3	23.2	941	1006	2.90	6.75	0	79	16	0.551	1.79	230.0	0.0800	0.01099	0.1146	0.290	0.40	44.4
					$\bar{X}$ = 2.87					$\bar{X}$ = 0.502	$\bar{X}$ = 1.79	Σ = 628.7	MWA = 0.0765		MWA = 0.1087	$\bar{X}$ = 0.290	0.37	$\bar{X}$ = 38.64
1339	4.6.1	23.3	945	1007	2.75	6.90	0	71	16	0.507	1.77	211.7	0.0766	0.01027	0.1071	0.290	0.37	39.1
1328	4.6.2	23.3	918	1006	2.80	6.85	0	68	16	0.491	1.77	205.0	0.1079	0.01457	0.1519	0.290	0.52	53.3
1317	4.6.3	23.3	929	1010	3.05	6.60	0	68	16	0.491	1.77	205.0	0.0993	0.01391	0.1450	0.290	0.50	49.1
					$\bar{X}$ = 2.87					$\bar{X}$ = 0.496	$\bar{X}$ = 1.77	Σ = 621.7	MWA = 0.0944		MWA = 0.1344	$\bar{X}$ = 0.290	0.46	$\bar{X}$ = 47.16
1357	4.7.1	23.3	936	1010	2.20	7.45	0	50	16	0.394	1.72	164.5	0.0906	0.01124	0.1172	0.290	0.40	35.9
1410	4.7.2	23.3	928	1010	2.35	7.30	0	61	16	0.453	1.72	189.1	0.1004	0.01272	0.1326	0.290	0.46	45.8
1420	4.7.3	23.3	943	1006	2.70	6.95	0	88	16	0.600	1.72	250.5	0.0777	0.01034	0.1079	0.290	0.37	47.0
					$\bar{X}$ = 2.42					$\bar{X}$ = 0.482	$\bar{X}$ = 1.72	Σ = 604.1	MWA = 0.0883		MWA = 0.1182	$\bar{X}$ = 0.290	0.41	$\bar{X}$ = 42.88
			OVERALL		2.66					0.494	1.51	4386.1	0.0849		0.1184	0.327	0.37	42.14

SYMBOLS AND NOMENCLATURE

DO	=	Dissolved Oxygen
C	=	DO concentration, mg/l
C^*_F	=	DO saturation value applicable for equipment in use and existing conditions, mg/l
C^*_F-C	=	DO driving force or effective DO deficit, mg/l
$C^*_{\infty 20}$	=	DO saturation value in clean water for system tested at standard conditions as time approaches infinity
C^*_{ST}	=	Tabulated DO surface saturation value at temperature T, taken from Standard Methods, mg/l
C^*_{20}	=	Tabulated DO surface saturation value at 20 C taken from standard Methods, mg/l
EPDM	=	E-Ethylene, P-propylene, D-Diene co-moners, M-polyMethylene backbone; synthetic rubber
FOTR	=	Field Oxygen transfer Rate in process water at existing conditions
fpm	=	Feet per minute
gpm	=	Gallons per minute
Hg	=	Mercury
Hood Area	=	Offgas Hood Collection Area, square feet

PO Box 044258 Racine, Wisconsin 53404-7005 (262) 681-0100

Symbols and Nomenclature
Page 2

K_La	=	Apparent volumetric mass transfer coefficient of oxygen in clean water and/or process water
MLSS	=	Mixed Liquor Suspended Solids, mg/l
MLT	=	Mixed Liquor Temperature, °C
M(og)	=	Gas phase oxygen sensor output in millivolts for offgas stream
M(r)	=	Gas phase oxygen sensor output in millivolts for reference stream
MWA	=	Mean weighted average
Offgas Flux Rate	=	Rate of offgas evolution per square foot of collection area as measured by offgas rotameters, scfm/sq ft
OTE_F	=	Process water oxygen transfer efficiency, mass fraction of oxygen transferred per unit of oxygen supplied, decimal fraction
OTE_{SP20}	=	Oxygen Transfer efficiency per each mg/l of driving force under Standard Conditions
OUR	=	Oxygen Uptake Rate by mixed liquor, mg/l/hr
P_b	=	Local barometric pressure for the site, in Hg
P_s	=	Standard barometric pressure, 29.92 in Hg
Rmm 1 & Rmm 2	=	Float Height in millimeters, from scale, for rotameters 1 and 2 in offgas analyzer

scfm	=	Air flow rate, Standard cubic feet per minute
SOTE	=	Standard Oxygen Transfer efficiency at 20°C and zero DO
SOTEcw	=	Standard Oxygen Transfer efficiency at Standard Conditions and zero DO in clean water
SOTEpw	=	Standard Oxygen Transfer efficiency at Standard Conditions and zero DO in process water
SOTR	=	Standard Oxygen Transfer Rate in clean water at 20°C and zero DO
SRT	=	Solids Retention Time or Sludge Age, days
Standard Conditions	=	Barometric Pressure of 29.92 in Hg and 20°C
Submergence	=	Height of liquid above diffusers, feet
T	=	Temperature, °C
TDS	=	Total Dissolved Solids in mixed liquor, mg/l
wg	=	Water gauge
α	=	Alpha, the ratio of mass transfer coefficients (KLa), or standard oxygen transfer efficiency, process water to clean water, decimal fraction
β	=	Beta, the ratio of steady state DO saturation concentration in process and clean water, dimensionless (basis total dissolved solids)

Ω	=	Pressure correction factor (P_b/P_s) for the steady state DO saturation concentration, dimensionless
Θ	=	Mass transfer coefficient temperature correction factor, generally taken to be 1.024, dimensionless
Y	=	Temperature correction factor (C^*_{ST}/C^*_{20}) for the steady state DO saturation concentration, dimensionless

APPENDIX I

FIELD DATA SHEETS

PAGE: 1
DATE: APRIL 25, 2006
SITE: ESCONDIDO
UNIT: TANK 1

MLT: 23.3°C
BAROM: 30.03-0.65 IN. HG.
TDS:

PLT ELEV. 610 FT AMSL HOOD: 2FT x 8FT.

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1006	DAA	2-90	70		-8.0	1003			
1008	1.1.1	2-95	+.2			953		2.49	
1009	"	2-106				956		3.27	23.3°C
1010	"	2-103				957		3.54	
1011	1.1.1	2-103				959		3.35	
1012	"	2-103				959-60		3.06	
1013	DAA				-8.0	1004			
1021	DAA				-8.0	1005			
1023	1.1.2	2-61				924		2.93	
1024	"	2-61				926		2.60	23.3°C
1025	"	2-65				936		2.69	
1026	1.1.2	2-65				931		2.95	
1027	"	2-71				934		2.77	
1028	"	2-74				938		2.91	
1029	DAA				-8.0	1005		3.21	
1111	DAA				-8.0	1002			
1112	1.1.3	2-82				951		2.44	
1113	"	2-81				950		2.78	23.3°C
1114	"	2-89				954		2.41	
1115	1.1.3	2-85				951		2.61	
1116	"	2-86				950		2.49	
1117	DAA				-8.0	1000-1			
1124	DAA				-8.0	1002			
1125	1.1.4	2-76				950		2.70	
1126	"	2-69				949		2.55	23.3°C
1127	"	2-58				948		2.76	
1128	1.1.4	2-60				948		2.71	
1129	"	2-67				946-7		2.78	
1130	DAA				-8.0	1001			

PAGE: 2
DATE: APRIL 25, 2006
SITE: ESCONDIDO -
UNIT: TANK 1

MLT: 23.4°C
BAROM: 30.03 - 0.60
TDS:

HOOD: 2FT x 8FT -

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1150.5	DAA				1004	-8.0		3.07	
1152	1.2.4	2-94				936		2.90	
1153	"	2-93				933		2.79	23.4°C
1154	"	2-93				924		2.80	
1155	1.2.4	2-95				923		2.77	
1156	"	2-98				934		2.95	
1157	DAA				-8.0	1005			
1200.4	DAA				-8.0	1004		2.27	
1202	1.2.3	2-86				9.15		2.57	
1203	"	2-85	71F.			906		2.54	23.3°C
1204	"	2-85				919		2.64	
1205	1.2.3	2-85				924		2.22	
1206	"	2-85				917		2.56	
1207	DAA				-8.0	1004			
1212	DAA					1003-4		2.42	
1213	1.2.2	2-99				948		2.69	23.3°C
1214	"	2-103				947		2.56	
1215	"	2-103				945		2.52	
1216	1.2.2	2-103	71F.			942		2.84	
1217	"					944		3.03	
1218	DAA				-8.0	1005			
1221	DAA					1003-4		3.63	
1222	1.2.1	2-101				951		3.53	23.4°C
1223	"	2-101				952		3.61	
1224	"	2-105	71F.			948		3.36	
1225	1.2.1	2-105				950		3.33	
1226	"	2-105				950		3.11	
1227	DAA				-8.0	1005		3.35	

PAGE: 3
DATE: APRIL 25, 2006
SITE: ESCONDIDO
UNIT: TAWRI

MLT: 23.4°C
BAROM: 30.03 -0.60
TDS:

HOOD: 2FT x 8FT.

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1246	DAA				-8.0	1004-5			
1247	1.3.1.	2-88				940		3.89	
1248	"	2-84				931		3.98	
1249	"	2-84				938		3.82	23.4°C
1250	1.3.1	2-89	71F.			943		3.82	
1251	"	2-103				947		3.86	
1252	DAA				-8.0	1006-7			
1256	DAA				-8.0	1006-7		3.72	
1257	1.3.2	2-84				934		3.69	23.4°C
1258	"	2-68				933		3.74	
1259	"	2-63	68F.			935		3.78	
1300	1.3.2	2-70				934		3.64	
1301	"	2-73 ✓				930		3.61	
1302	DAA				-8.0	1008			
1306	DAA				-8.0	1007-8			
1307	1.3.3	2-68				933		3.42	
1308	"	2-67	67F			932		3.45	23.4°C
1309	"	2-67				938		3.51	
1310	1.3.3	2-66				938		3.49	
1311	"	2-66 ✓				937		3.27	
1312	DAA				-8.0	1008-9			
1316	DAA				-8.0	1009			
1317	1.3.4	2-59				921		3.02	
1318	"	2-59	67F			921		3.03	23.4°C
1319	"	2-64				923		3.04	
1320	1.3.4	2-68				929		3.24	
1321		2-77				941		3.49	
1322	DAA				-8.0	1008			

PAGE: 4
DATE: APRIL 25, 2006
SITE: ESCONDIDO
UNIT: TANK 4

MLT: 23.4°C
BAROM: 30.03−0.60
TDS:

HOOD: 2FT x 8FT.

