Brown AND Caldwell

Prepared for
City of St. Petersburg, Florida

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Southwest Water Reclamation Facility Treatment Process and Hydraulic Evaluation

DRAFT

April 16, 2012

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City of St. Petersburg's Southwest Water Reclamation Facility Treatment Process and Hydraulic Evaluation

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List of Abbreviations

˚C	degrees Celsius
˚F	degrees Fahrenheit
2Dc	two-dimensional secondary clarifier hydrodynamic model
AADF	annual average daily flow
AOR	actual oxygen rate
AWWRF	Albert Whitted Reclamation Facility
BFP	belt filter press
$CBOD_5$	carbonaceous biochemical oxygen demand, 5 days
CCB	chlorine contact basin
CCT	chlorine contact tanks
CFD	computational fluid dynamics
COD	total chemical oxygen demand
CT	contact time
DO	dissolved oxygen
ESS	effluent suspended solids
FDEP	Florida department of environmental protection
ffCOD	floc and filtered chemical oxygen demand
g/L	grams per litter
GBT	gravity belt thickener
gpd	gallons per day
gph	gallons per hour
HLR	hydraulic loading rate
ISS	inorganic suspended solids
k	sludge-specific settling parameter
K_A	floc aggregation rate coefficient
K_B	floc break-up rate coefficient
MDF	maximum daily flow
MG	million of gallons
mg/L	milligrams per litter
MGD	million of gallons per day
mL/g	millilitter per gram
MLSS	mixed liquor suspended solids
MMF	maximum month flow
MOP	manuals of practice
NH_3-N	ammonia as nitrogen
NH_4	ammonium ion
NO_2-N	nitrite as nitrogen
NO_3-N	nitrate as nitrogen

pCOD	particulate chemical oxygen demand
PHF	peak hour flow
PO_4-P	phosphate
pph	pounds per hour
RAS	return activated sludge
SBD	sludge blanket depth
Scfm	standard cubic feet per minute
sCOD	soluble chemical oxygen demand
SLR	solids loading rate
SOR	surface overflow rate
SOTE	standard oxygen transfer efficiency
SRT	solids retention time
sTKN	soluble total kjeldahl nitrogen
sTP	soluble total phorsphorus
SVI	sludge volume index
SWD	side water depth
SWWRF	Southwest Water Reclamation Facility
TKN	total kjeldahl nitrogen
TN	total nitrogen
TP	total phosphorus
TSS	total suspended solids
V_o	initial reference settling velocity
VS	average volatile solids
V_s	interface settling velocity
VSS	volatile suspended solids
WAS	waste activated sludge
WRF	water reclamation facility

Executive Summary

The Southwest Water Reclamation Facility (SWWRF) is one of the four water reclamation facilities owned and operated by the City of St. Petersburg. The facility is permitted to treat an annual average of 20 million gallons per day (MGD) of wastewater generated in the southwest section of the City. Final effluent at the SWWRF is distributed in the City's reclaimed water system or disposed through deep injection wells located on the plant property.

The City is currently considering removing the Albert Whitted Water Reclamation Facility (AWWRF) from service and diverting that wastewater to the SWWRF. The plan is to convert the AWWRF to a pump station and to divert all of its wastewater to the SWWRF for treatment and disposal. As an effort to be proactive and to plan for future needs, the City desired to investigate the treatment capacity of the SWWRF and to determine possible hydraulic and process bottlenecks that could restrict the facility for handling the additional flows and pollutant loadings generated in the AWWRF's service area.

A scope of work was developed by Brown and Caldwell and approved by the City with the objective of establishing the maximum treatment capacity for the SWWRF to meet the existing effluent requirements including the flows and pollutant loadings from the AWWRF. In addition, the scope of work included planning and recommendations to eliminate hydraulic and treatment process bottlenecks at the SWWRF to handle its permitted capacity of 20 MGD. A combination of historical data analysis, special sampling data collection, hydraulic and process modeling, and mass balances were used to assess the treatment capacity of the liquid and solids processing units at the SWWRF.

Based on these treatment evaluations, a number of recommendations have been made and are provided in this report. The recommendations are based on the prevailing conditions at the time of its preparation. The recommendations are intended to provide the City with strategies to plan for future needs at the SWWRF.

Summary of Hydraulic and Treatment Process Assessment

Historical daily operational data, from January 2007through November 2011, for the SWWRF and AWWRF were reviewed to determine current and future flows and pollutant loadings for the SWWRF. This information was complemented by a three week wastewater characterization study to determine important influent information and performance at the facility. Hydraulic and process models for the SWWRF were built and calibrated to simulate current and future loading conditions for the SWWRF.

The treatment process evaluation presented in this report did not include the potential capacity of the "old plant", which is currently permitted for 4 MGD AADF.

The hydraulic modeling evaluation of the SWWRF shows the facility to be capable of hydraulically passing the projected peak hour flow of 40 MGD if all the process units present in the "new plant" are operational. In the case that one secondary clarifier is out of service, the hydraulic capacity of the plant will reduce to approximately 37 MGD PHF. At flows exceeding 37 MGD, wastewater would still pass through the plant; however, the Clarifier weirs would be submerged, increasing the amount of solids passing into the secondary effluent and carrying into the filters.

The treatment process evaluation indicates that the existing treatment capacity of "new plant" facilities is limited by the secondary clarification system at approximately 17 MGD AADF. The aeration system (for carbonaceous removal only), filtration and disinfection facilities have treatment capacities exceeding 20

MGD AADF. In the case that nitrification occurs at the facilitiy, the capacity of the aeration system will be limited to 15 MGD AADF.

Evaluation of the sludge handling facilities at the 20-MGD design flow showed that the WAS holding tank and the gravity belt thickener apparently limit the overall plant capacity. The WAS holding tank is used to minimize the operating schedule of the GBT, and if the GBT hours of operation are increased, the WAS holding requirements decrease. Consequently, the W AS holding tank is not considered to be a true limiting process. The limiting process is the GBT. Based on the results of this plan, the treatment capacity of the GBT will be exceeded when the average flow reaches approximately 11.5 MGD based on a schedule of 12 hours per day, 7 days week. Even though, the treatment capacity of the GBT could be increased by extending the run time of the unit, it was not prudent to assume this since it is already almost continuously. The existing anaerobic digesters (two units) and BFPs have adequate capacity to stabilize and dewatered the thickened sludge produced at the plant when influent flow reaches 20 MGD AADF.

Recommendations to Increase Treatment Process Capacity

Based on the treatment process results presented in this plan, the treatment capacity of the facility is limited by the secondary clarification capacity of the “new plant” at 17 MGD AADF. The treatment capacity is limited by the combination of the projected peak hour flow and the high mixed liquor concentration. Therefore, treatment process alternatives were evaluated to reduce the operating mixed liquor in the aeration basins. Two possible treatment modifications were analyzed, being the step-feed process and the addition of a new primary clarification system. Both alternatives will increase the capacity of the existing secondary clarification process at the SWWRF. The addition of new primary clarifiers will not only increase the capacity of the plant but it will significantly reduce the aeration requirements in the biological process and will increase the biogas production in the anaerobic digesters. Therefore, even though the addition of the new primary clarifiers might have a larger capital investment, it will reduce the overall costs associated with operation by decreasing the aeration requirements and increasing the biogas production, which could subsequently be used to produce energy at the plant.

The new fine-bubble aeration system was designed for carbonaceous removal only. Brown and Caldwell recommends effective automatic DO control and online ammonia analyzers be implemented at the SWWRF to control the aeration system. This will provide operational flexibility to minimize nitrification and to produce good-quality settling sludge, which was an important assumption during this plan.

The sludge processing facilities are limited by the GBT. Therefore, one additional GBT is required at the SWWRF to handle the sludge production at flows of 20 MGD AADF.

Section 1

Introduction

1.1 Purpose

The City of St. Petersburg owns and operates four water reclamation facilities (WRF). The Southwest WRF (SWWRF), located at 3800 54th Avenue South, is permitted to treat an annual average daily flow (AADF) of 20 million gallons per day (MGD) of wastewater generated in the southwest section of the City. From 2007 to 2011, the plant flow averaged approximately 9.4 MGD. For 2011, the plant flow averaged about 10.4 MGD. Water treated at the SWWRF is distributed in the City's reclaimed water system or disposed through deep injection wells located on the plant site.

The SWWRF was originally designed to meet secondary effluent standards (carbonaceous and TSS removal only) and to produce effluent suitable for reuse. The 20-MGD facility was designed and is permitted to consist of an "old" plant with a capacity of 4 MGD and a "new" plant with a capacity of 16 MGD.

The existing treatment system consists of the following unit processes.

- Influent pumping
- Headworks with screening and grit removal
- Aeration basins using surface aerators
- Secondary settling tanks
- Dual media filtration (anthracite and sand)
- Chlorination for high-level disinfection
- Ground storage
- Effluent pumping to reuse
- Sludge thickening using gravity belt thickening
- Anaerobic digestion
- Sludge dewatering using belt filter presses

In an effort to upgrade aging equipment and to operate more efficiently, the City is currently converting the existing surface aerators to a new system that uses fine bubble diffusers with centrifugal blowers. In addition, new headworks is currently being built at the facility to improve preliminary treatment, reliability and to increase capacity.

The City is currently considering removing the Albert Whitted Water Reclamation Facility (AWWRF) from service and diverting that wastewater to the SWWRF. The plan is to convert the AWWRF to a pump station and to divert all of its wastewater to the SWWRF for treatment and disposal. As an effort to be proactive and to plan for future needs, the City desires to investigate the treatment capacity of the SWWRF and to determine possible hydraulic and process bottlenecks that could restrict the facility for handling the additional flows and pollutant loadings generated in the AWWRF's service area.

The scope of work presented for this study includes:

- Treatment process capacity evaluation plan,
- Hydraulic evaluation analysis,

- Sludge processing evaluation analysis.

1.2 Approach

The treatment process assessment of the SWWRF proceeded in several phases to provide a systematic approach to determine possible treatment limitations at the SWWRF. Unit process simulations were conducted to represent expected operating conditions. The plan was conducted collaboratively with City of St. Petersburg and the SWWRF staff. The following activities were conducted as part of the plan:

- Extensive analysis of historical operation data from 2007 through 2011
- Special sampling campaign to determine the wastewater characteristics
- Field tests to evaluate the impact of additional loads and solids that would be diverted to the SWWRF
- Evaluation of biological processes and development of simulation models
- Evaluation and stress testing of secondary clarifiers to determine process capacity
- Evaluation of existing sludge thickening, anaerobic digesters and sludge dewatering equipment
- Integration of unit process performance results to determine overall plant capacity
- Preparation of recommendations for plant upgrades and modifications

1.3 Objectives

The objectives of the treatment process assessment included the following:

1. Establish the maximum treatment capacity for the SWWRF to meet the existing effluent requirements including the flows and pollutant loadings from the AWWRF,
2. Determine the capacity beyond current flow and pollutant loadings to accommodate future conditions with combined influent flows and solid loadings.
3. Recommend improvements and/or operational adjustments for projected increases in flows and loadings to handle permitted flow at the SWWRF.

Section 2

Description of the SWWRF

This section provides an overview of the City's SWWRF, and permit requirements.

2.1 Plant Description

The SWWRF is located at 3800 54th Avenue South and serves the southwest section of the City of St. Petersburg, along with the Tierra Verde area of Pinellas County, a portion of the City of Gulfport, and Fort Desoto Park. Originally constructed in the 1950s as a primary treatment plant, the facility has undergone major expansions and upgrades over the last 60 years. Today, the SWWRF provides secondary treatment with effluent filtration. Final effluent is distributed as reclaimed water to a public access urban reuse irrigation system. Deep well injection has been also used as the backup effluent disposal method.

In general, the liquid treatment facilities at the SWWRF include influent screening; grit removal; activated sludge process, including mechanical aeration and secondary clarification; deep bed filtration; and gaseous chlorine disinfection. The "old plant" activated sludge treatment train is permitted for 4 MGD and the "new plant" is permitted for 16 MGD, accounting for a total annual daily flow of 20 MGD as annual average flow. Residuals treatment includes gravity belt thickening (GBT), anaerobic digestion, and dewatering by belt filter presses (BFP). Residuals are treated to Class B standards and land applied by a contract hauler.

Figure 2-1 and Figure 2-2 show the aerial view of SWWRF and a simplified process flow schematic for the existing treatment facility, respectively. Table 2-1 provides a summary of the design data for the major unit processes at the treatment facility.

The existing facilities are grouped into the following categories and discussed in detail in this section.

- Preliminary Treatment
- Secondary Treatment Facilities
- Filtration Facilities
- Disinfection Facilities
- Effluent Storage and Disposal Facilities
- Residuals Processing Facilities

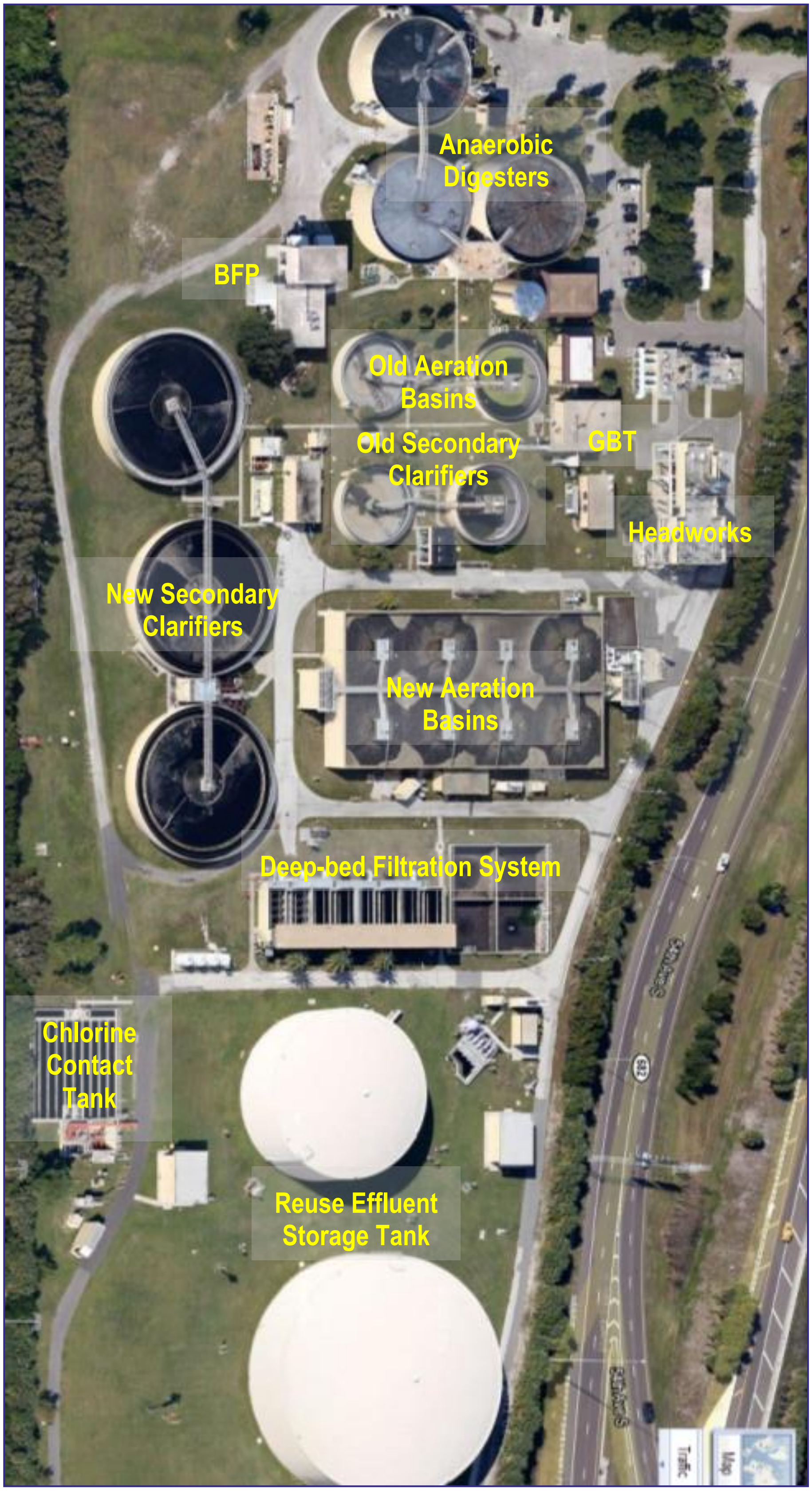


Figure 2-1. Aerial View of the SWWRF

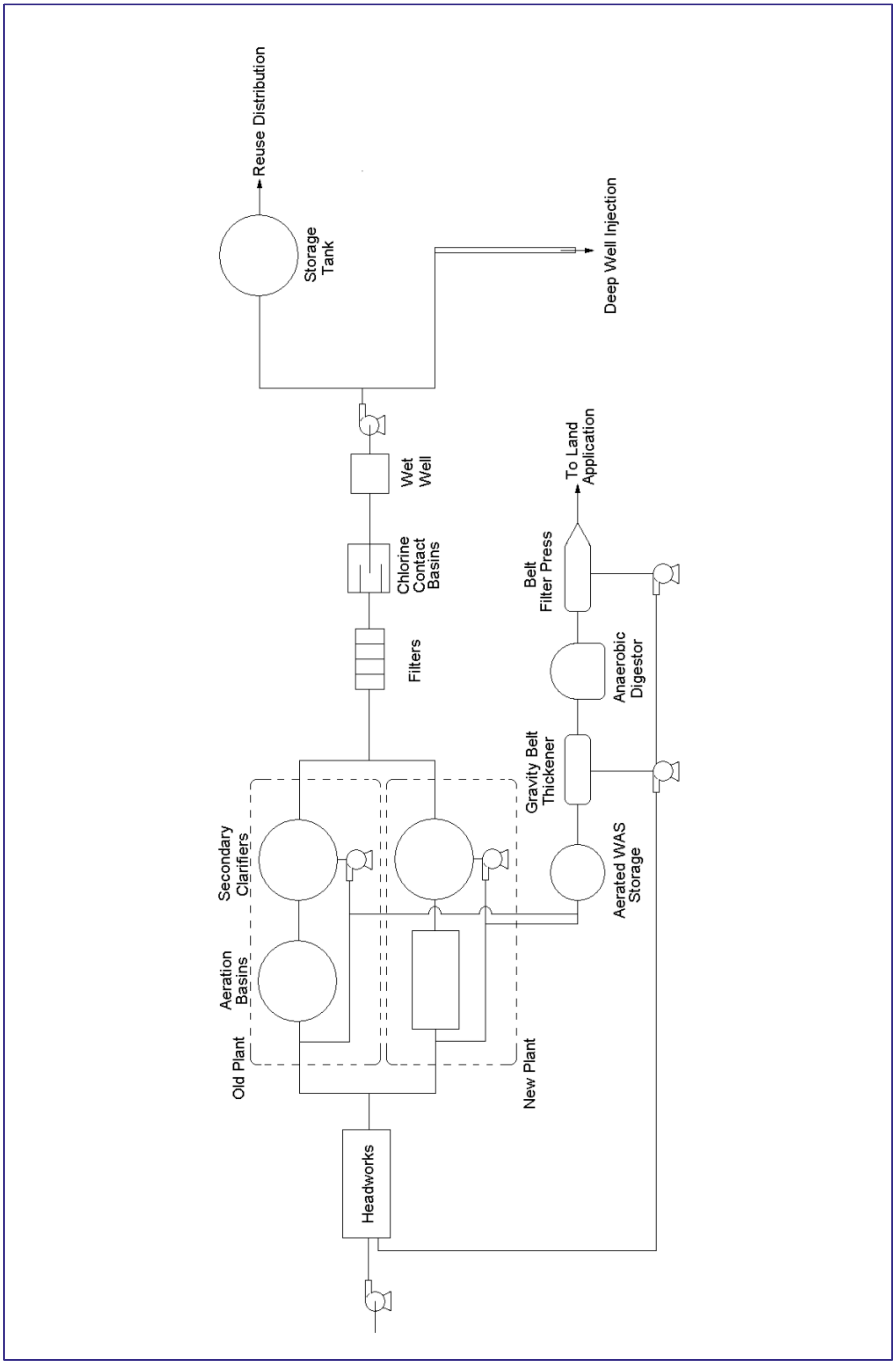


Figure 2-2. Process Flow Diagram of the SWWRF

Table 2-1. Summary of Existing Unit Processes at the SWWRF

	Parameter	Unit	Value	Notes
Preliminary Treatment				
Mechanical Screens	Coarse Screens	number	2	located in the influent wet well
	Fine Screens	number	2	located in the headworks structure
Influent Pumping	Submersible pumps			
Grit Removal	Grit King	number	2	
Secondary Treatment				
Aeration Basins				
Old Plant	Number of basins	--	2	Circular
	Dimensions (diameter x depth)	ft	65 x 13	
	Total Reactor Volume	MG	0.65	
New Plant	Number of basins	--	2	Rectangular
	Dimensions (L x W x D)	ft	268 x 67 x 15	
	Total Reactor Volume	MG	4.03	
Secondary Clarifiers				
Old Plant	Number	--	2	Circular
	Diameter	ft	65	
	Side Water Depth (SWD)	ft	13	
	Total Clarifier Volume	MG	0.64	
New Plant	Number	--	3	Circular
	Diameter	ft	135	
	Side Water Depth (SWD)	ft	12 (#1 &#2) – 15 (#3)	Old clarifiers and the new clarifier has different SWD
	Total Clarifier Volume	MG	3.85	
	RAS pump capacity	GPM	3 pumps at 4200	
Secondary Effluent				
Deep Bed Filtration	Number of Filters	--	4	
	Dimensions (L x W x D)	ft	38 x 37 x 9	
	Total Filter Area	ft^2	5,624	
Disinfection	Number of CCT's		2	Uses liquid sodium hypochlorite
	Dimensions (L x W x D)	ft	88 x 103 x 7	
	Total Volume	ft^3	126,896	
Reclaimed Water Storage	Number of Tanks	--	1	
	Volume	MG	5	
Solids Handling Processes				
Sludge Holding Tank	Number of Tanks	--	1	
	Volume	gal	110,000	
Gravity Belt Thickener	Number of Units	--	1	
	Size	m	2	Belt width
Anaerobic Digestion	Number of Tanks	--	3	2+1 (not currently operational)
	Diameter	ft	100	
	Side Water Depth	ft	22.5	At max level
	Volume	MG/each	1.3	
Belt Filter Press	Number of Units	--	2	
	Size	m	2	Belt width

2.1.1 Preliminary Treatment

The preliminary treatment facility at the SWWRF consists of mechanical bar screening, and grit removal units located in the headworks structure. The influent comes to the headwork structure directly from a 24-inch force main and a 54-inch gravity sewer line from the Southwest area of the city center. The headworks system includes two fine bar screens that remove debris and grit removal chambers that separate grit from organics. There is also an in-plant recycle pumping station that returns various side stream flows back to the head of the treatment plant. These side streams include GBT filtrate, BFP filtrate and filter backwash water.

Flow metering at this facility consists of five magnetic flow-meters that are capable of measuring the flow from, the old and the new influent pump stations, the recycle stations, and the forcemain.

2.1.2 Secondary Treatment Facilities

The secondary treatment process at the SWWRF comprises an “old plant” with a rated capacity of 4 MGD and a “new plant” with a rated capacity of 16 MGD.

The “old plant” comprises two circular aeration basins and two secondary settling tanks. The “old plant” has been out of service for several years. However, according to the City, these units can be brought online at any time if additional capacity is necessary.

The secondary treatment at the “new plant” consists of a conventional activated sludge process. The treatment includes two rectangular aeration basins with mechanical aerators for carbonaceous removal only and three secondary settling tanks. Currently, the City is replacing their old surface aeration system at the “new plant” with a new fine-bubble aeration system and aeration blowers. As part of the aeration project in the “new plant”, anaerobic selectors are being added within the aeration basins for process control, stability, and improved performance.

The wastewater is mixed, at the inlet of the aeration basins, with return activated sludge (RAS) from the secondary settling tanks. Each aeration basin contains four two-speed mechanical aerators. The mixed liquor from the aeration basins then flows into the secondary settling tank distribution structure by gravity.

The flow is then diverted to the online settling tanks for liquid-solids separation. The "new plant" treatment train includes three circular clarifiers. All three clarifiers are circular with diameters of approximately 135-feet. All three clarifiers are equipped with scum skimming equipment and served by a common RAS pump system. A portion of the settled solids are wasted from the system to control the biomass inventory in the aeration basins.

2.1.3 Filtration Facilities

The filtration facilities include four multi-media filters, and a backwash system that includes a sweep system and a backwash water holding basin. The filters are made of Leopold compound, duplex-tile filter blocks covered with multiple layers of media.

From the clarifiers, the effluent water flows by gravity to the deep bed multimedia filters. This filter system is comprised of sand, gravel and anthracite. The multiple layers and irregular shaped media allow for the effective removal of fine suspended matter. The water settles through the filter bed and then flows into the Chlorine Contact Basin.

2.1.4 Disinfection System

Disinfection facilities at the plant include sodium hypochlorite storage, chlorine feed equipment, and a chlorine contact basin. The chlorination system is capable of discharging upstream of the filters and/or into the mixing boxes of the chlorine contact chamber.

Filtered effluent is conveyed to the chlorine contact tanks for final disinfection with liquid sodium hypochlorite before being stored in the reuse tanks located on the property.

2.1.5 Effluent Storage and Disposal Facilities

The SWWRF's effluent storage comprises two reclaimed water storage tanks. The final effluent is stored in the two above ground storage tanks to be available for use in the City's public access urban reuse program. If the effluent water does not meet the mandated water quality standards then it is sent to deep injection wells. These wells also take the excess reclaimed water in times of decreased demand.

2.1.6 Solids Processing Facilities

Waste activated sludge (WAS) is pumped to a holding tank before feeding them regularly to a GBT with polymer addition. The sludge holding tank is equipped with an aeration system to ensure good mixing of the sludge. The GBT facility is a three-sided, roofed structure that contains one GBT and has space for a future unit. A polymer feed system is used to enhance the thickening process.

Thickened WAS is then pumped to the anaerobic digesters which are operated at mesophilic temperatures (95-102 °F). The solids are retained in two of the three digesters for about 30-45 days for stabilization, breakdown of the organics and pathogen destruction.

There are currently three circular anaerobic digesters equipped with heaters. Digesters #1 and #3 are currently in use. All of the three digesters are approximately 25 feet tall, 100 feet diameter.

Anaerobically digested sludge is routed to BFP. The BFP facility consists of a filter processing room, an associated electrical equipment room, and a truck loading area. The facility has flow meters that measure the amount of flow to the two BFPs.

Finally, the dewatered biosolids, which generally has 13-15 percent solids, is hauled off-site for land application as Class B biosolids.

2.2 Permit Requirements

The SWWRF is currently permitted for a rated capacity of 20 MGD as AADF. Operation of the Southwest's treatment and disposal facilities is subject to state and federal regulations stated in Florida Department of Environmental Protection (FDEP) Operating permit FLA 128848-01 which expired in June 2005. The City is currently in the process of renewing their operating permit. However, by the time this study was completed a new copy of the permit had not been received. No changes to the permit are expected. Table 2-2 summarizes the effluent quality standards for reuse and land application system as stated in the existing operating permit.

Table 2-2. SWWRF Effluent Quality Standards for Reuse and Land Application

Parameter	Unit	Limit
Flow	MGD	20 – Annual Average
Carbonaceous Biochemical Oxygen Demand – 5 days ($CBOD_5$)	mg/L	20 – Annual Average
		30 – Monthly Average
		45 – Weekly Average
		60 – any one sample
Total Suspended Solids (TSS)	mg/L	5 – any one sample
Fecal Coliform	#/100 mL	25
pH	SU	6.0 - 8.5
Nitrate (NO_3-N)	mg/L	12
Total Residual Chlorine	mg/L	1 – State Minimum
		4 – Operating Protocol Minimum
Turbidity	NTU	5 - Operating Protocol Maximum

At the SWWRF, reclaimed water which exceeds the reuse demand, or does not meet the treatment requirements of the reclaimed system, is disposed through three existing Class I injection wells located at the SWWRF site. The wells have a combined permitted disposal capacity of 27 MGD and are currently being operated under the temporary authorization of FDEP Consent Order 92-0092. Table 2-3 summarizes the effluent quality standards for deep well injection.

Table 2-3. SWWRF Effluent Quality Standards for Deep Well Injection

Parameter	Unit	Limit
Flow	MGD	27
Carbonaceous Biochemical Oxygen Demand – 5 days ($CBOD_5$)	mg/L	20 – Annual Average
		30- Monthly Average
		45-Weekly Average
		60-any one sample
Total Suspended Solids (TSS)	mg/L	20 – Annual Average
		30- Monthly Average
		45-Weekly Average
		60-any one sample
pH	SU	6.0-8.5

Section 3

Influent Flow and Pollutant Loadings

This section summarizes the historical flows and pollutant loadings to the SWWRF, and the combined anticipated flows and loadings to the facility when the Albert Whitted WRF (AWWRF) discontinues operation. The City of St. Petersburg is currently planning on consolidating their water reclamation facilities by converting the AWWRF to a pump station to divert its wastewater to the SWWRF for treatment. Daily average historical influent and effluent data from January 2007 through November 2011 were obtained for this analysis.

Appendix A presents historical data for the SWWRF on influent and effluent flow, $CBOD_5$, TSS, NH_3-N, TKN, and TP. In addition, operational data such as mixed liquor TSS and VSS and SVI, RAS concentration, WAS flow rate, thickening and dewatering data are also available in Appendix A.

3.1 Historical Influent Flows and Pollutant Loads to the SWWRF

The SWWRF is permitted as a 20 mgd Type I activated sludge facility. The design parameters used as the basis of the permitted capacity are summarized in Table 3-1.

Table 3-1. SWWRF Original Basis of Design[1]

Parameter	Annual Average
Flow (MGD)	20
$CBOD_5$ (lb/d – mg/L)	33,360 – 200
TSS (lb/d – mg/L)	36,696 – 220

[1.] *Capacity Analysis Report for the City of St. Petersburg SWWRF, April 1998*

Table 3-2. Influent Basis of Design for the SWWRF as presented in the 2004 Master Plan

Parameter	Annual Average	Maximum Month
Flow (MGD)	20	--
$CBOD_5$ (lb/d – mg/L)	17,392 – 104	24,846 – 149
TSS (lb/d – mg/L)	21,119 – 127	32,300 – 194
TKN (lb/d – mg/L)	4,472 – 27	5,839 – 35

For this analysis, flows and pollutant loadings were determined using daily plant operating data from January 2007 through November 2011. It should be noted that pollutant concentrations from September 01, 2007 through December 31, 2007 and from February 01, 2008 through May 31, 2008 were not provided by the City. Figures 3-1 through 3-5 present several graphs showing the monthly average influent flows and pollutant loadings for the period of 2007-2011. Pollutant loadings are plotted for $CBOD_5$, TSS, TKN, NH_3-N and TP.

Figure 3-1 shows that the historical monthly average flow rates have averaged approximately 9.4 MGD over the last 5 year. During the period analyzed, the maximum month average flow (MMF) is approximately 14 MGD. The maximum day flow during the 5-year period was approximately 20 MGD, and occurred in September 2011. As Figure 3-1 indicates, influent flows to the SWWRF have steadily increased by 28 percent during the period of 2007 to 2011.