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1411	DAA				-8.0	1009			
1412	1.4.3	2-48				926		3.75	23.4°C
1413	"	2-48				934		3.75	
1414	"	2-48	64F.			938		3.60	
1415	1.4.3	2-45				933		3.48	8-PANELS.
1416	"	2-45				930		3.41	
1417	DAA				-8.0	1010			
1424	DAA				-8.0	1007-8			
1426	1.4.2	1-219				930.		3.69	
1427	"	1-240				931		3.66	23.4°C
1428	"	1-250				936		3.59	
1429	1.4.2	1-250	65F			934		3.63	
1430	DAA				-8.0	1008			
1435	DAA				-8.0	1006		3.83	
1437	1.4.1	1-195				929		3.54	23.4°C
1438	"	1-195	65F			927		3.30	
1439	"	1-200				930		3.36	
1440	1.4.1	1-200				930		3.42	
1441	DAA				-8.0	1006-7			
1539	DAA				-8.0	1008			
1541	1.5.1	2-60				936		3.15	
1542	"	2-60				940		3.21	23.4°C
1543	"	2-53				939		3.38	
1544	1.5.1	2-53				940		3.46	
1545	DAA					1008			

PAGE: 5
DATE: APRIL 25, 2006
SITE: ESCONDIDO
UNIT: TANK 1

MLT: 23.4°C
BAROM: 30.03-0.60
TDS:

HOOD: 2ft x 8ft

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1557	DAA				-8.0	1008			
1558	1.5.2	2-50				948		3.21	
1559	"	1-197				944		3.44	23.4°C
1600	"	1-175				945		3.55	
1601	1.5.2	1-167				942		3.41	
1602	"	1-170				940		3.27	
1603	DAA				-8.0	1009			
1608	DAA				-8.0	1008		3.09	
1609	1.5.3	1-180				934		2.98	23.4°C
1610	"	1-220				935		2.88	
1611	"	1-218				937		2.94	
1612	1.5.3	1-228				937		2.87	
1613	"	1-250				939		2.84	
1614	DAA				-8.0	1008			
1628	DAA				-8.0	1006			
1629	1.6.3	1-168				920		3.05	
1630	"	1-150				922		3.17	23.4°C
1631	"	1-115	65F.			924		2.95	
1632	1.6.3	1-115				928		3.01	
1633	"	1-130				929		3.05	
1634	DAA				-8.0	1007			
1639	DAA				-8.0	1006			
1640	1.6.2	1-190				912		3.05	
1641	"	1-203				915		3.22	23.4°C
1642	"	1-211	64F.			914		3.34	
1643	1.6.2	1-225				913		3.29	
1644		1-225				913		3.04	
1645	DAA				-8.0	1007			

PAGE: 6
DATE: APRIL 25, 2006
SITE: ESCONDIDO
UNIT: TANK 1

MLT: 23.3°C
BAROM: 30.03 −0.60
TDS:

HOOD: 2FT X 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1656	DAA				−8.0	1005			
1657	1.6.1	2-62				934		3.14	
1658	"	2-65				937		2.96	23.3°C
1659	"	2-68				942		2.92	
1700	1.6.1	2-70	64F.			945		3.11	
1701		2-73✓				946		2.70	
1702	DAA				−8.0	1006			
1720	DAA				−8.0	1007			
1721	1.7.1	1-120				939		2.92	
1722	"	1-~~[illegible]~~-130				935		2.82	23.4°C
1723	"	1-167				933		2.83	
1724	1.7.1	1-180				930		2.79	
1725	"	1-195				930		2.86	
1726	DAA	1-240✓			−8.0	1007			
1730	DAA				−8.0	1006			
1731	1.7.2	1-250				930		2.92	
1732	"	1-238	65F			932		3.08	23.4°C
1733	"	1-200				930		3.06	
1734	1.7.2	1-185✓				930		3.06	
1735	DAA				−8.0	1006			
1736									
1747	DAA				−8.0	1004-5			
1749	1.7.3					935		2.79	
1750	"	2-75				937		2.77	23.3°C
1751	"	1-190				939		2.81	
1752	1.7.3	1-250				939		2.77	
1753	"	1-250✓				937		2.67	
1754	DAA				−8.0	1004-3			

PAGE: 1
DATE: APRIL 26, 2006
SITE: ESCONDIDO
UNIT: TANK 3

MLT: 23.3°C
BAROM: 29.98 - 0.60
TDS:

WOOD: 2FT x 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
0931.5	DAA				-8.0	1005			
0933	3.1.1	2-140				967		2.69	23.3°C
0934		2-140				966		2.66	
0935		2-116	72F.			963		3.10	12 PANELS.
0936	3.1.1	2-116				968		3.38	
0937	"	2-116✓				970		3.56	TANK 3
0938	DAA					1007		3.41	3975 scfm
0942	DAA				-8.0	1009			
0943	3.1.2	2-60				951		2.56	
0944	"	2-60				950		2.70	23.2°C
0945	"	2-66	70F			950		3.24	
0946	3.1.2	2-78				945		3.11	
0947	"	2-78✓				946		2.75	
0948	DAA				-8.0	1011			
0953	DAA				-8.0	1004-5			
0954	3.1.3	2-101				961		2.35	
0955	"	2-91	70F			962		2.51	23.2°C
0956	"	2-93				961		2.48	
0957	3.1.3	2-93				963		2.49	
0958	"	2-93✓				963		2.61	
0959	DAA				-8.0	1009		2.58	
1004	DAA				-8.0	1004			
1005	3.1.4	2-60				955		3.22	
1006	"	2-68				957		2.95	23.3°C
1007	"	2-80				960		2.88	
1008	3.1.4	2-74				956		2.77	
1009	"	2-74✓				958		2.82	
1010	DAA				-8.0	1008			

PAGE: 2
DATE: APRIL 26, 2006
SITE: ESCONDIDO
UNIT: TANK 3

MLT: 23.4°C
BAROM: 29.96-0.60
TDS:

Hood: 2ft x 8ft

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1148	DAA				-8.0	1007			
1149	3.2.4	2-98				944		2.61	
1150	"	2-99	76F.			951		2.71	23.4°C
1151	"	2-80				944		2.86	
1152	3.2.4	2-80				939		2.70	
1153	"	2-80				950		2.74	
1154	DAA.				-8.0	1009			TANK 3
									4490 scfm
1157	DAA				-8.0	1008			
1158.5	3.2.3	2-74				935		2.42	
1159	"	2-59				928		2.82	
1200	"	2-50				924		2.48	23.3°C
1201	3.2.3	2-57				929		2.61	
1202	"	2-57✓				933		2.70	
1203	DAA				-8.0	1009			
1208	DAA				-8.0	1005			
1209	3.2.2	2-95				936		3.55	
1210	"	2-90				942		3.33	23.3°C
1211	"	2-103	73F.			958		2.83	
1212	3.2.2	2-114				963		2.86	
1213	"	2-103✓				958		2.61	
1214	DAA				-8.0	1007			
1218	DAA				-8.0	1007			
1219	3.2.1	2-133				963		2.45	
1220	"	2-133				967		2.90	23.3°C
1221	"	2-133				969		3.10	
1222	3.2.1	2-133				967		2.67	
1223	"	2-133✓				968		2.64	
1224	DAA				-8.0	1008			

PAGE: 3
DATE: APRIL 26 2005
SITE: ESCONDIDO
UNIT: TANK 3

MLT: 23.4°C
BAROM: 29.96-0.60
TDS:

ADOD: 24T x 8T

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1243.5	DAA				-8.0	1004			
1244	3.3.1	2-134	73F			954		3.80	
1245	"	2-137				957		3.75	23.4°C
1246	"	2-137				958		3.96	
1247	3.3.1	2-130				957		3.83	
1248	"	2-135✓				960		3.97	
1249	DAA				-8.0	1002			12 PANELS.
1252	DAA				-8.0	1005			
1253	3.3.2	2-88				941		4.01	
1254	"	2-94				940		4.05	23.4°C
1255	"	2-100	75F			949		4.08	
1256	3.3.2	2-105				945		4.04	
1257	"	2-105✓				943		3.99	
1258	DAA				-8.0	1004		4.07	
1303	DAA				-8.0	1001-2			
1304	3.3.3	2-79				946		3.63	
1305	"	2-73				937		3.58	23.4°C
1306	"	2-73				939-40		3.67	
1307	3.3.3	2-80				943		3.53	
1308	"	2-78✓				946		3.71	
1309	DAA				-8.0	1001			
1313	DAA				-8.0	1003			
1314	3.3.4	2-100				939		3.58	
1315	"	2-68				949		3.39	23.4°C
1316	"	2-68				950		3.54	
1317	3.3.4	2-80				947		3.27	
1318	"	2-78				942		3.28	
1319	DAA				-8.0	1003			

PAGE: 4
DATE: APRIL 26, 2006
SITE: ESCONDIDO
UNIT:
TANK 3

MLT: 23.5°C
BAROM: 29.96-0.60
TDS:

HOOD: 2RTX OFF

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1425	DAA				-8.0	1001			
1426	3.4.3	2-50				919		3.51	23.5°C
1427	"	2-58				926		3.43	
1428	"	2-63	71F			926		3.45	
1429	3.4.3	2-65				930		3.48	
1430	"	2-65				931		3.47	
1431	DAA				-8.0	1001			
1436	DAA				-8.0	1003			
1437	3.4.2	2-77				938		3.41	23.5°C
1438	"	2-82	74F			934		3.62	
1439	"	2-86				941		3.62	
1440	3.4.2	2-69				939		3.78	8 PANELS
1441	"	2-69				935		3.70	
1442	DAA				-8.0	1003			
1446	DAA				-8.0	1002			
1447	3.4.1	2-92				926		3.75	
1448	"	2-95				935		3.58	23.5°C
1449	"	2-84				942		3.59	
1450	3.4.1	2-84				937		3.65	
1451	"	2-90				935		3.69	
1452	DAA	~~2-90~~			-8.0	1001-2			
1510	DAA				-8.0	1008			
1511	3.5.1	2-90				945		3.29	8 PANELS,
1512	"	2-98				948		3.29	23.5°C
1513	"	2-102	70F			950		3.39	
1514	3.5.1	2-93				942		3.42	
1515	"	2-93				945		3.43	
1516	DAA				-8.0	1003-4			

PAGE: 5
DATE: APRIL 26, 2006
SITE: ESCONDIDO
UNIT: TANK 3

MLT: 23.5°C
BAROM: 29.96-0.60
TDS:

HOOD: 2FT x 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1522	DAA				-8.0	1005			
1523	3.5.2	2-112				960		3.39	
1524	"	2-97	72F			960		3.42	23.5°C
1525	"	2-95				960		3.42	
1526	3.5.2	2-96				960		3.24	
1527	DAA				-8.0	1004			
1532.5	DAA				-8.0	1004			
1533	3.5.3	2-91				954		3.07	
1534	"	2-87				956		2.97	23.5°C
1535	"	2-86				957		3.02	
1536	3.5.3	2-86	73F			956		2.99	
1537	"	2-86✓				958		3.00	
1538	DAA				-8.0	1004			
1555	DAA					1004-5			
1556	3.6.3	2-83				951		2.92	23.5°C
1557	"	2-70				951		2.95	
1558	"	2-63				952		2.92	8 PANELS.
1559	3.6.3	2-57				941		2.68	
1600	"	2-57				947		2.55	
1601	DAA				-8.0	1004			
1605.5	DAA				-8.0	1004			
1606	3.5.2	2-83				947		2.69	
1607	"	2-78				945		2.90	23.5°C
1608	"	2-77	70F			937		2.84	
1609	3.5.2	2-62				941		2.82	
1610	"	2-68✓				944		2.67	
1611	DAA				-8.0	1003-4			

PAGE: 6
DATE: April 26 2006
SITE: ESCONDIDO
UNIT:
TANK 3

MLT: 23.5°C
BAROM: 29.92-960
TDS:

HOOD: 2A-K8FT.

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1616.5	DAA				-8.0	1005			
1618	3.8.1	2-114				961		2.95	
1619	"	2-87				962		2.93	23.5°C
1620	"	2-100	72F.			962		2.88	
1621	3.8.1	2-106				961		2.87	
1622	"	2-94				958		2.92	
1623	DAA				-8.0	1006			
1639	DAA				-8.0	1007			
1640	3.7.1	2-96				957		2.89	
1641	"	2-77				952		2.79	23.5C
1642	"	2-89	73F.			951		2.95	8 PANELS.
1643	3.7.1	2-71				951		2.78	
1644	"	2-83				946		2.79	
1645	DAA				-8.0	1007			
1649.5	DAA				-8.0	1007-8			
1651	3.7.2	2-80				950		2.68	23.5°C
1652	"	2-73				951		2.60	
1653	"	2-78				951		2.59	
1654	3.7.2	2-74				949		2.74	
1655	DAA				-8.0	1007		2.71	
1701	DAA				-8.0	1004			
1702	3.7.3	2-90				945		2.70	23.5°C
1703	"	2-93	73F.			944		2.52	
1704	"	2-79				952		2.59	
1705	3.7.3	2-85				954		2.43	
1706	"	2-96				953		2.52	
1707	DAA				-8.0	1004			

PAGE: 1
DATE: APRIL 27, 2006
SITE: ESCONDIDO
UNIT: TANK 4

MLT: 23.0°C
BAROM: 29.95-0.60
TDS:

HOOD: 2FT x 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
0855	DAA				-8.0	1005			TANK5 2820
0857	4.1.1	1-235				947		1.72	TANK4 3954
0858	"	2-80	60F.			952		1.60	TANK3 4410
0859	"	2-78				952		1.55	TANK2 4445
0900	4.1.1	2-69				952		1.55	TANK1 3300
0901	"	2-54				949		1.78	
0902	DAA				-8.0	1005			
0905	DAA				-8.0	1004			
0906	4.1.2	2-63				942		1.58	23.1°C
0907	"	2-67				945		1.75	
0908	"	2-73	60F			946		1.53	
0909	4.1.2	2-73				947		1.59	12 PANELS.
0910	"	2-70				948		1.66	
0911	DAA				-8.0	1004			
0916	DAA				-8.0	1004			
0917	4.1.3	2-74				943		2.11	
0918	"	2-57				946		2.30	23.1°C
0919	"	2-64	61F			946		2.31	
0920	4.1.3	2-70				948		2.25	
0921	"	2-68				944		2.43	
0922	DAA				-8.0	1004			
0925	DAA				-8.0	1004			
0926	4.1.4	2-96				960		3.02	23.1°C
0927	"	2-84	63F.			957		2.53	
0928	"	2-84				959		2.27	
0929	4.1.4	2-80				957		2.23	
0930	"	2-80				953		2.01	
0931	DAA				-8.0	1003			

PAGE: 2
DATE: APRIL 27 2006
SITE: BSCOWALBO
UNIT: TANK 4

MLT: 23.1°C
BAROM: 29.95 -0.60
TDS:

HOOD: 2FT x 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
0943	DAA				-8.0	1001			
0944	4.2.4	2-82				941		2.72	
0945	"	2-84				945		2.87	23.1°C
0946	"	2-89	62F			948		2.77	3770 SCFM
0947	4.2.4	2-85				945		2.90	
0948	"	2-85✓				949		2.91	
0949	DAA				-8.0	1003			
0953	DAA				-8.0	1002			
0954	4.2.3	2-89				930		2.34	
0955	"	2-72	61F.			924		2.57	23.1°C
0956	"	2-78				919		2.70	
0957	4.2.3	2-78				928		2.59	
0958	"	2-78✓				919		2.64	
0959	DAA				-8.0	1002			
1005	DAA				-8.0	1002			
1006	4.2.2	2-54				926		2.08	
1007	"	2-52				927		2.37	23.1°C
1008	"	2-71	62F			927		2.40	
1009	4.2.2	2-83				934		2.44	
1010	"	2-89				938		2.65	
1011	DAA				-8.0	1003			
1015	DAA				-8.0	1004			
1016	4.2.1	2-60				931		1.91	23.1°C
1017	"	2-50				932		1.70	
1018	"	1-180	64F.			923		1.66	
1019	4.2.1	1-230				921		1.74	
1020	"	1-250 +3				929		1.92	
1021	DAA				-8.0	1003			

PAGE: 3
DATE: APRIL 27, 2006
SITE: ESCONDIDO
UNIT: TANK 4

MLT: 23.1°C
BAROM: 29.95 - 0.60
TDS:

HOOD: 2FT x 8FT.

TIME	STATION	ROTA NO MM/P	ROTA TEMP	ROTA PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1037	DAA				-8.0	1004-5			
1038	4.3.1	2-83				934		3.05	
1039	"	2-88	65F			930		2.86	23.1°C
1040	"	2-72				927		2.79	12 PANELS
1041	4.3.1	2-61				924		2.87	
1042	"	2-61				929		3.02	
1043	DAA				-8.0	1006			
1046.7	DAA				-8.0	1004-5			
1048	4.3.2	2-56				915		2.99	23.1°C
1049	"	2-62	66F.			917		3.07	
1050	"	2-66				920		3.10	
1051	4.3.2	2-74				922		3.12	
1052	"	2-73✓				918		3.18	
1053	DAA				-8.0	1008			
1056.5	DAA				-8.0	1007			
1058	4.3.3	2-82				940		3.16	23.1°C
1059	"	2-83	65F			943		3.19	
1100	"	2-85				944		3.01	
1101	4.3.3	2-85				947		3.11	
1102	"	2-85✓				947		3.08	
1103	DAA				-8.0	1011			
1105.4	DAA					1003-4			
1107	4.3.4	2-89				950		3.02	
1108	"	2-85	64F			953		2.92	23.2°C
1109	"	2-85				952		2.74	
1110	4.3.4	2-85				952		2.87	
1111	DAA				-8.0	1007		2.74	
1112									

PAGE: 4
DATE: APRIL 27 2006
SITE: ESCONDIDO
UNIT: TANK 4

MLT: 23.2°C
BAROM: 29.95–0.60
TDS:

HOOD: 2FT x 8FT.