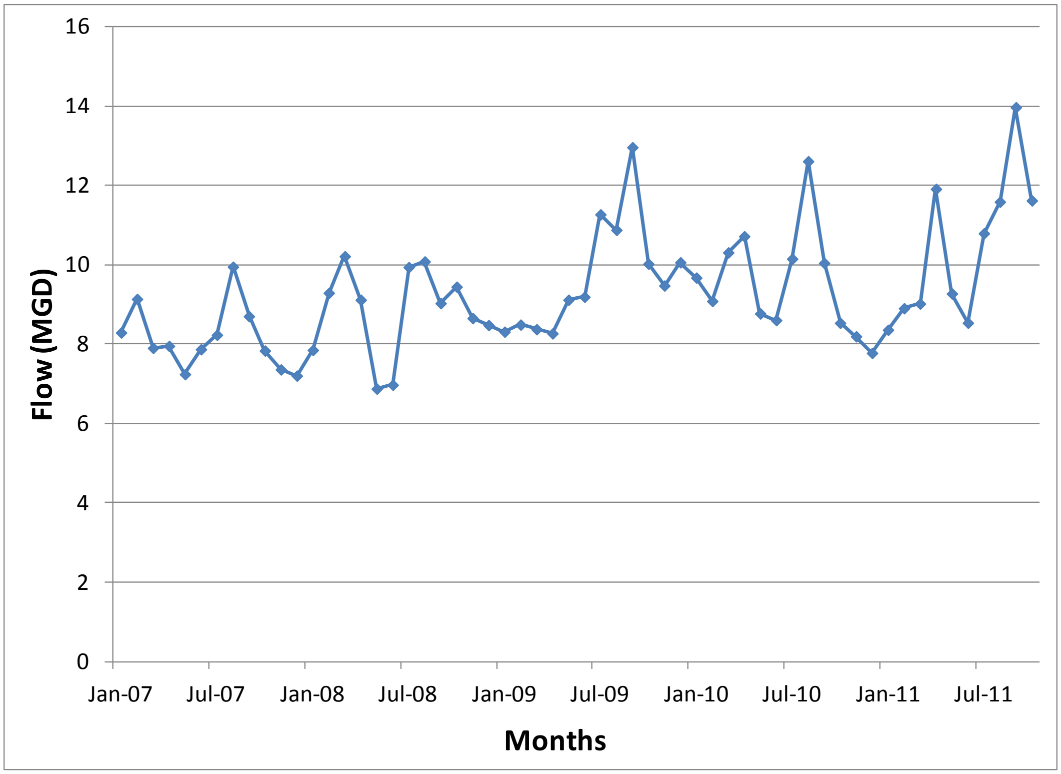


Figure 3-1. Monthly Average Flows to the SWWRF

Historical $CBOD_5$ and TSS loads to the SWWRF have shown significant variability over the last 5 years. After careful review of the data, periods of high $CBOD_5$ and TSS concentrations were observed at the facility; mainly because of operational issues associated with solids being sent to the head of the plant. In addition, after discussion with plant staff, it is believed that some of the high concentrations recorded in the influent might be attributed to solids accumulation in the vertical sampling lines located at the headworks. Due to the inconsistent occurrence of high $CBOD_5$ and TSS concentrations, it was decided to exclude $CBOD_5$ and TSS concentrations higher than the 90 percentile value from the 2007-2011 dataset. This approach eliminated outliers that could artificially increase the loadings to the plant. Figure 3-2 presents the historical influent $CBOD_5$ concentrations reported by the City (recorded $CBOD_5$) as well as the $CBOD_5$ data used for this analysis (analyzed $CBOD_5$). A similar approach was used to screen the influent TSS concentrations.

Figure 3-3 shows the historical influent $CBOD_5$ loads to the SWWRF. Based on this data, the monthly average influent $CBOD_5$ load for the last 5 years is approximately 14,782 lb/d with a maximum monthly average of 19,966 lb/d. Figure 3-4 shows that the monthly average influent TSS load, for the 2007-2011 period, is approximately 18,067 lb/d with a maximum monthly average of 29,160 lb/d.

Limited influent nutrient information has been historically collected at the SWWRF since no requirements to comply with either nitrogen or phosphorus species are currently included in the permit. Figure 3-5 shows the historical influent NH_3-N, TKN and TP loads to the SWWRF. The historical monthly average influent nutrient loads are approximately 1,788 lb/d, 3,391 lb/d, and 540 lb/d for NH_3-N, TKN and TP, respectively.

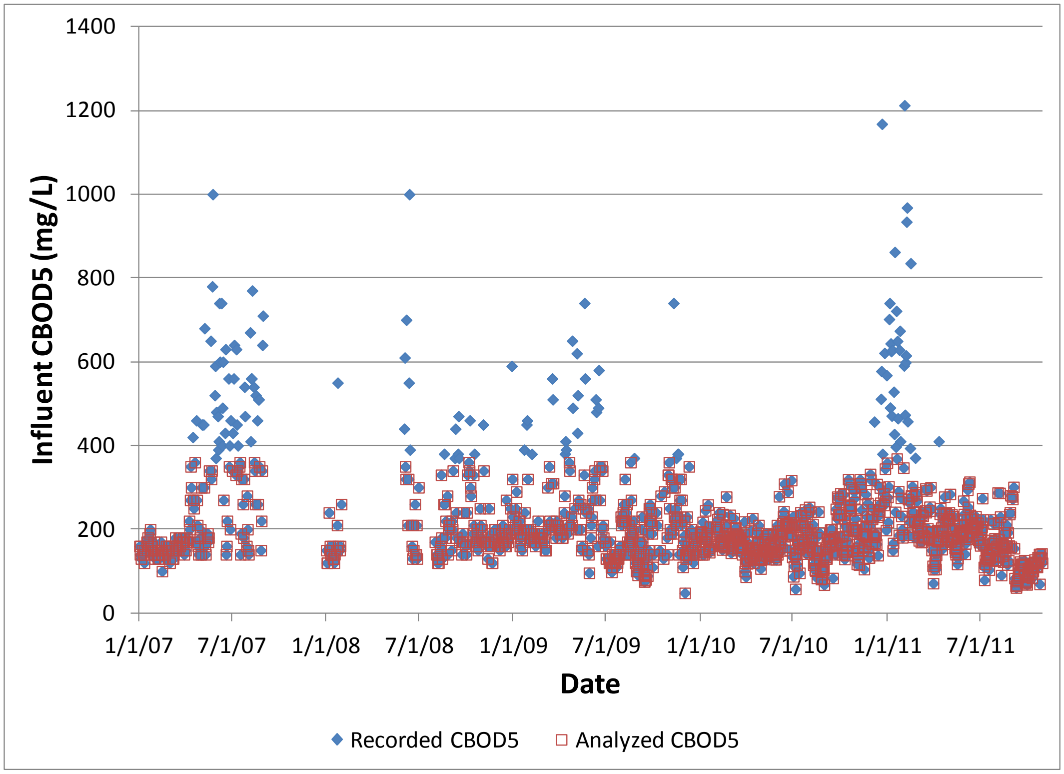


Figure 3-2. Historical Influent $CBOD_5$ Concentrations Reported by the City and the less-than-90-percentile $CBOD_5$ Concentrations used for this Analysis

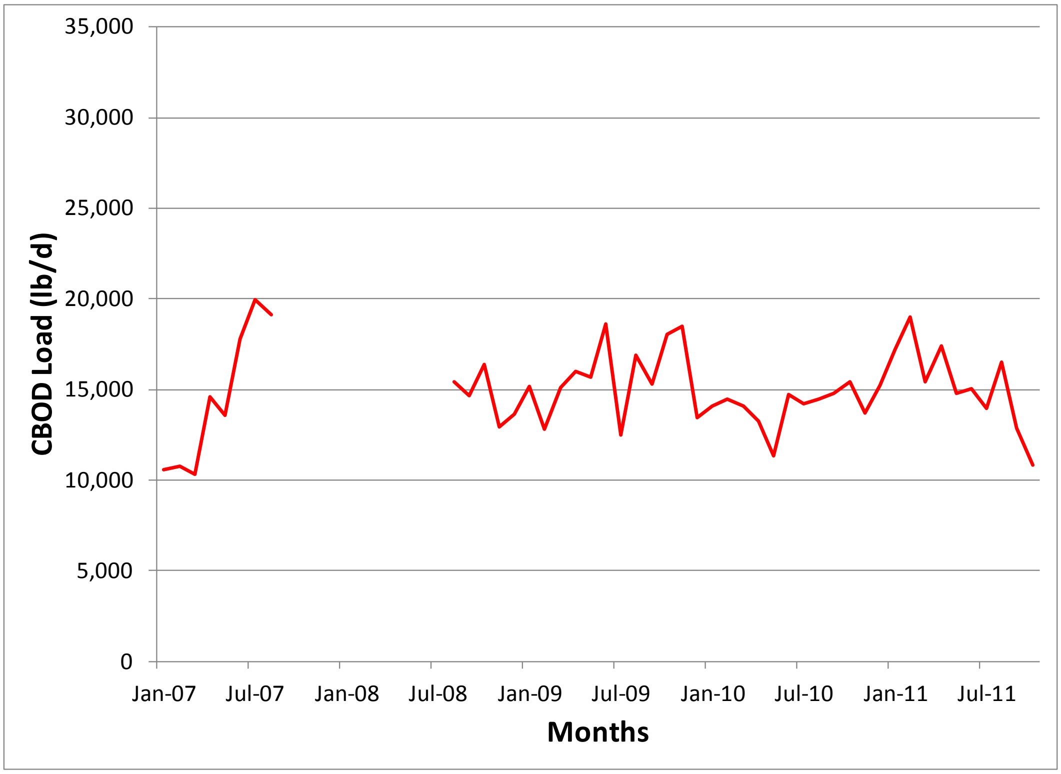


Figure 3-3. Monthly Average $CBOD_5$ Load to the SWWRF

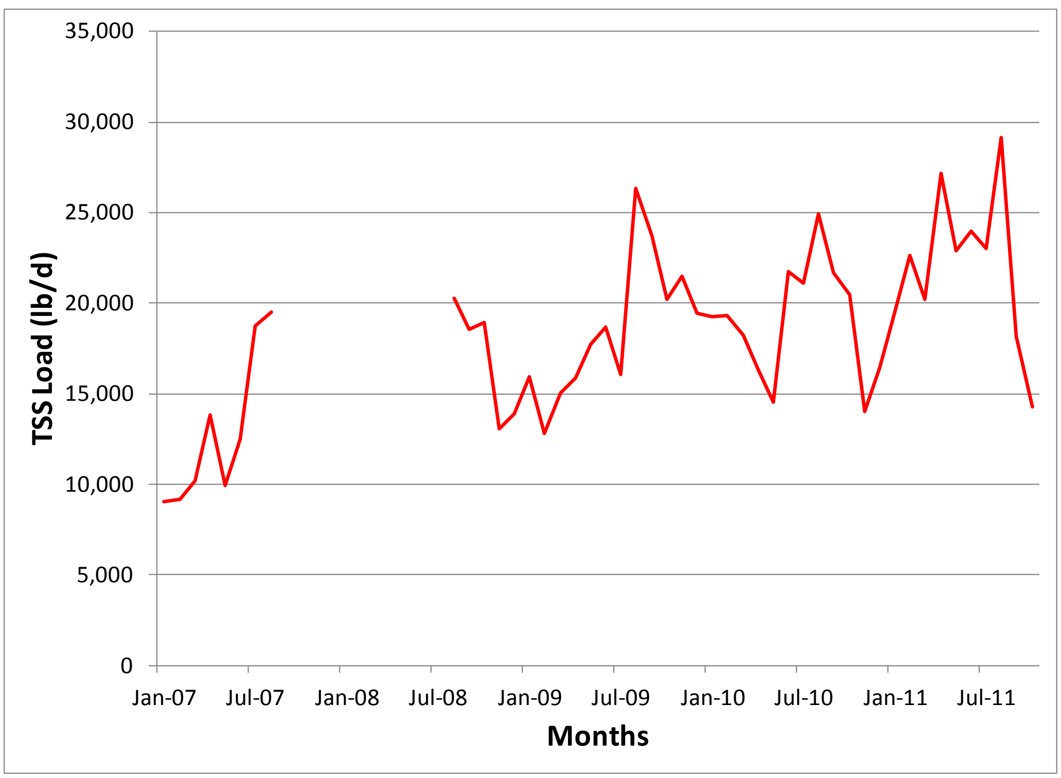


Figure 3-4. Monthly Average TSS Load to the SWWRF

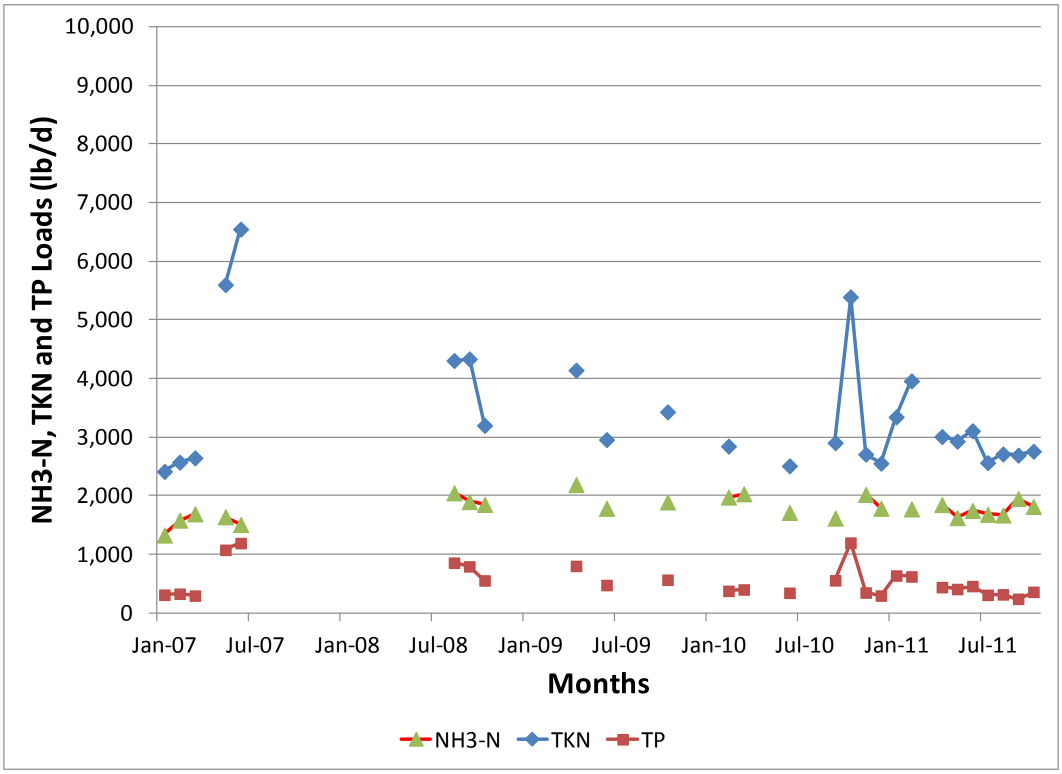


Figure 3-5. Monthly Average Ammonia, TKN and Total Phosphorus Loads to the SWWRF

Based on the historical influent information from January 2007 through November 2011, the monthly average and maximum month $CBOD_5$, and monthly average and maximum month TSS loadings are approximately 44 percent and 46 percent and 54 percent and 66 percent, respectively, of the plant's original design loadings (Table 3-1). The historical monthly average $CBOD_5$, TSS, and TKN loadings from January 2007 through November 2011 are approximately 15 percent, 14 percent and 24 percent higher than the values adopted in the 2004 Master Plan (Table 3-2). This is an indication that the influent concentrations observed at the SWWRF are considerably higher than the values presented in the 2004 Master Plan.

Table 3-2. Summary of Historical Flow and Pollutant Loads to the SWWRF – 2007 through 2011

Parameter	Condition	2007	2008	2009	2010	2011
Flow	Annual Average	8.12	8.83	9.71	9.55	10.40
	Maximum Month	9.95	10.22	12.97	12.61	13.97
	Maximum Day	13.72	12.43	17.23	17.63	19.57
$CBOD_5$	Annual Average	13,737	13,855	15,517	14,135	15,066
	Maximum Month	19,966	16,389	18,613	15,412	18,981
	Maximum Day	27,004	26,456	26,201	26,103	26,968
TSS	Annual Average	10,964	15,114	18,294	18,848	21,836
	Maximum Month	19,507	20,268	26,316	24,912	29,160
	Maximum Day	35,895	33,925	49,810	39,472	43,810
TKN	Annual Average	3,957	3,947	3,511	3,153	3,009

	Maximum Month	6,550	4,334	4,143	5,394	3,959
	Maximum Day	--	--	--	--	--
TP	Annual Average	640	735	614	502	421
	Maximum Month	1,192	857	801	1,199	637
	Maximum Day	--	--	--	--	--

Table 3-3. Summary of Historical Peaking Factors for Flow and Pollutant Loads to the SWWRF – 2007 through 2011

Parameter	Condition	2007	2008	2009	2010	2011	Average	Maximum
Flow	Maximum Month	1.23	1.16	1.34	1.32	1.34	1.28	1.34
	Maximum Day	1.69	1.41	1.77	1.85	1.88	1.72	1.88
$CBOD_5$	Maximum Month	1.45	1.18	1.20	1.09	1.26	1.24	1.45
	Maximum Day	1.97	1.91	1.69	1.85	1.79	1.84	1.97
TSS	Maximum Month	1.78	1.34	1.44	1.32	1.34	1.44	1.78
	Maximum Day	3.27	2.24	2.72	2.09	2.01	2.47	3.27
TKN	Maximum Month	1.66	1.10	1.18	1.71	1.32	1.39	1.71
	Maximum Day	--	--	--	--	--	--	--
TP	Maximum Month	1.86	1.17	1.31	2.39	1.51	1.65	2.39
	Maximum Day	--	--	--	--	--	--	--

Historical average influent ratios for $CBOD_5$:TSS, $CBOD_5$:TKN, NH_3-N:TKN, TP:$CBOD_5$ and PO4-P:TP were estimated with the following results: $CBOD_5$:TSS of 0.82; $CBOD_5$:TKN of 4.1; NH_3-N:TKN of 0.53; TP:$CBOD_5$ of 0.036; and PO_4-P:TPof 0.35.