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1122.3	DAA				-8.0	1008			
1123	4.4.3	2-76				920		2.91	
1124	"	2-62	63F			941		3.00	23.2°C
1125	"	2-85				949		2.99	8 PANELS
1126	4.4.3	2-84				951-2		3.14	
1127	"	2-74				950		3.06	
1128	DAA				-8.0	1009			
1133	DAA				-8.0	1008			
1134	4.4.2	1-230				920		3.10	
1135	"	1-250				922		2.95	23.2°C
1136	"	2-57	63F.			924		3.09	
1137	4.4.2	2-63				928		3.15	
1138	"	2-58✓				928		3.13	
1139	DAA				-8.0	1008			
1143	DAA				-8.0	1006			
1144	4.4.1	1-145				909		3.03	
1145	"	1-162	64F.			912		2.98	23.2°C
1146	"	1-230				917		2.99	
1147	4.4.1	1-238				924		3.02	
1148	"	1-184				924			
1149	DAA				-8.0	1007			
1210	DAA	~~[illegible]~~			-8.0	1005			
1211	4.5.1	2-73				945		2.78	23.2°C
1212	"	1-250				944		2.86	8 PANELS
1213.	"	2-74				941		2.87	
1214	4.5.1	2-50				941-2		2.88	
1215	"	2-57✓				942		2.94	
1216	DAA				-8.0	1006			

PAGE: 5
DATE: APRIL 27 2006
SITE: ESCONDIDO
UNIT: TANK 4

MLT: 23.2°C
BAROM: 29.95 -0.60
TDS:

Hood: 2FT x 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1223	DAA				-8.0	1003-4			
1224	4.5.2	2-64				948		2.97	
1225	"	2-75				947		2.90	23.2°C
1226	"	2-73				947		2.78	
1227	4.5.2	2-73	69F.			949		2.89	
1228	"	2-74✓				946		2.82	
1229	DAA				-8.0	1005			
1234	DAA				-8.0	1003			
1235	4.5.3	2-74				948		2.86	23.2°C
1236	"	2-83				939		2.87	
1237	"	2-74	69F.			937		2.92	
1238	4.5.3	2-79				939		3.00	
1239	"	2-79✓				944		2.97	
1240	DAA				-8.0	1006			
1311.5	DAA				-8.0	1008			
1313	4.6.3	2-63				924		2.91	
1314	"	2-70				931		3.04	23.3°C
1315	"	2-64	69F.			934		3.02	8 PANELS.
1316	4.6.3	2-73				933		3.09	
1317	"	2-68✓				922		3.16	
1318	DAA				-8.0	1010			
1323	DAA				-8.0	1004			
1324	4.6.2	2-74				931		2.77	23.3°C
1325	"	2-67				927		2.78	
1326	"	2-50	69F.			920		2.71	
1327	4.6.2	2-50				914		2.82	
1328	"	2-68				913		2.90	
1329	DAA				-8.0	1006			

PAGE: 6
DATE: APRIL 27, 2006
SITE: ESCONDIDO
UNIT: TANK 4

MLT: 23.3°C
BAROM: 29.95 – 0.60
TDS:

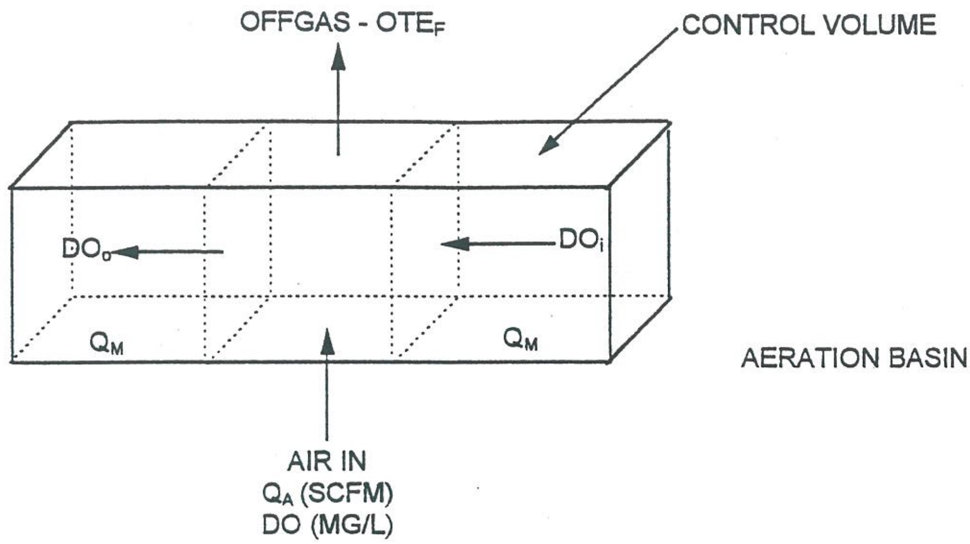
HOOD: 2FT X 8FT

TIME	STATION	ROTA NO MM/P	ROTA TEMP	PRESS	CELL PRESSURE	DIGITAL OUTPUT	CELL TEMP	DO CONC	MISC DATA & COMMENTS
1333.6	DAA				+8.0	1005			
1335	4.6.1	2-93				939		2.75	
1336	"	2-81				944		2.73	23.3°C
1337	"	2-70	68F.			947		2.71	
1338	4.6.1	2-68				945		2.77	
1339	"	2-71				946		2.92	
1340	DAA				-8.0	1007			
1352	DAA				-8.0	1008			
1353	4.7.1	2-50	67F			934		2.24	23.3°C
1354	"	2-50				936		2.46	8 PANELS.
1355	"	2-50				936		2.47	
1356	4.7.1	2-50				938		1.97	
1357	"	2-50✓				936		1.96	
1358	DAA				-8.0	1010			
1405	DAA				-8.0	1009			
1406	4.7.2	2-65				923		2.57	
1407	"	2-60	68F			926		2.36	23.3°C
1408	"	2-61				929		2.09	
1409	4.7.2	2-61				928		2.32	
1410	"	2-61✓				932		2.40	
1411	DAA				-8.0	1010			
1415	DAA				-8.0	1005			
1416	4.7.3	2-83	68F.			943		2.64	
1417	"	2-90				943		2.72	23.3°C
1418	"	2-88				945		2.69	
1419	4.7.3	2-89				943-4		2.68	
1420	"	2-88✓				943		2.72	
1421	DAA				-8.0	1006			

APPENDIX II

MASS BALANCE PROCEDURE
TO CALCULATE
OXYGEN UPTAKE RATE

OUR BY GAS-PHASE MASS BALANCE



MASS BALANCE

$$OUR = \frac{OTR - DO\ TRANSPORT}{VOLUME\ OF\ LIQUID\ IN\ CONTROL\ ZONE} - \frac{\Delta\ DO\ CONCENTRATION}{TIME}$$

WHERE:

OUR = OXYGEN UPTAKE RATE {MG/L/HR}

OTR = OXYGEN TRANSFER RATE {MG/HR}

= Q_A (1.036)(OTE_F)(454,000)

Q_A = AIR RATE TO CONTROL VOLUME IN SCFM

1.036 = LBS OF OXYGEN PER HOUR PER 1 SCFM

OTE_F = GAS PHASE OXYGEN TRANSFER EFFICIENCY UNDER PREVAILING CONDITIONS BY OFFGAS ANALYSIS, LBS OXYGEN TRANSFERED/LBS OXYGEN SUPPLIED

454,000 = MG/LB

DO TRANSPORT = (DO_{OUT} - DO_{IN})(Q_M)(60)(3.785) {MG/LHR}

Q_M = LIQUID FLOW RATE THROUGH CONTROL VOLUME IN GAL/MIN

3.785 = LITERS/GAL

VOLUME OF CONTROL ZONE = LITERS

$\frac{\Delta\ DO\ CONCENTRATION}{TIME}$ = END OF OBSERVATION PERIOD DIVIDED BY OBSERVATION TIME IN HOURS. AT STEADY STATE CONDITIONS THIS TERM IS ZERO SINCE DO IS ZERO.

APPENDIX I I I

REDMON, BOYLE, AND EWING
WPCF – NOVEMBER 1983

Oxygen transfer efficiency measurements in mixed liquor using off-gas techniques

David Redmon, William C. Boyle, Lloyd Ewing

It has been reported that approximately 1.75 million hp of aeration equipment is currently in place on the North American continent, in both municipal and industrial treatment facilities.[1] These facilities are being operated at a power cost exceeding $0.6 billion per year. Evidence suggests that the overall oxygen transfer efficiency for this equipment is low and the cost of power could be reduced by as much as 50% by improved design and operation.[1] Although there are many reasons for imperfect application of oxygen transfer devices in wastewater, one basic cause has been the unavailability of, or failure to use, optimal methods for the measurement of oxygen transfer.

The off-gas measurement technique may be a tool for obtaining more useful design data for aeration systems.

Consensus procedures for testing oxygen transfer devices in clean water are being developed.[2–4] Adequate test procedures for the assessment of aeration equipment under actual process conditions are less developed at this time. Several methods have been employed over the years to evaluate oxygen transfer in suspended growth systems. In a detailed review of these dirty water test procedures, the American Society of Civil Engineers (ASCE) Committee on Oxygen Transfer Standards outlined the assumptions required for these methods and the limitations for each procedure.[5,6] Table 1 briefly summarizes some of these constraints.

Most field test procedures may be classified according to two criteria: the presence or absence of wastewater flow (continuous versus batch tests); the rate of change of dissolved oxygen (DO) in the test volume (steady state versus unsteady state tests). In general, steady state tests are simpler to perform than unsteady state tests, but they do not provide an estimate of the effective DO saturation value in submerged aeration systems. Both procedures require an accurate determination of oxygen uptake rate (OUR) and test volume dissolved oxygen (DO) concentrations that are constant and greater than zero. In an effort to improve the accuracy of the OUR measurement and to ensure steady state conditions within the test cell with respect to DO, flow, and OUR, batch endogenous tests may be performed. These tests, however, often do not realistically project operating conditions.

Several other test procedures for field oxygen transfer are proposed to overcome some of these limitations. Tracer test methods have been used in both clean and dirty water oxygen transfer tests.[6–8] Although the procedure is very extensive and costly, the results obtained with this method are very precise and presumably accurate. The method does not require complete mixing in the test volume or aeration tank DO values greater than zero. In submerged aeration systems, however, this method suffers the same disadvantage as other steady state tests, which is the inability to estimate the effective DO saturation value.

The performance of a mass balance on oxygen in the gas phase under process conditions has been referred to as the off-gas method. This procedure offers a number of advantages over more traditional techniques currently used for this purpose. This paper describes this method, discusses its limitations, and provides data on recently conducted field studies.

HISTORICAL PERSPECTIVE

The use of off-gas measurements in biological reactors is not a new concept. Initially, off-gas analyses were performed to estimate the respiratory demand of biological cultures. As early as 1939, Sawyer and Nichols[9] described a volumetric method used in a closed system to determine the *in situ* oxygen uptake of activated sludge in the laboratory. Hoover *et al.*[10] in 1954 described a method of aeration control in a fermentation system using a paramagnetic oxygen analyzer developed earlier by Pauling *et al.*[11] Pirt and Callow[12] also used *in situ* respiratory demand measurements in studies on the continuous production of butanediol. Both oxygen and CO_2

OFFGAS PAPER

ERRATA: Page 1339

* In Equation (1), substitute $V\rho' \dfrac{dC}{dt}$ for $V\rho \dfrac{d\bar{Y}}{dt}$

* Substitute " = " for " − " between Y_{og} and $K_{L}a$

* Add Definition of ρ' = density of liquid
  after ρ = density of oxygen

Table 1—Assumptions and limitations for dirty water oxygen transfer testing.

Major assumptions	Limitations
Test cell DO constant (spatial/temporal)	Estimate of OUR
Influent flow to cell constant	Estimate of effective DO saturation value
Influent DO to cell constant	DO must be greater than zero
Test cell OUR constant	Test performed under true process conditions
Effective $K_L{}^a$ in cell constant	

were monitored by Orsat analysis. Some of the first field studies that were reported on the use of off-gas techniques to evaluate aeration devices under process conditions were outlined by Downing[13] and Downing *et al.*[14] In this method, oxygen was captured with a light hood covering all or a portion of the aeration tank [capture areas varied from 2.3 to 13.8 m^2 (25 to 149 sq ft)] and the captured oxygen was determined with a paramagnetic oxygen analyzer. These authors calculated the effective overall transfer rate ($\alpha K_L a$), therefore, both captured gas flow rate and equilibrium saturation DO had to be estimated. Gas flow rate was measured by CO_2 injection and material balance calculation. The DO saturation value for these diffused air systems was calculated by a mid-depth correction. Barker *et al.*[15] described an off-gas method used to estimate the oxygen transfer efficiency of a turbine aerator under process conditions. An inverted 0.2-m^3 (55-gal) drum was used to capture the off-gas. Oxygen was determined by a paramagnetic oxygen analyzer and transfer was expressed as percent oxygen transfer efficiency (OTE). Off gas methods were also described by Conway and Kumke[16] for analyzing a sparged turbine in clean water. Similar to the method of Barker *et al.*, off-gas was captured with a 0.2-m^3 (55-gal) container. Gas analyses were performed by both a mass spectrometer and a direct reading oxygen analyzer. Results were reported as percent OTE. Leary *et al.*[17,18] conducted extensive off-gas analyses of the Milwaukee Jones Island aeration tanks from 1967 to 1968. A 46-cm (18-in.) diameter hood was used to collect off-gas, and analyses were performed using both Orsat and gas chromatographic techniques. Data was reported as both percent OTE and $\alpha K_L a$. More recently, off-gas techniques have again been proposed as a procedure for aeration control in field installations[19,20] completing the cycle initiated by Hoover *et al.* in 1954.

THEORETICAL DEVELOPMENT

The oxygen transfer capability of a submerged air device may be estimated by means of a gas phase mass balance over the aerated volume. A number of assumptions may be made to simplify this analysis. These include the following:

- Inerts, including nitrogen are conservative; that is, there is no net absorption or desorption of the constituents in question;
- There is negligible denitrification at the test location;
- The air flow rate to the basin is constant during the test;
- The barometric pressure is constant during the test,
- The off-gas humidity is equivalent to the saturated value at mixed liquor temperature where the latter is less than the instrument inlet temperature, though in other cases it will be equivalent to the saturated value at the instrument inlet temperature; and
- Negligible oxygen transfer is taking place at the liquid surface.

Referring to Figure 1, a gas phase mass balance over the liquid volume, V, may be written

$$V\rho \frac{d\bar{Y}}{dt} = \rho q_i Y_R - \rho q_o Y_{og} - K_L a(C^* - C)V \qquad (1)$$

where

ρ = density of oxygen at temperature and pressure at which gas flow is expressed (M/L^3),
q_i, q_o = total gas volume flow rates of inlet and outlet gases (L_3/t),.
Y_R, Y_{og} = mole fractions (or volumetric fractions) of oxygen gas in inlet and outlet gases,
$K_L a$ = overall oxygen mass transfer coefficient, ($1/t$)
C^* = saturation concentration of oxygen in test liquid in equilibrium with exit gas (M/L^3),
C = equilibrium concentration of oxygen in test liquid, (M/L^3), and
V = test cell volume (L^3)

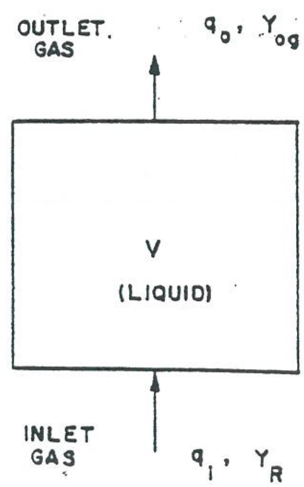


Figure 1—Gas phase mass balance.

At steady state

$$\frac{\rho}{V}(q_i Y_R - q_o Y_{og}) = K_L a(C^* - C) \tag{2}$$

If one assumes that the volume of CO_2 produced and imparted to the gas stream just equals that of oxygen absorbed, and that nitrogen is conservative, this equation reduces to

$$K_L a = \frac{\rho}{V} q \frac{(Y_R - Y_{og})}{(C^* - C)} \tag{3}$$

where $q = q_i = q_o$

The value of $K_L a$ may be estimated from Equation 2 or 3 provided that measurements are made of Y_R and Y_{og}, the inlet and outlet mole fractions of oxygen (q), the total gas flow rate, and C. In addition, an estimate must be made of C^* under test conditions.

Another method of reporting oxygen transfer is by the calculation of OTE expressed as a fraction.

$$\mathrm{OTE} = \frac{\rho q_i Y_R - \rho q_o Y_{og}}{\rho q_i Y_R} \tag{4}$$

Again, if one assumes CO_2 evolution is equivalent to oxygen absorption, this equation reduces to

$$\mathrm{OTE} = \frac{Y_R - Y_{og}}{Y_R} \tag{5}$$

Equations 4 and 5 simplify the computation of oxygen transfer in a given system because no estimate of C^* is required, although gas flow rates must be accurately monitored if a correction for CO_2 evolution is to be accounted for in Equation 4.

Gas flow measurements may be omitted from Equation 4 by using molar ratios of inlet and outlet oxygen to the inert gas fractions as given below:

$$\mathrm{OTE} = \frac{\text{mass } O_2 \text{ in} - \text{mass } O_2 \text{ out}}{\text{mass } O_2 \text{ in}}$$

$$= \frac{G_i(M_o/M_i)MR_{o/i} - G_i(M_o/M_i)MR_{og/i}}{G_i(M_o/M_i)MR_{o/i}}$$

and

$$\mathrm{OTE} = \frac{MR_{o/i} - MR_{og/i}}{MR_{o/i}} \tag{6}$$

where

G_i = mass rate of inerts (including nitrogen & argon) (M/t),
M_o, M_i = molecular weights of oxygen and inerts, and
$MR_{o/i}, MR_{og/i}$ = mole ratio of oxygen to inerts in inlet and in off-gas.

The mole ratio of oxygen to inerts may be expressed by Equations 7 and 8 as

$$MR_{o/i} = \frac{Y_R}{1 - Y_R - Y_{CO_2(R)} - Y_{W(R)}} \tag{7}$$

and

$$MR_{og/i} = \frac{Y_{og}}{1 - Y_{og} - Y_{CO_2(og)} - Y_{W(og)}} \tag{8}$$

where

$Y_{CO_2(R)}, Y_{CO_2(og)}$ = mole fractions of CO_2 in inlet gas (R) or off-gas (og), and
$Y_{W(R)}, Y_{W(og)}$ = mole fractions of water vapor in inlet gas (R) or off-gas (og).

Equations 7 and 8 may be substituted into Equation 6 to estimate OTE. It may be noted in the rare case that the mole fraction of CO_2 produced just equals that of oxygen absorbed, Equations 6, 7 and 8 reduce to Equation 5.

Finally, the value of Y_{og} and Y_R may be calculated as follows:

$$Y_R = 0.2095\,(1 - Y_{W(R)}) \tag{9}$$

and

$$Y_{og} = \left(\frac{MV_{(og)}}{MV_{(R)}}\right) Y_R \tag{10}$$

where a sensor with linear response of millivolts to partial pressure of oxygen is used and where $MV_{(og)}$, $MV_{(R)}$ are the millivolt output readings of the oxygen sensor, which have been corrected as required for absolute sensor cell pressures and temperatures.

Additional refinements in these equations for nitrogen solution or volatilization may also be made, but preliminary calculations indicate that this correction is minor and may normally be omitted from the calculations.

INSTRUMENTATION

There seems to be general agreement among investigators that off-gas techniques offer substantial advantages over most other procedures for determining oxygen transfer efficiencies for submerged aeration systems under process condition. The greatest drawbacks have been related to the instrumentation. Two practical problems had to be overcome to make the method more acceptable. The first was the need for a gas collection device that was light and easy to handle, but large enough to collect a representative off-gas sample. The early work reported by Downing[13] indicated that the smaller the hood the more variable was the measurement of $K_L a$. The second problem was the selection of an oxygen sensor that could precisely detect small differences in the partial pressure of oxygen, and be adaptable to *in situ* measurements. Over the years investigators have employed gas chromatography, paramagnetic oxygen analyzers and, more recently, polarographic probes and electrochemical galvanic cells.

The four major components of the off-gas equipment used in this investigation were a floating hood to capture the gas, a hose connecting the hood to the analytical circuit, an analytical circuit for monitoring off-gas composition, temperature, pressure, and gas flow rate, and a vacuum source to draw gas from the hood through the analytical circuit.

The hood used for these studies was a section of a 0.6-m (2-ft) diameter pipe cut longitudinally along the pipe diameter to a length of 2.67 m (8.75 ft), to provide a surface capture area of 1.62 m^2 (17.5 sq ft). The hood was provided with ballast tanks to ensure that it remained stable within a given sampling cross section. A 38-mm (1.5-in.) diameter connecting host carried the exhaust gases to an analytical circuit. Pressure under the hood was monitored by means of plastic tubing leading from a port on the hood to the analytical circuit. Suction of exhaust gas from the hood was provided by a vacuum cleaner. The suction line was valved to maintain a small, but constant negative or positive pressure (±0.2 in.) under the hood. A slight but constant vacuum on the order of −4.0 to −6.0 in. water was maintained in the analytical circuit when off-gas and reference air measurements were made.