Figure 3-6 presents the historical temperature data, which indicates that the average daily temperature at the SWWRF is approximately 77°F, with maximum day value 83 °F and minimum day value of 67°F daily values. The maximum and minimum month temperatures depicted in Figure 2-7 are approximately 82°F and 72°F, respectively.

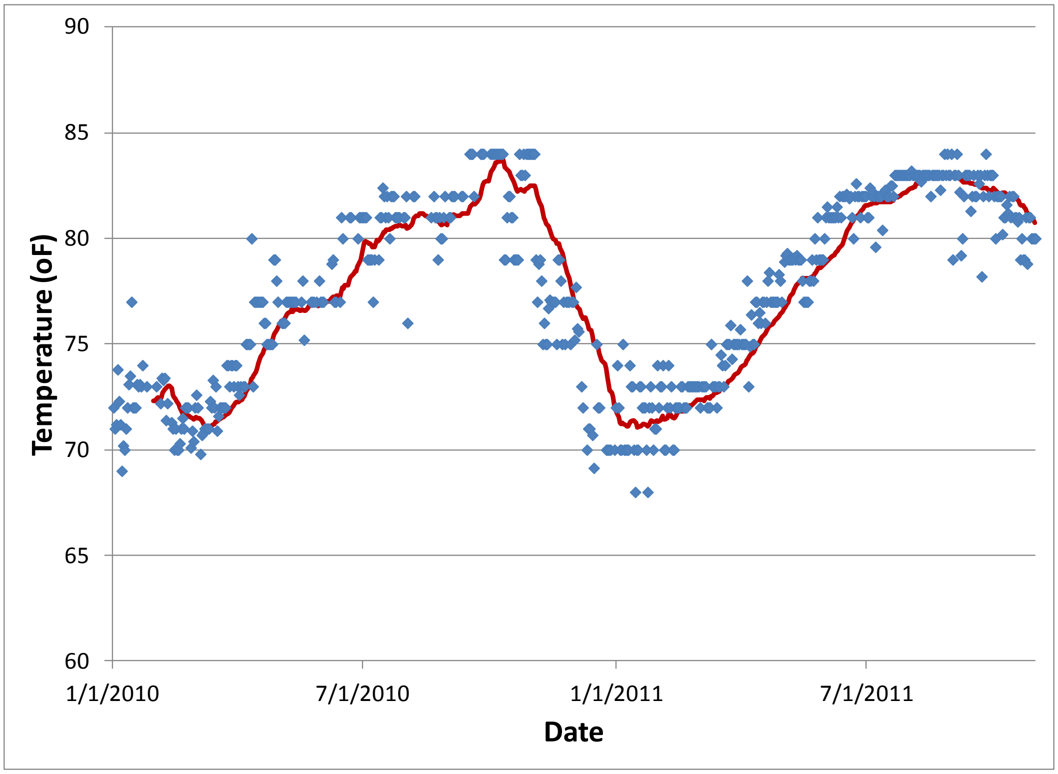


Figure 3-6. Daily Variation of Influent Temperature at the SWWRF

A summary of the plant data flows and pollutant loadings for annual average, maximum month and maximum day conditions are listed in Table 3-2. This information was used to determine flow and load peaking factors as presented in Table 3-3. The peaking factors for any given year represent the ratio of the highest peak condition to the annual average for a given year. Based on the data presented in Table 3-3, the following can be concluded:

- The average and maximum-month-to-annual-average flow peaking factors are 1.28 and 1.34. The maximum peaking factor corresponded to 2011.
- The average and maximum-day-to-annual-average flow peaking factors are1.72 and 1.88. The maximum peaking factor corresponded to 2011.
- The average and maximum-month-to-annual-average $CBOD_5$ load peaking factor are 1.24 and 1.45. The maximum peaking factor corresponded to 2007.
- The average and maximum-day-to-annual-average $CBOD_5$ load peaking factor are 1.84 and 1.97. The maximum peaking factor corresponded to 2007.
- The average and maximum-month-to-annual-average TSS load peaking factor are 1.44 and 1.78. The maximum peaking factor corresponded to 2007.
- The average and maximum-day-to-annual-average TSS load peaking factor are 2.47 and 3.27. The maximum peaking factor corresponded to 2007.
- The average and maximum-month-to-annual-average TKN load peaking factor are 1.39 and 1.71. The maximum peaking factor corresponded to 2010.
- The average and maximum-month-to-annual-average TP load peaking factor are 1.65 and 2.39. The maximum peaking factor corresponded to 2010.

3.2 Combined AWWRF and SWWRF Influent Flows and Pollutant Loads

Closing the Albert Whitted WRF is currently under consideration. If this plant is closed, the SWWRF would take all of the flow from the AWWRF service area. The plan is to convert the AWWRF to a pump station and to divert all of its wastewater to the SWWRF for treatment and disposal. Therefore, in order to properly determine the possible impacts of this on the capacity of the SWWRF, influent flows and pollutant loading information from the AWWRF were combined to the data presented in Section 3.1 from the SWWRF. This approach is used to estimate the possible future flows and loadings to the SWWRF.

Figure 3-7 and Figure 3-8 present the monthly average combined flows and pollutant loadings. By adding the flows from the AWWRF and SWWRF, the average flow from 2007 through 2011 is approximately 15.5 MGD, which is 65 percent higher than the monthly average flow for SWWRF only. The maximum month and maximum day average flows from the combined system are 22.96 MGD and 32.93 MGD, respectively, and they were recorded in 2011. The combined monthly average flow represents approximately 77.5 percent of the plant's rated capacity.

The combined $CBOD_5$ and TSS loading values are presented in Figure 3-8. Based on this data, the monthly average and maximum month influent $CBOD_5$ loads for the last 5 years are approximately 23,281 lb/d and 30,851 lb/d. The monthly average influent TSS load for the 2007-2011 period is approximately 28,079 lb/d with a maximum monthly average of 46,609 lb/d. It should be noted the monthly average and maximum month $CBOD_5$ and monthly average and maximum month TSS loadings of the combined influents are approximately 70 percent, 76 percent, 85 percent, and 113 percent of the plant's design loading conditions presented in Table3-1. The combined influent $CBOD_5$-to-TSS ratio is approximately 0.83, which is consistent with the influent ratio for the SWWRF only influent.

Similarly to Section 3.1, a summary of the combined plant data flows and pollutant loadings for annual average, maximum month and maximum day conditions are summarized in Table 3-4. This information was used to determine flow and load peaking factors and they are presented in Table 3-5. Based on the data, the following can be concluded:

- The average and maximum month-to-annual-average flow peaking factors are 1.32 and 1.39. These values are higher than the SWWRF influent-only condition.
- The average and maximum day-to-annual-average flow peaking factors are1.84 and 1.99. These values are higher than the SWWRF influent-only condition.
- The average and maximum month-to-annual-average $CBOD_5$ load peaking factor are 1.22 and 1.32. These values are lower than the SWWRF influent only condition, which indicates that the wastewater from AWWRF is less strong than that from SWWRF.
- The average and maximum day-to-annual-average $CBOD_5$ load peaking factor are 1.80 and 2.07, which are similar to SWWRF influent only condition.
- The average and maximum month-to-annual-average TSS load peaking factor are 1.51 and 1.90. These values are higher than the SWWRF influent only condition, which indicates that the TSS concentrations in the wastewater from AWWRF are higher than those from SWWRF.
- The average and maximum day-to-annual-average TSS load peaking factor are 2.25 and 2.53.
- Limited influent nutrient information was available for AWWRF.

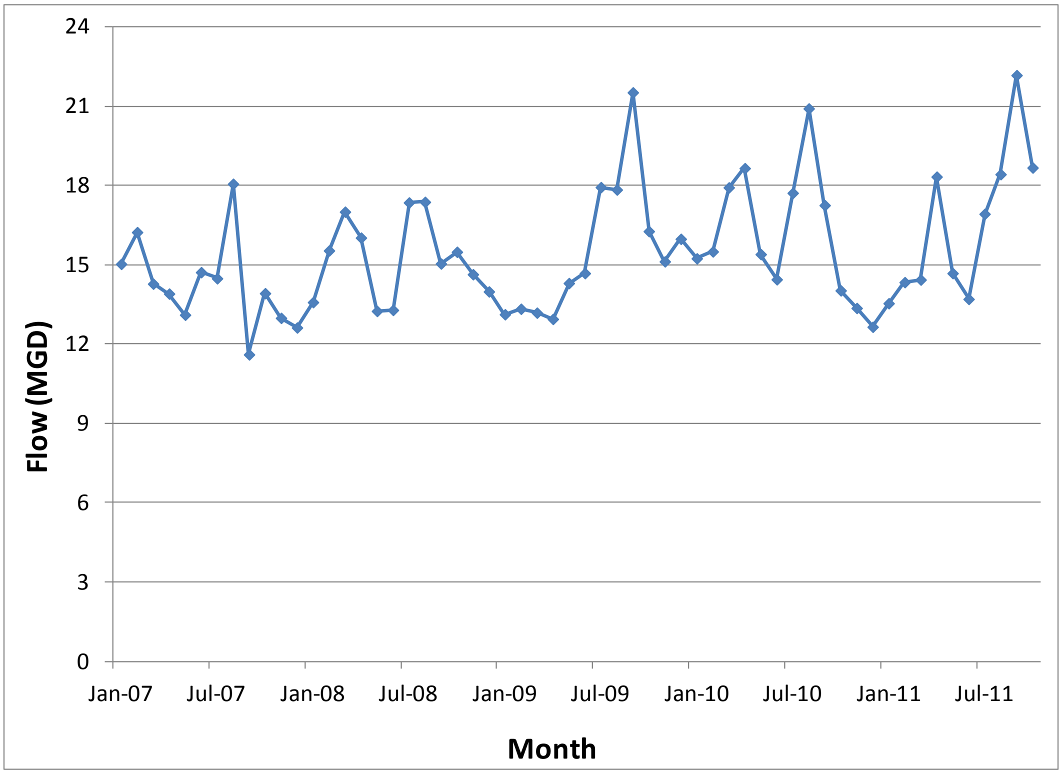


Figure 3-7. Combined AWWTF and SWWRF Monthly Average Flows

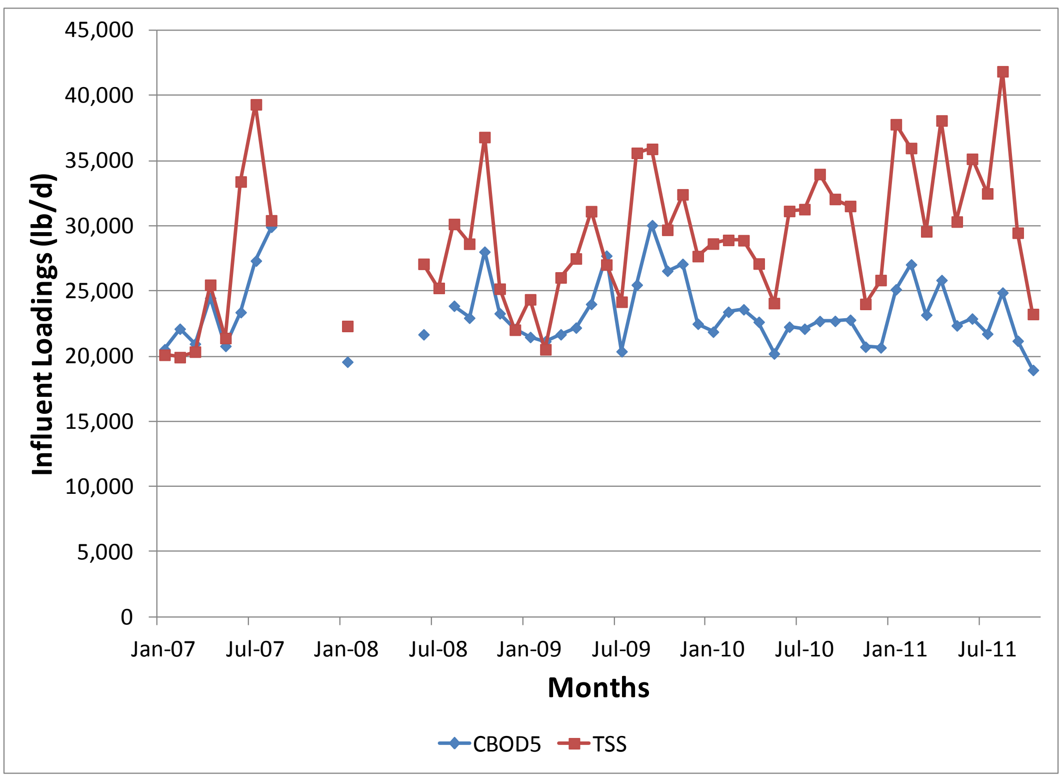


Figure 3-8. Combined AWWTF and SWWRF Monthly Average $CBOD_5$ and TSS Loads

Table 3-4. Summary of Historical Flow and Pollutant Loads for the Combined Influent of AWWRF and the SWWRF – January 2007 through November 2011

Parameter	Condition	2007	2008	2009	2010	2011
Flow	Annual Average	14.25	15.22	15.54	16.10	16.52
	Maximum Month	18.22	18.63	21.67	21.35	22.96
	Maximum Day	27.37	22.42	29.12	30.98	32.93
$CBOD_5$	Annual Average	23,327	23,112	24,833	22,239	22,894
	Maximum Month	30,851	29,025	30,627	24,628	27,146
	Maximum Day	38,110	40,453	51,305	35,113	45,066
TSS	Annual Average	23,281	26,581	28,697	29,127	32,710
	Maximum Month	44,341	46,609	37,415	35,704	43,852
	Maximum Day	49,462	64,221	61,027	59,895	82,649

Table 3-5. Summary of Historical Peaking Factors for Flow and Pollutant Loads for the Combined Influent of AWWRF and the SWWRF – January 2007 through November 2011

Parameter	Condition	2007	2008	2009	2010	2011	Average	Maximum
Flow	Maximum Month	1.28	1.22	1.39	1.33	1.39	1.32	1.39
	Maximum Day	1.92	1.47	1.87	1.92	1.99	1.85	1.99
$CBOD_5$	Maximum Month	1.32	1.26	1.23	1.11	1.19	1.22	1.32
	Maximum Day	1.63	1.75	2.07	1.58	1.97	1.80	2.07
TSS	Maximum Month	1.90	1.75	1.30	1.23	1.34	1.51	1.90
	Maximum Day	1.12	1.38	1.63	1.68	1.88	1.54	1.88

3.3 Flow and Pollutant Loading Peaking Factors

For the purpose of this analysis, emphasis was given, for the selection of the design conditions, to CBOD5 and TSS loadings, as they have the strongest influence on plant operations. Once maximum months were selected, all values reported for these chosen months were used to define the pollutant loadings on the plant. Therefore, the defined maximum month loadings represent actual conditions experienced rather than inflated conditions based on selecting the highest load for each characteristic. Studies have shown that selecting the highest load for each characteristic throughout the data set results in over-conservative values which could lead to overdesign of facilities or under estimation of their capacities.

For this analysis, both the SWWRF influent-only and the combined AWWRF-SWWRF influent data were used to establish important peaking factors to be used to define the flows and pollutant loadings for the treatment process assessment of the SWWRF. The maximum month chosen was September 2009 and corresponded to the combined AWWRF-SWWRF data. This month represents a month where $CBOD_5$ and TSS loadings are at or near the highest levels above its average daily load. The following list summarizes the peaking factors selected for this analysis:

- Maximum-month-to-annual-average flow factor of 1.35.
- Maximum-day-to-annual-average flow factor of 1.85.
- Maximum-month-to-annual-average loading factor of 1.30.
- Maximum-day-to-annual-average load factor of 1.85.

For the purpose of this analysis, the annual average $CBOD_5$ and TSS concentrations adopted were 215 mg/L and 240 mg/L, respectively. These concentrations are compatible with values used during the original design; however, they are almost twice as high as those values adopted during the Master Plan.

Section 5 provides detailed information on the influent basis of design for flow and pollutant loadings selected for this evaluation.

Section 4

Wastewater Characterization, Process Simulator Calibration and Validation

There are essentially two steps to calibrating a process simulator: (1) wastewater characterization and (2) model verification. This section describes the wastewater characterization study conducted at the SWWRF and AWWRF and the process simulator calibration under existing operating conditions.

4.1 Wastewater Characterization Study

Process simulation modeling requires accurate characterization of the different carbon, nitrogen and phosphorus fractions in the wastewater. To determine the specific wastewater characteristics at the SWWRF for possible future conditions, two special sampling campaigns were conducted during the August-November 2011 period. All samples were collected by the City staff and analyzed by the City's water quality laboratory. Appendices B and C present the special sampling plan as well as details of the special sampling data collected during this study.

4.1.1 Composite Sampling

Daily (24-hour) composite samplings were performed over a three-week period at the SWWRF and AWWRF. Five days of composite sampling for influent and effluent were collected at the SWWRF during September 26, 2011 through September 30, 2011. For AWWRF, five days of influent composite samples were collected during September 12 through September 16, 2011.

Table 4-1 presents the average parameters for the influent and effluent daily composite samples collected at the SWWRF. In the case of the SWWRF, both influent streams were flow weighted and combined into average influent concentrations. Table 4-2 presents average parameter values for the influent daily composite samples collected at the AWWRF. Based on these results, important influent wastewater fractions were determined and are presented in Table 4-3. These results allow an interpretation of the most consistent ratios and thereby, the most reliable parameter values from the data collected during the special sampling campaign.

4.1.1.1 COD Fraction

The daily composite COD and $CBOD_5$ concentrations recorded during the sampling campaign were used to determine the influent COD-to-$CBOD_5$ ratio. Based on the results presented in Table 4-3, the average influent COD-to-$CBOD_5$ ratios for the AWWRF and SWWRF are approximately 3.33 and 5.34, respectively. These values are higher than those fractions often found at other facilities, especially the values observed at the SWWRF. The influent average COD-to-TSS ratios for the AWWRF and SWWRF are 2.44 and 2.05, respectively. These are consistent with COD-to-TSS fractions found at other facilities. Therefore, it is reasonable to assume that the recorded influent $CBOD_5$ concentrations, especially at the SWWRF are lower than the actual values. For the purpose of this analysis and for the model calibration, the influent characteristics were based on the composite COD and TSS data collected during the special sampling campaign.

Based on the data presented in Table 4-3, it can be estimated that the influent COD concentrations for AWWRF and SWWRF comprise 69 and 73 percent particulate COD and 31 and 27 percent soluble COD.

Influent Readily Biodegradable COD Fraction (Fbs)

This fraction was calculated based on the following approach:

Fbs = [CODxf, Inf – CODxf, effl] / CODmx, Inf

Where:

CODxf, Inf = influent COD concentration of flocculated, filtered wastewater (mg/L)

CODxf, effl = effluent COD concentration of flocculated, filtered wastewater (mg/L)

CODmx, Inf = influent total COD concentration (mg/L)

The average Fbs based on the data collected for AWWRF and SWWRF are 0.15 and 0.14.

Influent Unbiodegradable COD Fraction (Fus)

This fraction was calculated using the following approach:

Fus = CODxf, effl / CODmx, Inf

The average Fus for AWWRF and SWWRF are 0.10 and 0.12.

Other influent COD fractions were calculated using the BioWin Influent Specifier model, which is COD/VSS mass balance model for COD wastewater fractions. Appendix D presents summary results of the COD fractions determined using the BioWin Influent Specifier. The bases for calculating other influent wastewater fractions are summarized below in Table 4-4.

4.1.1.2 Nitrogen Fraction

The influent daily composite NH_3-N and TKN concentrations recorded for AWWRF and SWWRF indicate average NH_3-N-to-TKN ratio of approximately 0.64 and 0.69, respectively.

The influent COD-to-TKN and COD-to-NH_3-N ratios from the AWWRF and SWWRF were derived from the composite daily samples with average values of 12.2 and 19 and 18.3 and 26.6, respectively. These values match values often found at other municipal treatment facilities.

4.1.1.3 Phosphorus Fraction

The influent daily composite TP and PO_4-P concentrations recorded for AWWRF and SWWRF during the special sampling indicate PO_4-P-to-TP (Fpo4) average ratio of 0.63 and 0.47, which are consistent with values often found elsewhere.

The influent TP-to-COD and TP-to-TN ratios from both facilities were derived from the composite daily samples with average values of 0.02 and 0.20 and 0.01 and 0.13 for AWWRF and SWWRF, respectively.

4.1.1.4 Solids Fraction

The average volatile solids (VS) fraction of the influent total suspended solids (TSS) calculated during the special sampling campaigns are 90 and 83 percent for AWWRF and SWWRF, respectively. Based on the special sampling campaign, influent VS-to-COD and ISS-to-COD ratios were calculated with average values of 0.38 and 0.04 and 0.40 and 0.08 for AWWRF and SWWRF, respectively.

Table 4-1. SWWRF Influent and Effluent Average Composite Results		
Parameters	Influent	Effluent
Alkalinity (mg/L as $CaCO_3$)	217	213.20
CBOD-5 day (mg/L)	77	2.36
COD (mg/L)	348	48.42
Floc Filtered COD (mg/L)	89.26	39.04
Soluble COD (0.45 micron) (mg/L)	104.36	49.68
Ammonia-N (mg/L)	12.91	10.12
Total Kjeldahl Nitrogen (mg/L)	18.67	11.01
Soluble TKN (0.45 micron) (mg/L)	13.81	10.48
Soluble TKN (GF) (mg/L)	14.34	10.86
Soluble Nitrate+Nitrite-N (0.45 micron) (mg/L)	--	1.02
Soluble Nitrate-N (0.45 micron) (mg/L)	--	0.25
Soluble Nitrite-N (0.45 micron) (mg/L)	--	0.77
Soluble Orthophosphate (0.45 micron) (mg/L)	1.12	0.56
Total Phosphorous (mg/L)	2.44	0.63
Soluble Total Phosphorus (0.45 micron) (mg/L)	1.17	0.61
Soluble Total Phosphorus (GF) (mg/L)	1.21	0.64
TSS (mg/L)	170	0.85
Field pH (SU)	7.41	7.42

Table 4-2. AWWRF Influent Average Composite Results	
Parameters	AWWRF
Alkalinity (mg/L $CaCO_3$)	202.4
CBOD-5 day (mg/L)	126
COD (mg/L)	349
Floc Filtered COD (mg/L)	87.64
Soluble COD (0.45 micron) (mg/L)	107.42
Ammonia-N (mg/L)	18.1
Total Kjeldahl Nitrogen (mg/L)	28.18
Soluble TKN (0.45 micron) (mg/L)	21.06
Soluble TKN (GF) (mg/L)	22.16
Soluble Orthophosphate (0.45 micron) (mg/L)	5.624
Total Phosphorous (mg/L)	3.766
Soluble Total Phosphorus (0.45 micron) (mg/L)	3.98
Soluble Total Phosphorus (GF) (mg/L)	3.594
TSS (mg/L)	142
ISS (mg/L)	12.8
Field pH (SU)	7.256

Table 4-3. Summary of Influent Wastewater Ratios for SWWRF and AWWRF		
Fractions	AWWRF	SWWRF
COD/ $CBOD_5$	2.77	5.34
COD/TSS	2.45	2.04
$CBOD_5$/TSS	0.89	0.54
NH4/ TKN	0.64	0.69
NH4/ COD	0.05	0.04
NH4/ $CBOD_5$	0.14	0.18
TKN/ COD	0.08	0.05
TKN/ $CBOD_5$	0.22	0.25
sTKN/TKN (0.45)	0.75	0.74
sTKN/TKN (GF)	0.79	0.77
TP/ COD	0.02	0.01
TP/ $CBOD_5$	0.04	0.03
TP/ NH3	0.31	0.19
TP/ TKN	0.20	0.13
sTP/TP (0.45)	0.66	0.48
sTP/TP (GF)	0.70	0.50
sO-P/TP	0.63	0.47
VSS/ TSS	0.91	0.72
VSS/COD	0.37	0.27
VSS/PCOD	0.55	0.38

4.1.1.5 Summary of Influent Wastewater Characterization Study

Results obtained from special sampling campaign were gathered together and assessed to determine the necessary influent wastewater fractions for process modeling. Table 4-4 summarizes important wastewater fractions adopted for the process simulation model.

Table 4-4. Influent Wastewater Fractions Adopted for BioWin Modeling for AWWRF and SWWRF Influent			
Name	BioWin Default	AWWRF	SWWRF
Fbs - Readily biodegradable (including Acetate)	0.16	0.15	0.14
Fac - Acetate	0.15	0.15	0.15
Fxsp - Non-colloidal slowly biodegradable	0.75	0.76	0.77
Fus - Unbiodegradable soluble	0.05	0.10	0.12
Fup - Unbiodegradable particulate	0.13	0.14	0.18
Fna - Ammonia	0.66	0.64	0.69
Fnox - Particulate organic nitrogen	0.50	0.50	0.50

Table 4-4. Influent Wastewater Fractions Adopted for BioWin Modeling for AWWRF and SWWRF Influent			
Name	BioWin Default	AWWRF	SWWRF
Fnus - Soluble unbiodegradable TKN	0.02	0.02	0.02
FupN - N:COD ratio for unbiodegradable part. COD	0.035	0.04	0.04
Fpo4 - Phosphate	0.50	0.63	0.47
FupP - P:COD ratio for unbiodegradable part. COD	0.011	0.011	0.011

4.1.2 Diurnal Sampling

Diurnal grab samples were collected at the SWWRF and AWWRF for three days, and analyzed for various parameters. The diurnal sampling results were used to determine the daily variations of flows and loads. Daily flows and loading factors were calculated by dividing the actual diurnal value to the daily average value. Figures 4-1 and 4-2 summarize the diurnal patterns for flow and COD, NH_3-N, and TP loads, respectively for the SWWRF. It should be noted that the two influent streams were combined into one influent, and the average results are presented below.

Figures 4-3 and 4-4 summarize the diurnal patterns for flow and COD, NH_3-N and TP loads, respectively for the AWWRF.

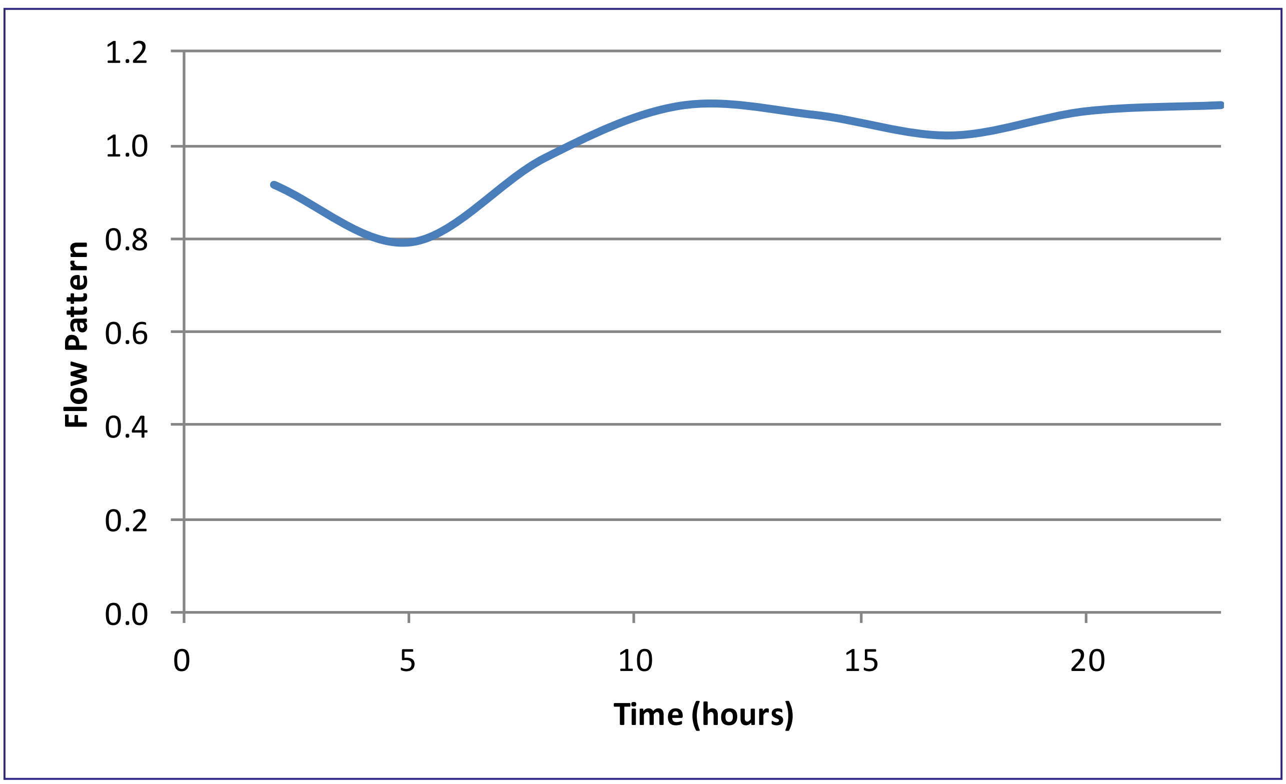


Figure 4-1. Average Influent Flow Diurnal Variation for the SWWRF

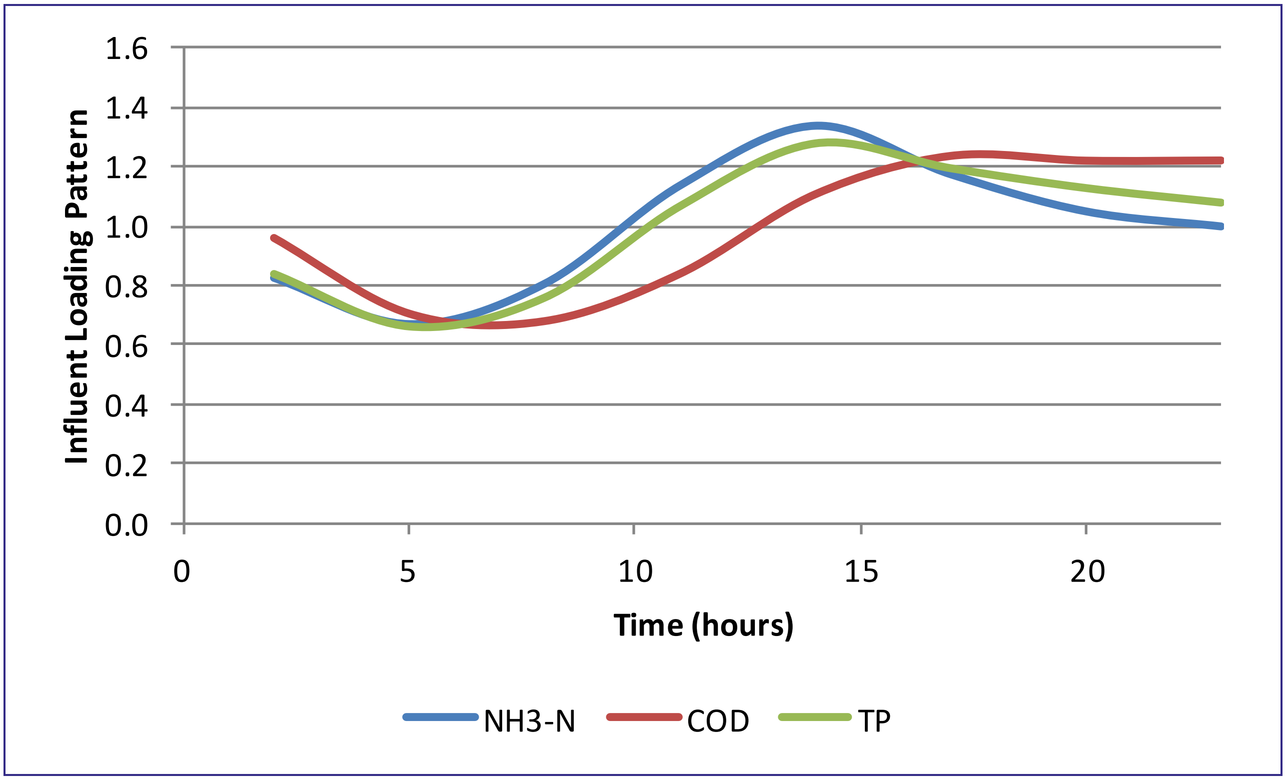


Figure 4-2. Average Influent COD, NH_3-N and TP Loads Diurnal Variation for the SWWRF