The analytical circuit is depicted in Figure 2. A polarographic DO probe was used to measure oxygen partial pressure in the off-gas and reference air samples. Later, during the investigations, an electrochemical galvantic cell was used in series with the probe. Carbon dioxide was monitored batchwise by bleeding off gas flowing through the circuit to a volumetric CO_2 analyzer.

The gas was analyzed in this circuit by passing a small portion of the test gas through a flowmeter and past the

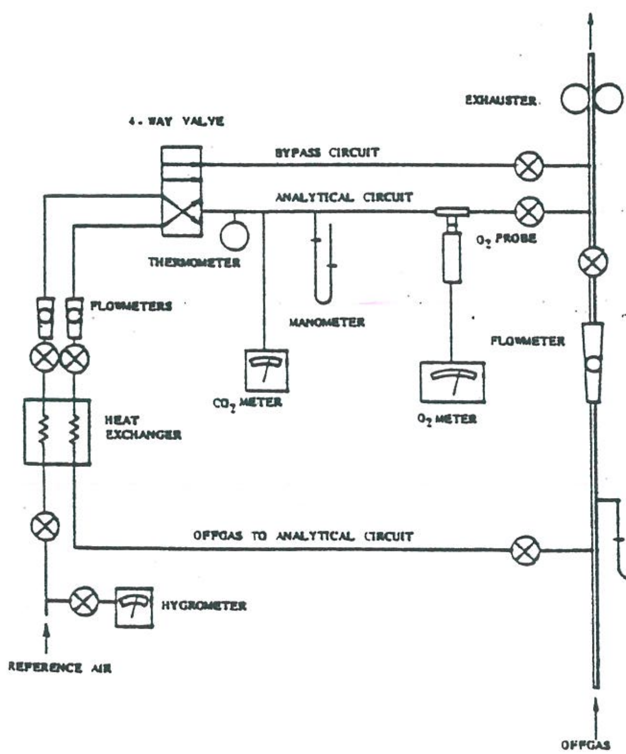


Figure 2—Schematic diagram of off-gas analyzer circuit.

oxygen probe. Gas temperature, humidity, and pressure were monitored and controlled within the circuit so that the difference in partial pressure of oxygen between the reference air and off-gas could be precisely obtained.

CONDUCT OF THE TEST

Prior to the field test, the DO sensor was checked daily for accuracy with gases of certified composition. Pure nitrogen was also passed through the analytical circuit under test conditions to ensure that no leaks existed. These tests were normally conducted several times during the day.

The linearity of the probe was checked periodically by drawing the reference air past the sensor under various levels of reduced pressure while keeping the gas temperature constant. Under these circumstances the partial pressure of oxygen was directly proportional to the total absolute pressure. The criterion applied was that the ratio of absolute pressures for the two conditions divided by the ratio of the meter outputs fell in the range of 0.995 to 1.005.

Using this test for linearity, it was found that the calibration setting for probe output (in millivolts) was not critical, and essentially the same relative change in the voltage output occurred regardless of the setting with reference at ambient conditions. To obtain the maximum sensitivity, the reference output was set as close to full scale as was practical, because probe error is normally a fixed fraction of the full-scale reading. It should be emphasized that this procedure required precise measurement of the difference in the two gas streams and, therefore, it was not necessary to have an accurate determination of the absolute value of the partial pressure of oxygen in either stream.

Once the analytical circuit was checked, the gas collection hood was fixed in place at a predetermined location and a vacuum drawn at the instrument discharge. Reference air was first drawn into the analytical circuit and, a series of observations of temperature, humidity, pressure, and sensor millivolt readings were recorded over a period of about 5 minutes. A portion of the off-gas was diverted through the analytical circuit for a typical period of 5 to 10 minutes. During this time adjustments to the volume of off-gas drawn from the collection hood were made to obtain an equilibrium pressure condition beneath the hood at near ambient pressure (±0.2-in. water). Parallel measurements were recorded for off-gas temperature, humidity, pressure, CO_2 concentration, and sensor millivolt reading. Total off-gas flow rate was also recorded and used later in calculation of a bulk OTE for the entire tank volume. In addition to these measurements, mixed liquor temperature, DO, and local barometric pressure were also recorded. The hood was then moved to a new location and reference gas was again drawn through the circuit and measurements recorded. The reference gas check between each off-gas measurement provided a means for simple probe drift correction if required. This technique reduced errors in measuring the differences between off-gas and reference streams. The entire procedure for one determination required approximately 15 minutes.

The method involved sampling a sufficient number of locations within a given basin to obtain a representative sample of off-gas from the tank or tank element. A reasonable agreement between the applied and collected air flow rates was used to indicate a representative sampling layout. Initial field studies were conducted by drawing reference air directly from the plant air line. However, experience at a number of plants indicated that ambient air drawn directly within the vicinity of the analytical system yielded nearly identical readings to plant air. This approach greatly simplified the test procedure, and was adopted as a standard procedure thereafter.

The value of OTE for each sampling point was calculated using Equations 6, 7, 8, 9, and 10. Temperature corrections were made using the expression:

$$OTE_{20^\circ} = OTE_T\, 1.024^{(20-T)} \tag{11}$$

In order to estimate a weighted bulk average OTE for an entire tank or tank element, the following equation was used.

$$OTE_w = \frac{\sum_0^n OTE_n q_{an}}{\sum q_{an}} \tag{12}$$

where

OTE_w = weighted OTE

OTE_n = OTE value at sampling point, n, and

q_{an} = the collected gas flow rate per unit surface area at sampling point, n, (L^3/tL^2).

To provide comparisons between values of OTE measured for different systems, one may normalize the measured OTE by dividing it by $(C^* - C)$,

$$OTE_{sp} = \frac{OTE}{C^* - C} \tag{13}$$

In this equation, C^* is the calculated saturation value of oxygen in the wastewater. In diffused air systems C^* may be estimated by

$$C^* = C^*_{20} \frac{P_b}{P_s} \frac{C_{sT}}{C^*_{s20}} \beta \tag{14}$$

where

C^*_{20} = clean water saturation value in the same geometry at comparable air flow rate and at standard conditions (M/L^3),

P_b = atmospheric pressure during the test (f/L^2),

P_s = standard atmospheric pressure (usually 1.00 atm at 100% relative humidity (f/L^2),

C_{sT} = surface saturation book value for dissolved oxygen at the test temperature (M/L^3),

C^*_{s20} = the surface saturation book value for dissolved oxygen at 20°C (M/L^3), and

β = the ratio of process wastewater saturation value to that of clean water.

The value of C^*_{20} is often available from clean water tests of the diffuser system under the appropriate process configurations and gas flows. If it is not available it must be calculated using a gas-side correction model assuming some effective saturation depth.[2]

The data presented in this paper are uncorrected field measured OTE values except where subscripted as OTE_{20° or OTE_{sp}.

RESULTS

Off-gas field studies have been conducted during the past 3 years at over nine treatment facilities, including two industrial plants, and a variety of diffused air systems. Early tests were conducted at Whittier Narrows, California, Madison, Wisconsin, Brandon, Wisconsin, Ridgewood, New Jersey, and a DuPont facility. These tests were conducted in parallel with other field test procedures for comparative purposes. Other off-gas studies were conducted for a variety of purposes including:

- Evaluation of diffuser clogging problems,
- Evaluation of effectiveness of diffuser cleaning procedures,
- Evaluation of several diffuser types in side-by-side tests under process conditions, and
- Evaluation of aeration system control procedures.

Comparative test results. Table 2 presents the results of comparative tests conducted at four field sites. In general, the comparisons of the off-gas procedure with other currently-used field techniques are good. These tests reported were conducted under ideal conditions where the comparative test methods were applicable.

For example, field tests at Sites A and B were conducted at plants where mixed liquor DO values generally exceeded 0.5 mg/L and where approximate steady state conditions were achieved. Site A used both tapered aeration and step aeration thereby producing a wide variation in point values of OTE as measured by off-gas methods. Point values of steady state respirometeric tests versus off-gas data were, of course, not applicable, but overall bulk average values were comparable. At Site B, the small aeration tank was almost completely mixed with respect to DO. Point values of off-gas OTE varied substantially along test cross sections depending on off-gas flow rates (discussed later), but comparisons between tests were reasonable.

Results at Site C, another municipal plant, were also very favorable. At this plant, a non-steady state method with hydrogen peroxide was used for comparison. This

Table 2—Selected field test results off-gas versus other field methods.

Site	Diffuser system	Off gas analyses OTE-% (avg)	Off gas analyses OTE-% (range)	DO-mg/L (range)	OTE-% (comparison test)	Comparison method
A	Floor coverage ceramic	11.85	8.0–16.7	0.9–3.9	12.68	Respir. rate steady state
A	Dual roll coarse bubble	6.16	5.1–6.8	2.0–4.1	6.29	Respir. rate steady state
B	Jet aeration along longitud. wall					
	(q = 67 L/s)	5.34	4.5–7.7	3.8[a]	5.31	Respir. rate steady state
	(q = 18 L/s)	9.58	8.7–11.1	1.1[a]	12.5	Respir. rate steady state
C	Spiral roll coarse bubble	3.2	—	1.8–4.1	3.3	H_2O_2/non-steady state
C	Spiral roll coarse bubble	1.1	—	6.8–7.2	1.7	H_2O_2/Non-steady state
D	Floor coverage coarse bubble	7.7	5.5–11.1	0	7.3–7.8[b]	Radioactive tracer

[a] Average DO in approximately CSTR system.
[b] Range of values depending on actual air flow rate.
[c] All OTE values calculated under field conditions and not corrected to 20°C.

method, described by Kayser,[5,21] must be performed under steady state conditions. Such conditions were established at Site C by diverting a substantial amount of the wastewater flow from the test system. Of special note, here, was the excellent agreement achieved between these two methods at an OTE value of 3.3%. The apparent poor agreement at the second test condition was not surprising considering the extremely low operating efficiency at that condition. The effectiveness of off-gas methods at low OTE values was of great concern to the investigators owing to the extremely small differences between reference air and off-gas oxygen concentrations. The excellent comparability of these methods at Site C was further reinforced by nonradioactive tracer tests performed at this site.