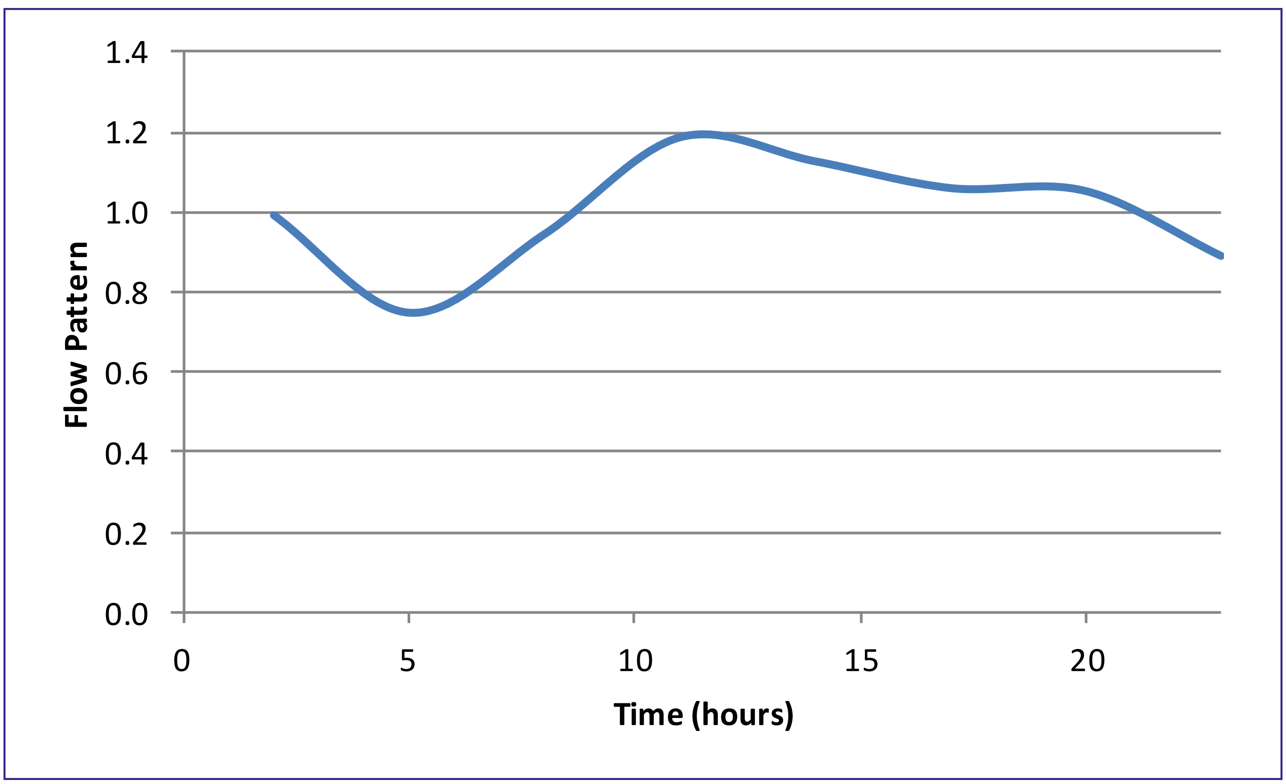


Figure 4-3. Average Influent Flow Diurnal Variation for the AWWRF

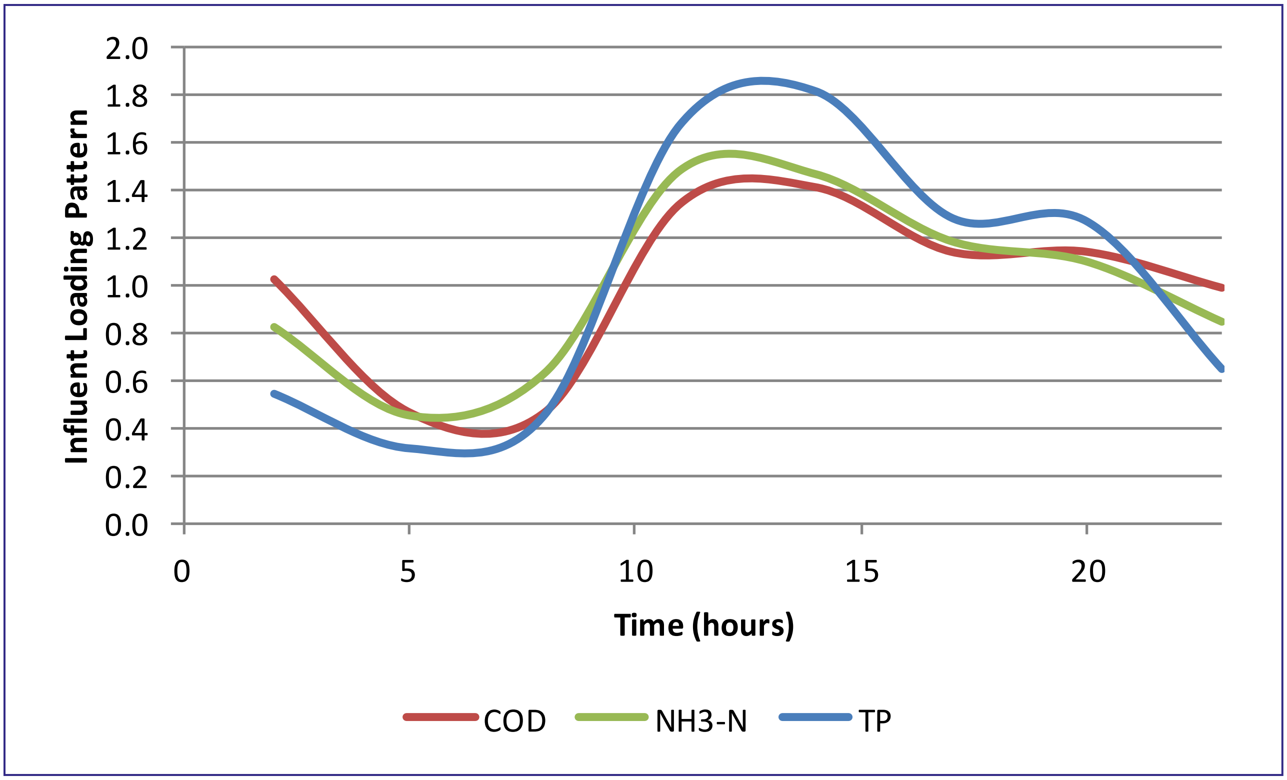


Figure 4-4. Average Influent COD, NH_3-N and TP Loads Diurnal Variation for the AWWRF

4.2 Activated Sludge Simulator Calibration

A model for the SWWRF was created using the BioWin simulator, developed by EnviroSim Associates, Ltd of Flamborough ON, Canada. BioWin allows the prediction of complex biological interactions using various mechanistic and empirical models to represent material transformations and pollutant removals in the plant for both liquid and solids process streams. It enables the user to simulate carbonaceous oxidation and the fate of nutrients in activated sludge treatment facilities.

Process simulators are considered calibrated when their predictions can mimic measured performance data. For the purpose of this analysis, the simulator was calibrated using plant daily operational data collected during the special sampling camping conducted in September 2011. Important influent wastewater fractions summarized in Table 4-4, were adopted for the calibration of the process simulator. Other important influent fractions were adopted as BioWin default values.

Figure 4-5 is an example of the SWWRF's plant process flow schematic as created in the BioWin simulator.

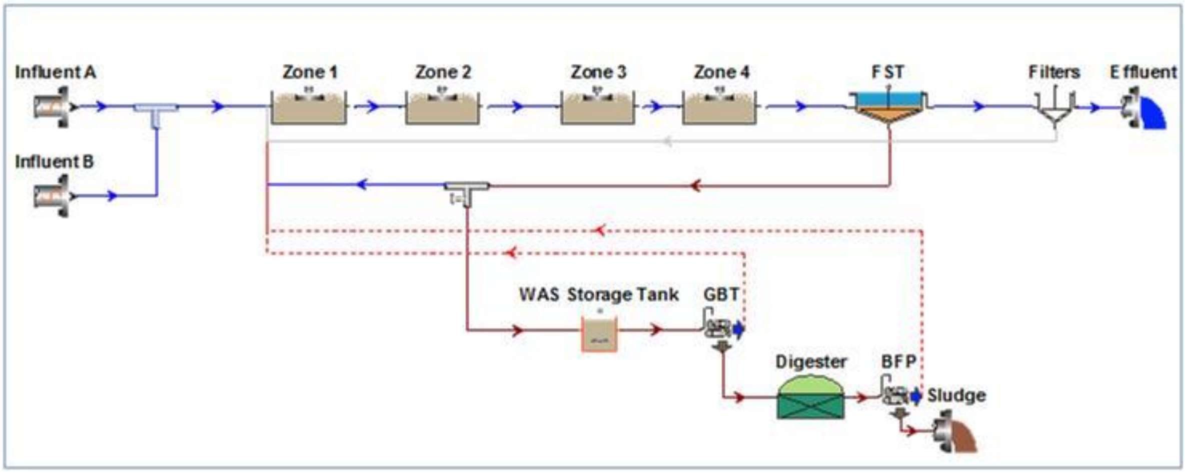


Figure 4-5. BioWin Process Schematic for the SWWRF

Steady-state and dynamic simulations of the plant operation for the September 2011 period were performed using the wastewater characterization data discussed above. The graphical representation of the plant layout and flow scheme was created as shown in Figure 4-5, in which physical data such as tank volumes and clarifier areas were specified as were process data such as influent flow rates and compositions, return activated sludge (RAS) flows and waste activated sludge (WAS) flows. For modeling purposes, the two aeration basins were combined into a single train and were configured as a series of sequential, complete-mix reactors to simulate their plug flow configuration. The aeration was set based on the rated capacity of the mechanical aerators and allowed the model to calculate the operating DO concentration in each zone. The three secondary clarifiers were combined into a single process unit with an area equivalent to the area of all the units in operation. The filtration system was configured as a simple liquid-solids separation unit with high removal efficiencies. The process simulator calculated all recycle flows and compositions from the filter's backwash water, and solids processing facility. For modeling purposes, it was assumed that all these recycle flows were directed to the head of the plant.

Simulator calibration was achieved by matching the predicted plant operating data with the diurnal and composite results for the plant performance during the September 2011 period. Table 4-5 summarizes the steady-state model predictions and compares these with the average values for plant measurements over the simulation period. Appendix E presents the results of the dynamic calibration of the BioWin simulator for the period of September 2011. Overall, the BioWin-predicted parameters are in close agreement with the plant-measured values for SRT, MLSS and effluent quality.

For the September 2011 period, the WAS rate needed to be increased significantly to match the observed MLSS concentrations in the reactors. For the calibration period, the WAS flow rate was modified to match the measured mixed liquor suspended solid concentration, measured WAS mass rate and the effluent parameters, within a reasonable margin of error. A mass balance around the liquid process units was conducted to verify the reported WAS flow rates. This analysis indicated that the plant reported WAS flow rates were very unlikely to be accurate and therefore they were not used for calibration purpose. Based on the mass balance results and for the model, the WAS flow rate was kept at approximately at 0.4 MGD. This average WAS flow rate matched well the operating SRT of the plant as well as the mixed liquor concentration and effluent constituents. It should be noted that WAS and RAS solids concentrations were not matched by the BioWin model. Two different mechanisms are used for WAS and RAS removal at the SWWRF, resulting in a higher solids concentration for WAS. The BioWin model could not match the measured WAS concentrations but it did predicted well WAS solids loads with those observed at the plant.

The effluent quality predicted by the model is in agreement with the values observed at the plant. Based on the effluent data collected during the special sampling, the low operating DO levels in the aeration basins allow for simultaneous nitrification and denitrification within the same reactor. Therefore, for modeling purposes the aerobic denitrification DO half saturation coefficient was modified to simulate the simultaneous nitrification-denitrification process taking place within the reactors.

For the purpose of this analysis, the solids processing facilities were included in the model. The BioWin model estimated the removal efficiencies in the anaerobic digesters well as well as the quality of the filtrate from the BFP. The observed and predicted total sludge yield coefficients were calculated to be 0.85 and 0.81, respectively, which are within values often reported in the literature. The overall sludge produced at the plant after solids processing was found to be 11 percent higher than the value predicted by the simulator. Despite the difference between the observed and predicted sludge production values, they are considered to be within adequate range.

The calibrated BioWin simulator was used to evaluate the process capacity of the SWWRF as presented in the following sections.

Table 4-5. BioWin Calibration Results Summary

Parameter	Units	Measured	BioWin Calibration
Aeration Basins			
MLSS	mg/L	1,800	1,760
MLVSS	mg/L	--	1,337
SRT	days	4.9	4.5
Return Activated Sludge (RAS)			
Flow	MGD	11.1	--
Concentration	mg/L	4,036	3,900
Waste Activated Sludge (WAS)			
Flow	MGD	0.0745	0.4
Concentration	mg/L	17,300	3,900
Mass Rate	lbs/d	10,750	11,915
Final Effluent			
TSS	mg/L	0.98	0.97
COD	mg/L	48.4	--
Soluble COD	mg/L	44.6	42.5
$CBOD_5$	mg/L	2.12	1.35
TKN	mg/L	10.8	11.8
NH_3-N	mg/L	9.3	10.3
NO_3-N	mg/L	0.25	0.17
NO_2-N	mg/L	0.75	0.74
TP	mg/L	0.64	0.75
Sludge Handling			
An. Digesters VSS in	%	2.8	2.5
An. Digesters VSS out	%	1.8	1.6
Filtrate NH_3-N	mg/L	734	799
Filtrate TP	mg/L	200	276
Cake	%	13.98	14.0
Total Mass of Sludge	lb/d	6,799	7,726

Section 5

Influent Basis of Design

This section presents a summary of the influent basis of design adopted for the treatment process assessment of the SWWRF. For the purpose of this analysis, it was assumed that the AWWRF is out of service and all the wastewater from its service area is diverted to the SWWRF for treatment and disposal.

5.1 Summary of Influent Conditions

The SWWRF is currently permitted to treat 20 MGD as annual average daily flow. The facility was designed to handle an average loading of approximately 33,360 lb/d as $CBOD_5$.The capacity of wastewater treatment plants are often associated with annual average conditions, as is the case in most of facilities in Florida including the SWWRF. However, these plants are required to handle not only the average flows and pollutant loading but also the maximum month and peak conditions associated with flow and pollutant loading variability. Therefore, when assessing the capacity of an existing treatment facility, the maximum month loading represents the design rated capacity of the plant. The maximum month loading condition represents the maximum sustained 30-day average load which the plant can treat without exceeding its discharge limits. The annual average loading condition is often used primarily to determine equipment capacity for optimum performance, to size chemical storage and sludge holding facilities, and to predict operation and maintenance costs. Peak day and peak hour loadings must be defined to accurately evaluate and design the aeration system to support the activated sludge process, the capacity of the secondary settling tanks, and some mechanical process units.

Plant operational data, in combination with data collected during the wastewater characterization study presented in Section 4 were used to develop the basis of design for the SWWRF. The basis of design represents the most rigorous conditions to meet the future limits and to maintain capacity.

Table 5-1 summarizes the influent criteria adopted for this analysis based on selected historical peaking factors for flow and loading conditions. Important influent fractions were adopted from the daily composite samples collected during the special sampling campaign. The peak hour peaking factors were adopted from data collected during the diurnal sampling campaign. It should be noted that based on information provided by the City, the future peak hour flow to the SWWRF was assumed at 40 MGD. In addition, it was assumed that the influent pollutant concentrations would not change in the future due to infiltration and inflow rehabilitation in the collection system or reduced water consumption.

The information presented in Table 5-1 represents the combined AWWRF-SWWRF influent characteristics. It should be noted that the pollutant loadings presented in Table 5-1 are slightly higher than the values originally used to design the SWWRF. The design values are presented in Section 3, Table 3-1.

Table 5-1. Summary of Influent Flow and Pollutant Loadings	
Parameter	Influent
Flow (mgd)	
Annual Average	20.00
Maximum Month	27.00
Maximum Day	33.54
Peak Hour	40.00
Average Temperature (°C)	25
Summer Temperature (°C)	28
$CBOD_5$ Load (lb/d)	
Annual Average	35,862
Maximum Month	46,621
Maximum Day	71,003
Peak Hour	89,954
TSS Load (lb/d)	
Annual Average	40,032
Maximum Month	52,042
Maximum Day	79,259
Peak Hour	100,413
TKN Load (lb/d)	
Annual Average	6,750
Maximum Month	8,775
Maximum Day	13,364
Peak Hour	16,931
TP Load (lb/d)	
Annual Average	998
Maximum Month	1,298
Maximum Day	1,976
Peak Hour	2,504

Section 6

Hydraulic Profile

In order to conduct the hydraulic analysis of the existing SWWRF, a hydraulic model of the liquid treatment train was created using Brown and Caldwell's PROFILE modeling software, a proprietary program developed by BC for calculating hydraulic and energy grade lines in water and wastewater treatment plants. The hydraulic model included existing facility structures from the filtration system to the Headworks structure. The model was constructed using available as-built drawings and other facility data provided by City. As-built facility elevations shown on the various drawings were field verified as part of this evaluation by the City.

The hydraulic profile model was constructed to take the most conservative path through the SWWRF, (excluding the "old plant"). Specifically, the flow path taken was through Clarifier # 3 which operates at a higher hydraulic level than Clarifiers # 1 and # 2 due to its higher V-notch weir setting. The modeled flow path continues through the northern-most filter, which is also the longest path through the filter process. In order to account for the worst case scenario of blinded filters, the hydraulic elevation in the filters was conservatively maintained at the filter overflow elevation of 115.3 feet.

The hydraulic capacity of the SWWRF was determined to be adequate, assuming the process units can maintain adequate amount of freeboard during peak flows and did not submerge any flow controlling weirs. These controlling weirs include the weirs in the Aeration Basin and Clarifier Splitter boxes and the Clarifier weirs.

6.1 Existing Plant Configuration

The hydraulic profile for the current permitted annual average flow (20 MGD) and peak hour (40 MGD) flow conditions with 18 MGD of RAS flow is shown in Figure 6-1. The modeling shows the SWWRF to be capable of hydraulically passing the flows at both flow conditions using only the "new plant".

6.2 Redundancy Evaluation

Another investigation was performed to model the hydraulic effect of certain process units being taken offline at peak flows. Four scenarios were modeled with the following units out of service:

- One of two aeration basins
- One of three secondary clarifiers
- One of four filter bays
- One of two chlorine contact basins

Figure 6-1 contains a table that identifies the hydraulic grade elevations for the locations shown in the hydraulic profile for each of these four scenarios. For the purposes of this investigation, it was assumed that all the process units were online.

6.2.1 One of Two Aeration Basins Out of Service

The aeration basin limiting hydraulic condition was to provide a minimum of a 1.5-foot of freeboard within the tank. With a wall height at elevation 125 feet, this meant the hydraulic grade of the aeration basin should not exceed elevation 123.5 feet. With one aeration basin out of service, the hydraulic elevation in the operating aeration basin is only minimally affected and remains under elevation 123.5

feet. The major hydraulic limitation for the tank is actually in the combined effluent channel and single 48-inch pipe conveying the flow to the Clarifier Splitter Box and sending all the flow through one basin does not affect the hydraulics in this area.

6.2.2 One of Three Clarifiers Out of Service

When modeling a scenario in which a clarifier is taken out of operation, the conservative assumption is that Clarifier # 2 is in operation along with Clarifier # 3 to allow the longest stretch of effluent piping to be used. As discussed previously, Clarifier # 3 is located the furthest from the filters with the highest hydraulic operating elevation and is therefore part of the most conservative flow path through the SWWRF.

With only two clarifiers in operation (Clarifiers # 2 and # 3), the plant would be able to pass a total of 37 MGD of wastewater (with 20 MGD of RAS) before submerging the weirs. The higher flows to each of the two operating clarifiers and associated effluent piping would limit the total plant capacity. The Clarifier effluent piping has minor bottleneck at the segment of 48-inch pipe that conveys the combined flows from all 3 clarifiers to the filters. With only Clarifiers # 3 and # 2 in operation, the combined effluent flows occur further upstream in the 48-inch pipe (after Clarifier # 2), worsening the bottleneck condition.

At flows exceeding 37 MGD, wastewater would still pass through the plant; however, the Clarifier weirs would be submerged, increasing the amount of solids passing into the secondary effluent and carrying into the filters. A parallel effluent pipe could be provided to relieve this bottleneck, but it would not be cost effective to relieve this bottleneck as this operating scenario is rare and the consequences of this hydraulic restriction are minor.

However, if an additional Clarifier is planned in the future with additional flow, the effluent from that clarifier should not be added to the existing 48-inch pipe. An alternate routing should be developed to convey the secondary effluent to the filters.

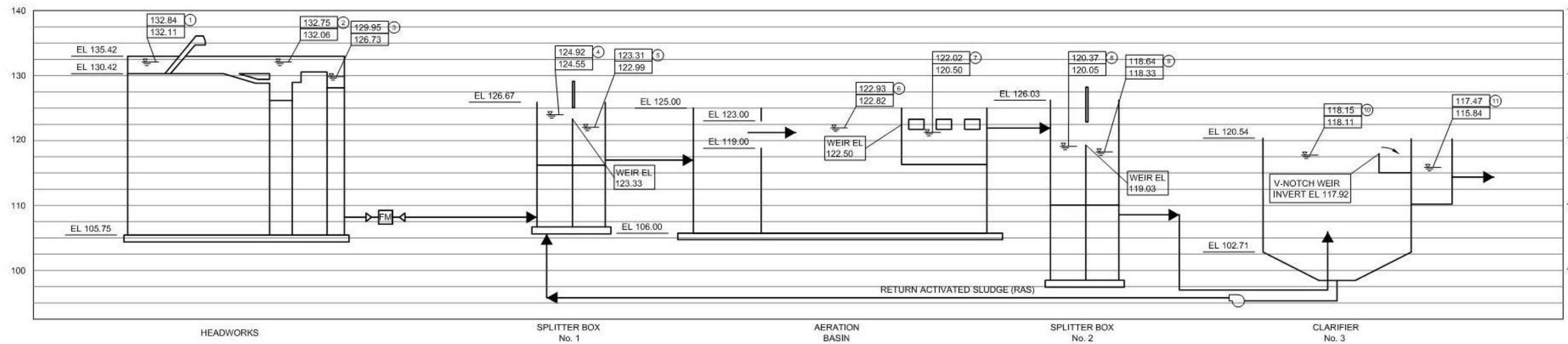
6.2.3 One of Four Filter Bays Out of Service

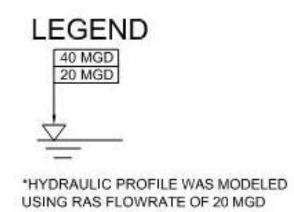
The internal flow distribution (channels and distribution troughs) and effluent piping of the deep bed filters are adequate to pass 40 MGD of flow using only three filter bays. This is particularly important as this scenario occurs on a regular basis during normal operation as the deep bed filters will periodically go into backwash cycles, cutting off that filter bay from passing any flow.

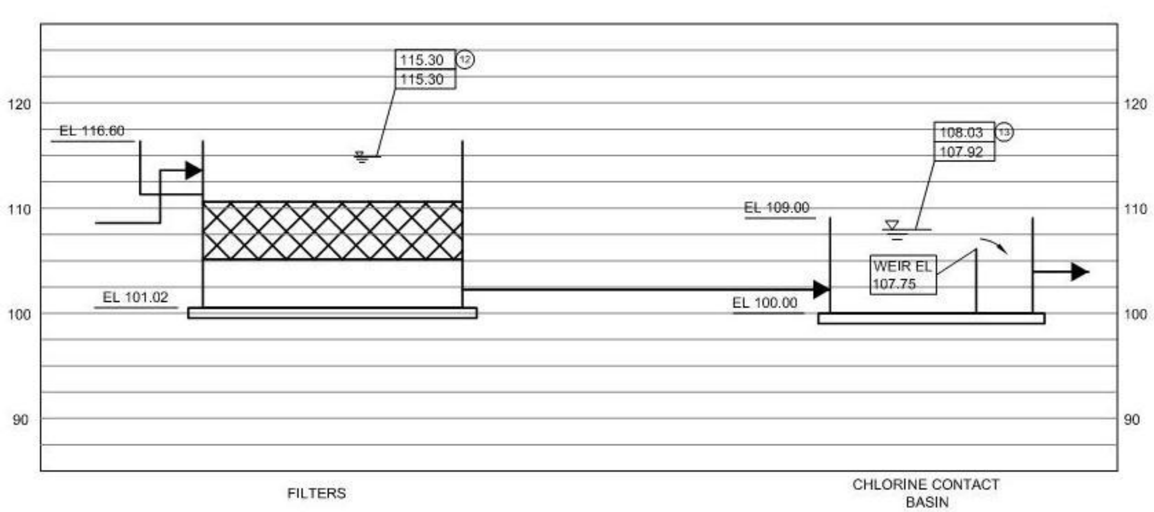
In this scenario, five feet of hydraulic head is available to pass the flow through the filter media. However, as the media becomes blinded with solids, the amount of hydraulic head required to pass the flow through the media will increase. As the operating hydraulic elevation rises in the filters to elevation 115.1 feet, the filter bed should engage in a backwash cycle to remove contaminants in the filter media and restore performance before incoming flow is discharged through the overflow which is set at elevation 115.3 feet. However, if the incoming secondary effluent has a high concentration of solids at a high flow rate, it is possible that the filter media can become blinded faster than backwash cycles can be performed.

6.2.4 6.2.4 One of Two Chlorine Contact Basins Out of Service

One side of the Chlorine Contact Basin (CCB) is capable of hydraulically passing 40 MGD of flow with minimal effect on the upstream processes although the increased flow rate through the basin would require higher chlorine dosage to ensure adequate disinfection.







Location		Modeled Condition and Flow						
Number	Description	All Units in Service - 20 MGD	All Units in Service - 40 MGD	1 Aeration Basin Out of Service - 40 MGD	1 Clarifier Out of Service - 40 MGD	1 Clarifier Out of Service - 37 MGD	1 Filter Bay Out of Service - 40 MGD	1 Chlorine Contact Basin Out of Service - 40 MGD
1	Headworks Upstream of Screen	132.11	132.84	132.84	132.84	132.74	132.84	132.84
2	Headworks Downstream of Screen	132.06	132.75	132.75	132.75	132.65	132.75	132.75
3	Headworks Effluent Box	126.73	129.95	130.90	129.95	129.16	129.95	129.95
4	Aeration Basin Splitter Box Upstream	124.55	124.92	125.86	124.92	124.87	124.92	124.92
5	Aeration Basin Splitter Box Downstream	122.99	123.31	124.76	123.31	123.26	123.31	123.31
6	Aeration Basin	122.82	122.93	123.24	122.93	122.91	122.93	122.93
7	Aeration Basin Effluent Channel	120.50	122.02	122.02	122.40	122.19	122.02	122.02
8	Clarifier Splitter Box - Upstream	120.05	120.37	120.37	120.79	120.73	120.37	120.37
9	Clarifier Splitter Box - Downstream	118.33	118.64	118.64	119.51	119.17	118.64	118.64
10	Clarifier No. 3	118.11	118.15	118.15	118.41*	118.18	118.15	118.15
11	Clarifier No. 3 Effluent Box	115.84	117.47	117.47	118.16	117.76	117.58	117.47
12	Filters	115.30	115.30	115.30	115.30	115.30	115.30	115.30
13	CCB	107.92	108.03	108.03	108.03	108.01	108.03	108.23
	* Clarifier weir is submerged							

Figure 6-1. SWWRF Hydraulic Profile

6.3 Future Modifications

Section 7 discusses recommended process changes in order to handle future loadings to the plant. The primary clarifiers discussed in that section would be located downstream of the headworks and upstream of the aeration basins. The primary clarifier and associated conveyance piping, flow splitting structures, and flow measurement devices will need to be designed to fit hydraulically between those process units.

At peak flow, there is approximately 8.5-foot hydraulic grade difference between the headworks channel and the upstream side of the aeration basin splitter box. The available hydraulic grade differential is sufficient to include future primary clarifiers. As comparison, flow through the secondary clarifiers results a little more than 3 feet of head loss at peak flow.

The only potential bottleneck in this area is the influent flow meter located after the headworks. The 48-inch pipe is reduced to 30-inch for the flow meter generating significant headloss at peak flows. As part of the design of the primary clarifiers, the influent flow meter would likely need to be replaced or relocated. Considerations may need to be made for the removal of this bottleneck as part of that design.

Section 7

Treatment Assessment of the Liquid Plant and Alternative Analysis

This section describes the treatment process assessment of the liquid treatment process units at the SWWRF, and the evaluation of possible process modifications to restore the capacity of the facility. Detailed wastewater characterization data as well as a calibrated process simulator presented in Section 4 and a clarifier hydrodynamic model were used during this analysis. It should be noted this analysis focused only on the secondary process capacity of the SWWRF. This chapter does not include possible process modifications or requirements for hydraulic improvements, preliminary treatment, or solids processing facilities. These are presented in other sections of the report.

A treatment process assessment was conducted for each of the existing facilities at the SWWRF. The projected pollutant loadings used for this evaluation were developed in Section 5 and are presented in Table 5-1.

Currently, the secondary treatment process at the SWWRF comprises two 2.015 MG rectangular reactors operated in parallel. Aeration is provided by a series of mechanical surface aerators. However, for the purpose of the capacity analysis, it was assumed that these were replaced by new fine bubble aeration system and new blowers. As presented in the 2005 Aeration Conversion Project, Basis of Design Report prepared by Boyle Engineering Corporation, the new aeration system will consists of 3,500 9-inch flexible membrane diffusers (per reactor) and four new aeration blowers (with one full standby) with a total capacity of approximately 18,000 scfm. The aeration system was designed for the facility at an average flow of 20 MGD and for carbonaceous removal only. It addition, it was assumed that anaerobic selectors with a volume of 0.40 MG per reactor were installed at the front-end of the aeration basins as recommended in the 2005 report. The secondary settling system at the SWWRF comprises three center-fed 130-foot diameter clarifiers. Secondary clarifiers # 1 and 2 are equipped with a combination of inboard and outboard launders whereas clarifier # 3 only uses outboard launder. The RAS system comprises three pumps with a total capacity of approximately 18 MGD. Excess sludge is wasted from the system to a holding tank, and then it passes through GBT, mesophilic anaerobic digestion and BFP. Dewatered digested sludge is finally land applied as a Class B biosolids. Return streams from the filters, GBT and GBT are sent to the head of the plant for treatment.

For the purpose of this analysis, it was assumed that the “old plant” remained out of service and that the facility had a rated capacity of 4 MGD as annual average flow. According to the City, this facility could be brought online if capacity limitations are observed at the “new plant”.

7.1 Treatment Assessment

The approach adopted for the treatment assessment was to evaluate each unit process within the plant independently to identify any shortcomings that could be limiting the overall capacity and performance of the facility. The treatment process assessment was conducted based on the ability to meet the current treatment requirements for the SWWRF, which are listed in Section 2, Tables 2-2 and 2-3.

The calibrated BioWin model developed in Section 4 was used to determine the treatment capacities of the existing facilities and future required facilities for the SWWRF. Figure 7-1 shows the BioWin process schematic used for this analysis. Plant operating parameters used in the model are, to the greatest extent possible, developed from actual plant data and verbal information supplied by plant operators and staff. Operating parameter information that was not available through these sources was based on Brown and Caldwell experiences with similar plants.

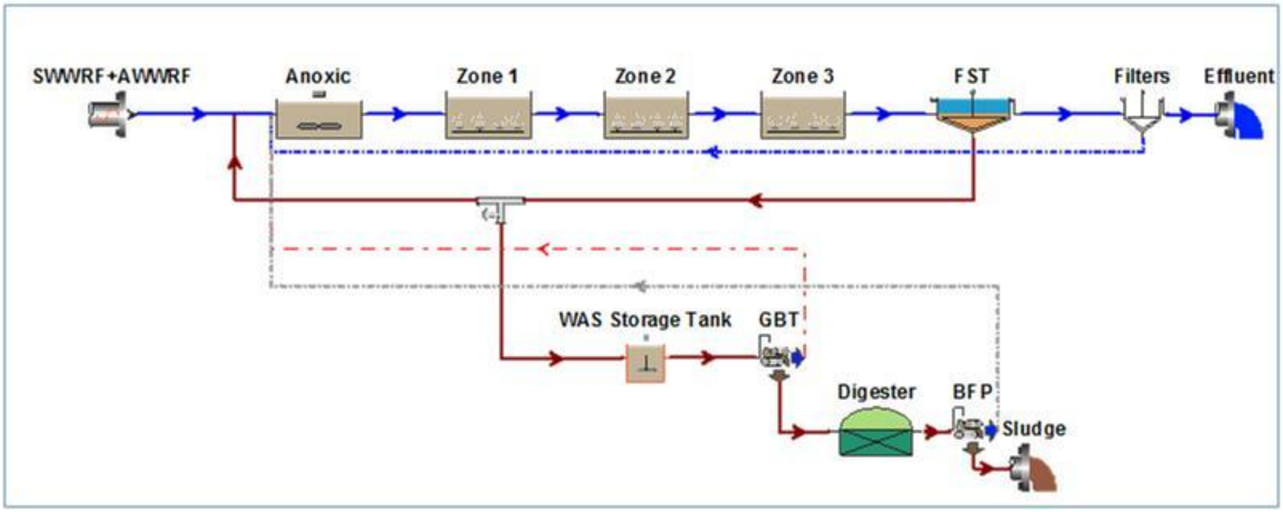


Figure 7-1. BioWin Process Flow Schematic for the SWWRF

The existing biological system at the SWWRF is operated at a solids retention time (SRT) of approximately 4.5 days based on historical operational data. Based on process modeling observation and review of the effluent quality data, limited nitrification is currently being observed at the facility due to the low DO levels currently carried in the biological reactors. However, such mode of operation has resulted in poor mixed liquor settling as observed by the high sludge volume index (SVI) values at the plant with average and 90-percentile values of 205 mL/g and 270 mL/g. These high SVI values could potentially limit the capacity of the secondary clarification system at the SWWRF.