The tests conducted at Site D were reported by Campbell.[6] Here, the radioactive tracer technique using Krypton-85 and tritium was compared against the off-gas procedure. Excellent agreement was reported between the two methods in this highly loaded, single aeration tank where complete-mix conditions were approximated. The wide range of off-gas OTE point values measured at this facility was primarily the result of the variations in off-gas flow rates [3.4 to 4.5 L/m^2/s (0.67 to 0.88 cfm/ft^2)]. Even though the coarse bubble diffusers were distributed over the entire tank floor, there was a substantial localized boiling along the tank surface.

Important to note in all of these comparative tests, is the method of computation used to convert $\alpha K_L a$ values, which were estimated by all the other field test procedures, to OTE values measured by the off-gas procedure. The conversion was executed by the following general relationship,

$$OTE = \frac{\alpha K_L a(C^* - C)V}{q\rho} \quad (15)$$

To achieve this comparison, it was necessary to have good estimates of C^* for the diffuser system geometry in the process wastewater at the appropriate temperature, pressure, and q, the gas flow rate of the diffuser system.

Other test results. The value of off-gas measurements goes far beyond the ability to estimate bulk OTE values for a given system. No other field test for oxygen transfer can provide discrete point information for a given system. The analysis of oxygen transfer at a specified location can greatly benefit the design engineer and the operator. A few examples of this information are presented to illustrate this extra benefit of the method.

Figure 3 presents the results of OTE and off-gas flow rate measurements that were observed at twelve cross-sections along the aeration tank at Site A. Floor coverage ceramic diffusers were used at this site. Tapered aeration was employed and primary effluent was discharged in equal amounts at three points along this folded tank. Tapered air was achieved in these tanks by varying diffuser density and, to some extent, by throttling gas flow rates to the diffuser headers. Aeration control was accomplished through DO monitoring and manual shut-down or start-up of blowers. Figure 3 indicates that during this period, DO control appeared to be effective. Because OTE for ceramic diffusers are relatively insensitive to air flow rates per diffuser, it seemed that increases in OTE along each tank section were the result of increases in alpha values.

A similar plot for the cross roll configuration at this same site appears as Figure 4. These folded tanks were equipped with fine bubble tubes in the first bay and coarse bubble units in the last two bays. All diffusers were uniformly spaced and primary effluent was evenly split to three points. This data suggests that gas flow rates were not well distributed, perhaps because of ineffective throttling of air control valves. The decline in OTE values along the tank length was not expected and could not be

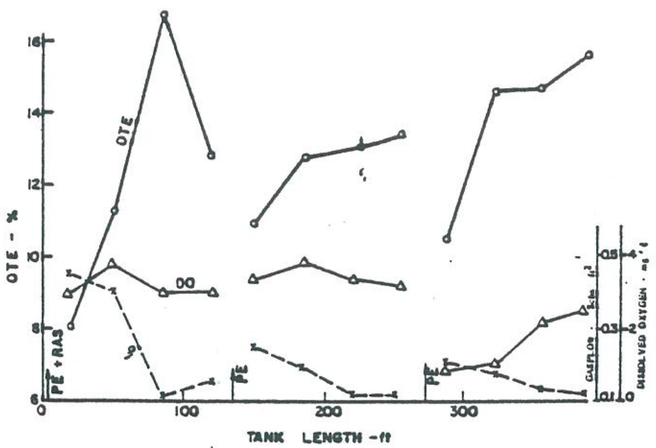


Figure 3—Gas transfer analyses along tank length—Site A. Floor coverage, ceramic domes, and tapered air.

entirely attributed to changes in gas flow rates. It is clear that operational corrections in this system were necessary.

An additional study was conducted near the discharge end of a three-pass cross roll system with various types of wide band tubular diffusers. A plan view of the tank indicating the header locations, diffuser locations and sampling plan is shown in Figure 5. The test results are presented in Table 3. Of particular interest was the apparent utility of the gas phase analysis which permits evaluation of several aerator types within a relatively small portion of a single tank. With tracers or liquid phase methods, such analysis is not possible.

The used units had been operating at the test plant for about 3 years. Comparing new and used Type C devices, there was a reduction in performance over that period. A similar reduction had most likely occurred with Type D diffusers, however, new Type D units were not available for test at that time.

It should be noted that the discharge end of the basin was selected to minimize potential alpha variations in the test region. The diffuser layout was such that the

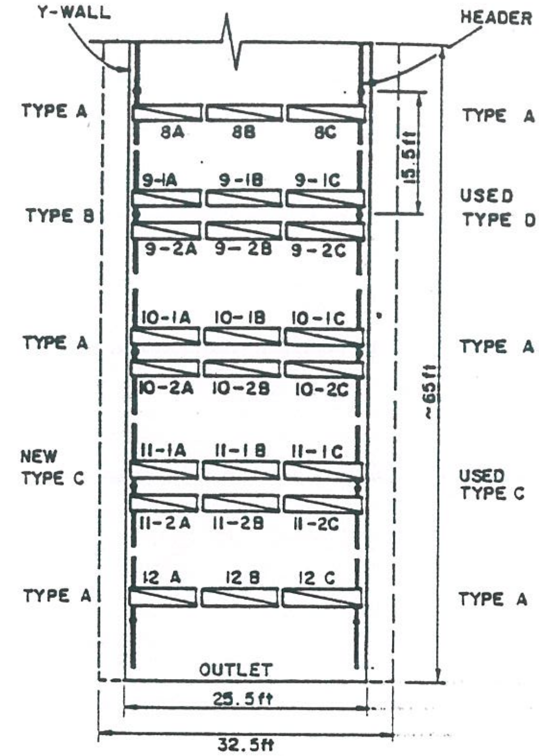


Figure 5—Aerator and sample site layout for comparative analysis of diffusers.

reference system, Type A, was on both ends of the test zone to pick up the relative apparent change in alpha across the test boundary. The data substantiated a constant alpha for the test region. Based on the above results, this method can be a useful tool in aeration system retrofitting consideration. It can measure the dirty water performance characteristics of various systems at a particular site under actual field conditions, with modest expenditure of time and effort. However, these systems should not be com-

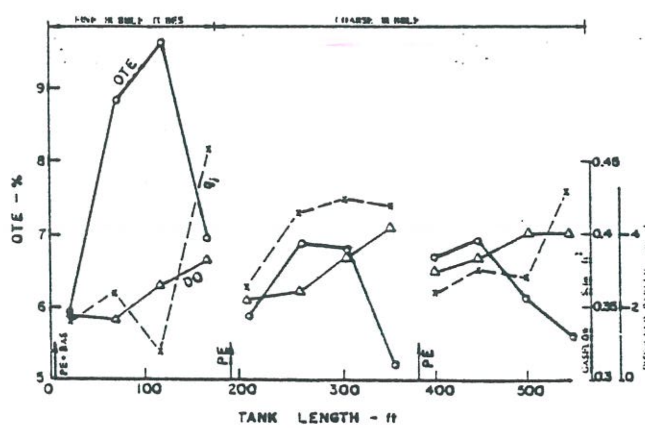


Figure 4—Gas transfer analyses along tank length—Site A. Cross roll, fine, and coarse bubble.

Table 3—In-process tubular diffusers comparisons with off gas procedures.

Diffuser	Location[a]	OTE-%[b]	DO-mg/L	$OTE_{20°-0}$[c]
Type A	Station 8	7.95	0.9	8.29
Type B	Station 9	11.62	1.7	14.18
Used Type D	Station 9	9.17	1.7	11.33
Type A	Station 10	7.96	2.0	10.28
New Type C	Station 11	12.94	1.7	15.96
Used Type C	Station 11	9.05	1.7	11.00
Type A	Station 12	6.99	1.2	8.29

[a] See Figure 5.
[b] Field OTE.
[c] OTE corrected 20°C at 0 DO (Equations 11 and 13).

pared solely on the basis of OTE measurements when new, because other factors including back pressure, power, available driving force, and maintenance should also be considered.

Sample point selection in evaluating oxygen transfer data for a given system is critical to proper assessment of the system. Figure 6 presents a layout of the aeration tank at Site B that was equipped with jet aeration along one longitudinal wall directed across the tank. Although DO values were uniform in this small basin, off-gas flow rates and OTE values were not (Table 4). As would be expected, OTE values were generally higher over points of lower gas flow discharge. To estimate overall basin transfer efficiency using Equation 12, total captured gas flows were compared against measured values. Accordance between the two (±15%) indicates a reasonable sample point selection. In this particular study, gas capture flow rates were much higher than the rated capacity of the blowers. Further evaluation of the rating curves by the manufacturer revealed that the blower capacity had been seriously underestimated. No gas flow metering devices were available at Site B.

DISCUSSION

The results of 3 years of field experience with the off-gas procedure have been very encouraging. The procedure is relatively simple and straightforward, and the equipment required for precise and accurate *in situ* measurement is available. Hood designs will continue

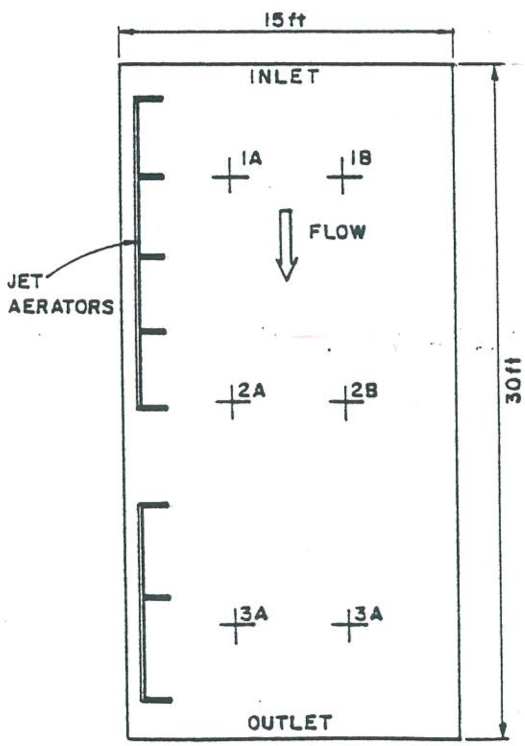


Figure 6—Aerator and sample site layout at Site B.

Table 4—Off gas analysis—Site B[a]

Station	q_A-L/m²/s	OTE-%
1A	0.34	11.0
1B	0.80	9.3
2A	0.79	6.9
2B	0.75	8.8
3A	0.73	10.2
3B	0.40	11.1
Weighted Average	—	(9.58)
1A	2.21	4.5
1B	0.66	7.7
2A	2.27	4.9
2B	2.09	6.2
3A	2.61	4.4
3B	0.31	11.9
Weighted Average	—	(5.34)

[a] See Figure 6.

to improve so that they are lighter and more portable than the ones used in this study.

There is currently no way to effectively evaluate the precision of this method under process conditions because it is difficult to achieve constant performance of aeration equipment under field conditions. Under the most uniform aeration conditions, reproducibility seems to be well within acceptable ranges for this type of field measurement (less than ±10%). It should be noted that the estimate of the reproducibility of the method may be primarily the result of changing conditions at a given sampling point rather than the precision of the analytical system.

The accuracy of this technique cannot be determined because, to date, there is no standard against which to compare field OTE measurements. Many researchers feel that the radioactive tracer procedure described by Neal *et al.*[7] represents the state-of-the art available today for oxygen transfer rate measurement under process conditions. The results of the study at Site D were very encouraging in this regard. It should be remembered, however, that the tracer measurements provided a bulk average $\alpha K_L a$, whereas the off-gas measurements represented an average of local oxygen transfer efficiencies. It is also pertinent that where the objective of the test is to measure efficiency or predict air rate requirements, the off-gas method is significantly less subject to errors resulting from air rate measurement.

The accuracy of the off gas-measurement for prediction of local OTE is dependent on a number of measured variables including gas phase oxygen concentration, gas phase carbon dioxide concentration, gas phase humidity, gas phase temperature, gas phase pressure, and rate of off-gas flow. The accuracy of the oxygen sensor was continuously checked using certified gases or reduced pressure as described earlier. Results with the sensors used in this study were excellent, which indicates a high degree of precision and accuracy. Most sensors are temperature compensated. The effectiveness of this compensation varies, therefore each device should be carefully checked. Gas phase CO_2 and humidity seem to be the other major variables that influence computation of OTE. Off-gas flow rate measurement is used in Equation 12 primarily to weight OTE values. Its accuracy, therefore, is of secondary interest. On the other hand, if Equation 4 is used to estimate OTE values, a very accurate assessment of off-gas (and inlet gas) rates is required. Downing[13] employed CO_2 injection in the outlet gas with subsequent CO_2 analysis in the gas stream to ensure accurate estimates of gas flow. A sensitivity analysis of the parameters influencing the calculation of OTE will be published in the near future.

In evaluating the off-gas procedure against other methods currently available, several advantages and disadvantages may be enumerated. Some off-gas methods are particularly suitable because they:

- Measure local performance,
- Yield OTE directly and are relatively insensitive to the precision of air flow measurement,
- Are applicable in tanks where spatial variation of gas flow rate, loading, DO, and alpha exist,
- Seem to have exceptionally good precision and accuracy as compared to other conventional methods,
- Produce relatively fast and inexpensively,
- Provide a simple and reliable means of measuring alpha values in aeration systems of known clean water performance,
- Provide a means for simultaneous side-by-side comparisons of different aeration systems under process conditions,
- Are applicable in anoxic tanks, and
- Do not require process interruption.

Off-gas methods can be disadvantageous because:

- Technique is not applicable to mechanical aeration systems;
- Tanks must be accessible to personnel,
- Severe foaming may complicate gas sampling,
- Severe turbulence may cause difficulty in hood placement, and
- Method requires accurate measurements of CO_2 and humidity of reference air and off-gas.

Additional field research continues with the off-gas procedure. The method has a wide list of applications that extend beyond the routine measurement of bulk transfer efficiency. As indicated previously, off-gas provides air distribution and transfer efficiency profiles that can be used to evaluate system operation and maintenance requirements. Furthermore off-gas may be used to monitor temporal changes in oxygen transfer caused by diffuser clogging, alpha variations, and gas flow adjustments. Recently the off-gas method has been used to evaluate a variety of diffuser devices in side-by-side tests under process conditions.

The translation of clean water oxygen transfer data to process conditions continues to produce a significant amount of uncertainty in the design of aeration systems. Through the use of field measuring techniques, a compilation of useful data will eventually be accumulated to provide the design engineer with better scale-up data. Hopefully, the off-gas technique will provide an additional tool to achieve that end.

ACKNOWLEDGMENTS

Credits. The authors wish to acknowledge the advice and counsel of the ASCE Oxygen Transfer Standards Committee whose work in the development and verification of oxygen transfer testing and evaluation methodology has been supported in part by cooperative research agreements with U. S. EPA over the last 5 years. The valuable assistance and advice given by Jerome Wren, Sanitaire, Jeffrey Brochtrup, Madison Metropolitan Sewerage Commission, and Bob Arthur, Arthur Technology is also gratefully acknowledged. This paper was presented at the 55th Annual Conference of the Water Pollution Control Federation in St. Louis, Mo. on October 7, 1982.

Authors. David Redmon and Lloyd Ewing are engineers at Ewing Engineering Co., Milwaukee, Wis. William Boyle is a professor of civil and environmental engineering at the University of Wisconsin-Madison. Correspondence should be addressed to William Boyle at 3230 Engineering Bldg., University of Wisconsin-Madison, Madison, WI 53706.

REFERENCES

1. Barnhart, Edwin L., "An Overview of Oxygen Transfer Systems." Proc. of Workshop on Aeration System Design, Operation, Testing and Control, EPA/Environment Canada, Aug. 2–4, 1982, Madison, Wis. (Accepted for publication).
2. ASCE Oxygen Transfer Committee, "Development of Standard Procedures for Evaluating Oxygen Transfer Devices—Section 4—Modeling and Data Interpretation." EPA, U. S. Environ. Prot. Agency, Cincinnati, Ohio (1983).
3. Paulson, W., and Baillod, C. R., "Specifications and Compliance Testing—Clean Water Tests." Proc. of Workshop on Aeration System Design, Operation, Testing and Control, EPA/Environment Canada, Aug. 2–4, 1982, Madison, Wis. (Accepted for publication).
4. Black, S., "Aerator Testing Performance—The Ontario Perspective." Proc. of Workshop on Aeration System Design, Operation, Testing and Control, EPA/Environment Canada, Aug. 2–4, 1982, Madison, Wis. (Accepted for publication).
5. ASCE Oxygen Transfer Committee, "Development of Standard Procedures for Evaluating Oxygen Transfer Devices—Section 7—Oxygen Transfer Measurements in

Respiring Systems." EPA, U. S. Environ. Prot. Agency, Cincinnati, Ohio (1983).

6. Campbell, Jr., H. J., "Oxygen Transfer Testing Under Process Conditions." Proc. of Workshop on Aeration System Design, Operation, Testing and Control, EPA/Environment Canada, Aug. 2–4, 1982, Madison, Wis. (Accepted for publication).
7. Neal, L. A., "Use of Tracers for Evaluating Oxygen Transfer." Proc. Workshop Toward an Oxygen Transfer Standard, EPA–600/9–78–021, U. S. Environ. Prot. Agency, Cincinnati, Ohio (April 1979).
8. Madsen, P. P., *et al.*, "Oxygen Transfer Rate Measurement in Activated Sludge by the Gaseous Tracer Method." *VaHen*, **37**, 13 (1981).
9. Sawyer, C. N., and Nichols, M. S., "Activated Sludge Oxidations—I. Effect of Sludge Concentration and Temperature on Oxygen Utilization." *Sew. Works J.*, **11**, 52 (1939).
10. Hoover, S. R., *et al.*, "Measurement of Oxygen Transfer." *Inst. and Autom.*, **27**, 774 (1954).
11. Pauling, L., *et al.*, "An Instrument for Determining the Partial Pressure of Oxygen in a Gas." *J. Am. Chem. Soc.*, **68**, 795 (1946).
12. Pirt, S. J., and Callow, D. S., "Exocellular Product Formation by Microorganisms in Continuous Culture I. Production of 2:3–Butanediol by Aerobacter aerogenes in Single Stage Process." *J. Appl. Bacteriol.*, **21**, 188 (1958).
13. Downing, A. L., "Aeration in the Activated Sludge Process." *Inst. Public Health Eng.*, **59**, 80 (1960).
14. Downing, A. L., *et al.*, "Aeration in Biological Purification Processes." Soc. Chem. Ind. Monograph No. 12, Page Bros. (Norwich) Ltd., UK, 187 (1961).
15. Barker, W. G., *et al.*, "Pharmaceutical Waste Disposal Studies—II. Turbine Mixer Aeration in Activated Sludge Plant." Proc. 15th Purdue Ind. Waste Conf., Lafayette, Ind., 58 (1960).
16. Conway, R. A., and Kumke, G. W. "Field Techniques for Evaluating Aerators." *J. Sanit. Eng. Div. Am. Soc. Civil Eng.*, **92**, SA2, 21 (1966).
17. Leary, R. D., *et al.*, "Effect of Oxygen Transfer Capabilities on Wastewater Treatment Plant Performance." *J. Water Pollut. Control Fed.*, **40**, 1298 (1968).
18. Leary, R. D., *et al.*, "Full Scale Oxygen Transfer Studies of Seven Diffuser Systems." *J. Water Pollut. Control. Fed.*, **41**, 459 (1969).
19. Kubota, H., *et al.*, "Automatic Monitoring of Exhaust Gas Analysis in Activated Sludge Wastewater Treatment Process." *Water Sci. Technol.*, **13**, 159 (1981).
20. Tanuma, R., *et al.*, "Dissolved Oxygen Control Using Aeration Exhaust Gas." *Water Sci. Technol.*, **13**, 183 (1981).
21. Kayser, R., "Messung der Sauerstoffzufuhr in Reinwasser und Unter Betriebsbeding ungen." ("Measurements of Oxygen Transfer in Clean Water and Under Process Conditions".) IAWPR Specialized Conference on Aeration, Amsterdam, The Netherlands, Sept. 19–22, 1978, 22 (1978).