For the purpose of this analysis, the BioWin process model was used to investigate the effect of the operating SRT on effluent quality and on the mixed liquor concentration in the aeration basins at annual average conditions. The SWWRF currently operates, most of the time, as a carbonaceous removal only facility; hence, low SRT values are required. Based on microbial kinetics at the annual temperature of 25^{o}C, the washout SRT for nitrifiers (minimum SRT required for the growth of nitrification bacteria) is approximately 1.0 day if oxygen availability does not limit the reaction. Figure 7-2 depicts the BioWin modeling steady-state results, and shows that a SRT value of approximately 2.0 days or less is required to minimize nitrification at average conditions at the SWWRF. It should be noted this analysis was conducted at an average DO concentration of 2.0 mg/L as specified in the 2005 report. Normally, plants operating at such low SRT are unstable and difficult to operate due to the limited biomass inventory available for biological oxidation of pollutants. High-rate activated sludge plant such as those using pure oxygen have achieved stable operation at SRT values as low as 1.5 days. However, in the case of air-activated sludge facilities, such as the SWWRF, minimum SRT values of approximately 2.5 days have shown operational stability. For the purpose of this analysis and based on Brown and Caldwell's experience, a design SRT of approximately 2.5 days was adopted for this analysis. Figure 7-2 also presents the effect of the SRT on the average mixed liquor concentration in the aeration basins, which indicates that at the selected SRT of 2.5 days, an average mixed liquor concentration of approximately 3,600 mg/L would be obtained.

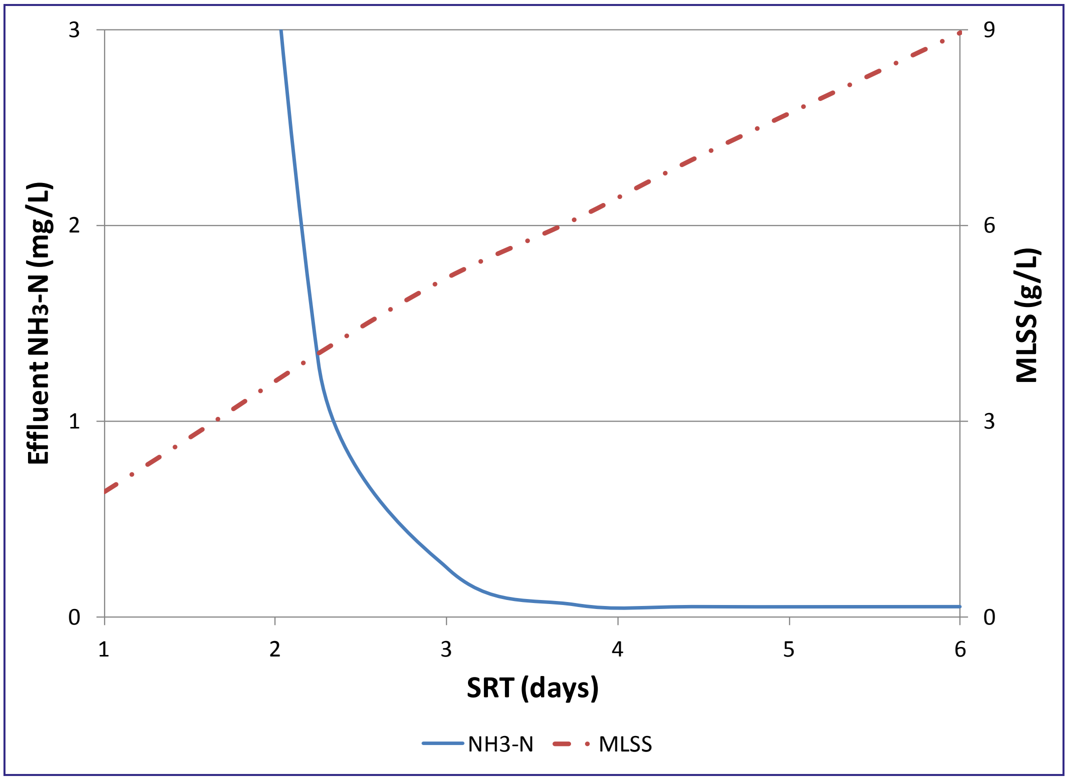


Figure 7-2. Effect of Solids Retention Time on Effluent Ammonia and MLSS Concentrations at the SWWRF at an Annual Average Temperature of 25°C and Average DO Concentration of 2.0 mg/L

7.1.1 Aeration System

The BioWin model was used to estimate the oxygen uptake rates (OUR) for each zone of the aeration tanks. For the purpose of this analysis, two aeration scenarios were evaluated – 1) carbonaceous removal only; and 2) carbonaceous removal and nitrification. Brown and Caldwell is of the opinion that because of the high water temperatures in Florida, avoiding nitrification at the SWWRF will be a challenge; hence, the aeration system should be designed to accommodate the higher aeration demands as a result of nitrification. Table 7-1 summarizes the total aeration requirements for the "new plant" under the two scenarios. Overall, the new aeration system currently being added to the facility has the capacity to treat 20 MGD as annual average flow if carbonaceous removal only is considered. Based on the modeling predictions, an average and a peak hour aeration demands of 13,250 scfm and 16,600 scfm are required for carbonaceous removal. In the case that nitrification and carbonaceous removal occur in the aeration basins, the average and peak hour air demands are 18,160 scfm and 22,400 scfm, which are approximately 35 percent higher than the values presented in Scenario 1. Scenario 2 will require an increase to the capacity of the aeration to include nitrification and carbonaceous removal. Therefore, if the existing aeration system is kept as presented in the 2005 report, the capacity of the aeration system would be decreased from 20 MGD to 15 MGD as annual average flow.

Brown and Caldwell recommends the addition and implementation of online, automatic DO control and ammonia online analyzers to control the aeration system. This additional instrumentation system will allow provide operational benefits at the SWWRF including minimizing the nitrification reaction in the aeration tanks.

Table 7-1. Summary of Aeration Requirements for Base Scenario		
Parameter	Scenario 1	Scenario 2
Zone volume, MG	3.426 (aerobic volume)	3.426 (aerobic volume)
Sidewater Depth, ft	17.0	17.0
AOR (average load/ peak load), lb O/d	1,610/ 2,170	2,300/ 3,100
DO (average load/ peak load), mg/L	2.0/ 1.0	2.0/ 1.0
Average Alpha	0.50	0.50
Average SOTE, %	31.4	31.4
Average Air Flow, scfm	13,250	18,160
Maximum Air Flow, scfm	16,600	22,400
Diffuser Type	Fine Bubble Diffuser	Fine Bubble Diffuser
Diffuser Number	7,000	12,000
Diffuser density, At/Ad	10.51	6.08

7.1.2 Secondary Clarifiers

For the purpose of this evaluation, a two-dimensional secondary clarifier hydrodynamic model "2Dc" was used to assess the capacity of the secondary clarification system. This model was developed by a research team led by Professor J. Alex McCorquodale at the University of New Orleans, Louisiana, and it accounts for axi-symmetric hydrodynamics (including swirl components), sludge settling, turbulence, sludge rheology, flocculation, clarifier geometry, and varying hydraulic loadings. Discrete particle settling, flocculation-induced settling, hindered settling, and compression settling also are described by the model.

An extensive field data collection campaign and clarifier stress testing were conducted at the SWWRF by Brown and Caldwell's process engineers with the goal of collecting adequate information to understand specific on-site information to calibrate the hydrodynamic clarifier model. Appendix F presents a technical memorandum summarizing the field data collection, CFD model calibration and treatment process assessment results for the SWWRF's secondary clarification process.

Table 7-2 summarizes some of the clarifier characteristics. The secondary clarification system has a maximum RAS capacity of 18 MGD.

Table 7-2. Secondary Clarifier Characteristics		
Parameter	Clarifiers 1 & 2	Clarifier 3
Clarifier Diameter, ft	135	135
Depth of Outer Wall, ft	12	15
Centerwell Diameter (Internal), ft	16	16
Centerwell Depth, ft	7	7
Effluent Launders	Inboard and Outboard	Outboard
Sludge Collection	Suction - Organ Pipe	Suction - Organ Pipe

The aeration basins and the secondary clarifiers operate as a combined process. The solids loading rate (SLR) on the secondary clarifiers is a function of both the flow to the clarifiers (including influent flow and RAS flow) and the MLSS concentration in the aeration basin. As the MLSS concentration increases, the clarifier SLR also increases. In addition, the secondary clarifiers also have hydraulic loading limitations to prevent solids washout; and these are represented by the surface overflow rate (SOR). Therefore, the CFD model was used to estimate the capacity of the secondary clarifiers in terms of SLR and SOR. For the purpose of this analysis, failure was defined as an excursion of the effluent suspended solids exceeding 60 mg/L, which corresponds to the maximum daily value in the existing permit.

One of the most important factors when assessing the capacity of secondary clarification system is the selection of the design mixed liquor settling characteristics. Often, historical operation data on SVI is available and can be used as reference or guideline in the selection of the design SVI value. There is an inherent uncertainty in sizing clarifiers based on historical SVI data, although Brown and Caldwell recommends using the 90-percentile value. Based on the January 2007- November 2011 data, average and the 90-percentile SVI values for the SWWRF are approximately 205 mL/g and 270 mL/g. These values are considered high and might be the result of low DO bulking conditions in the aeration basis. With the modifications of the existing aeration basins, including new anaerobic selectors and new aeration systems, it is expected that the quality of the mixed liquor be improved dramatically. An analysis of SVI data from facilities similar to the SWWRF but with adequate aeration control indicated that 90-percentile SVI values ranging from 120 to 150 mL/g can be achieved. Therefore, for the purpose of this analysis, this SVI range was adopted for the SWWRF. It should be noted that Brown and Caldwell recommends the following in order to minimize high SVI values:

- Effective automatic DO control to minimize the risk of low DO bulking.
- The ability to add polymer to enhance settling rates during high flow events.

The aeration basin's mixed liquor concentration, adopted for this analysis, was obtained from the BioWin modeling dynamic results. The average mixed liquor concentration adopted for this analysis was approximately 3,600 mg/L.

Table 7-3 summarizes the CFD modeling results for the secondary clarifier assessment of the secondary clarifiers at the SWWRF. As expected, the capacity of the secondary clarifiers is significantly affected by the selection of the SVI value; the higher the SVI, the lower the capacity of the clarifiers would be. Based on the information presented in Table 7-3, the capacity of the secondary clarification system at the SWWRF varies between 14.5 and 17 MGD as annual average daily flow or 29 to 34 MGD as peak hour flow. This table also presents, for reference purposes, the capacity of the secondary clarifier if no improvements in the SVI are achieved. It should be noted that the capacity at the existing SVI is less than the current flow because the mixed liquor concentration was kept at 3,600 mg/L which is approximately 63 percent higher than the current values. As the CFD modeling results indicate, the capacity of the secondary clarification system at the "new plant" does not have adequate capacity to handle the SWWRF's rated capacity of 20 MGD as annual average flow. However, based on the original design of the "new plant", this was designed to treat an average flow of 16 MGD, not 20 MGD.

Table 7-3. CFD Modeling Results for the Treatment process assessment of the Secondary Clarifiers			
Parameter	SVI of 270 mL/g	SVI of 150 mL/g	SVI of 120 mL/g
Average MLSS (g/L)	3.60	3.60	3.60
RAS (MGD)	18	18	18
Effluent SS (mg/L)	50	49	36
Sludge Blanket Height (% of total depth)	72	62	65
SLR (lb/d-fs)	22.5	32.85	36.35
SOR (gpd/fs)	326	675	792
Capacity [Average/ Peak] (MGD)	7/ 14	14.5/ 29	17/ 34

Figure 7-3 shows the velocity vector fields and the concentration distributions of the simulation with an average SVI of 120 mL/g. As depicted in this figure, the sludge blanket (defined by the dark orange color) builds up above of the center well skirt creating an area prompt to short-circuiting of the flow. This allows for the development of a very limited clarification zone between the sludge blanket and the surface of the clarifier. Therefore, the higher velocities on the top of the sludge blanket are scouring the solids from the blanket to the effluent, limiting the capacity of the secondary clarifiers at the SWWRF. In addition, the size of the existing center well is too small providing limited time for flocculation and energy dissipation.

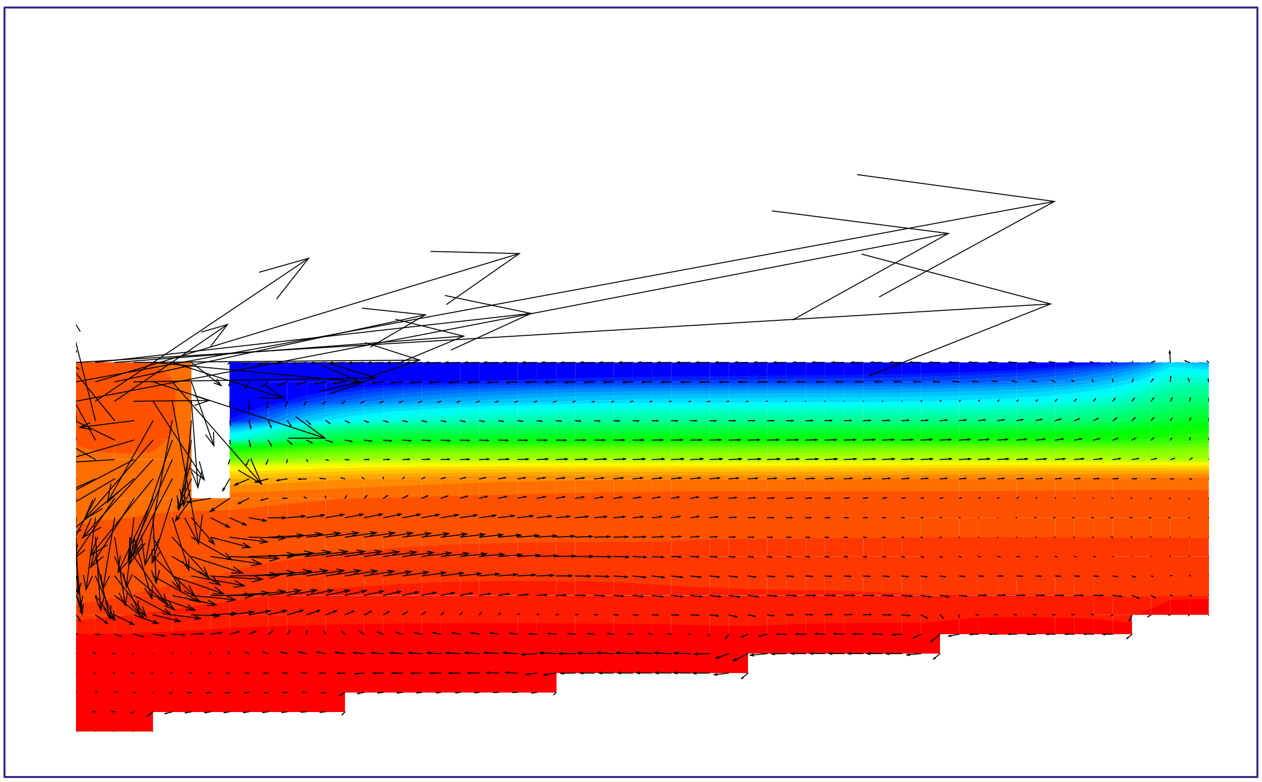


Figure 7-3. CFD model prediction for Secondary Clarifier # 1 at a design SVI of 120 mL/g - velocity vector field and solids suspended concentration at the maximum blanket level

As part of this assessment, physical modifications to the internal mechanisms of the secondary clarifiers were evaluated in order to increase capacity. Additional modeling was performed to determine if additional secondary clarifier capacity could be achieved by increasing the center wells of the secondary clarifiers. The flocculator center well is designed to dissipate energy of the incoming flow from the reactors as well as to provide contact and adequate detention time to promote flocculation of dispersed solids that may have broken up due to high degree of energy and flow conveyance between the reactors and the clarifiers. The existing center wells are sized smaller than modern design criteria for achieving optimal flocculation. The existing center wells are located at approximately 12 percent of the clarifier diameter and at 40 percent of total clarifier depth. Based on Brown and Caldwell's experience, the optimization alternative proposes a flocculator center well with a diameter of approximately 30 percent the clarifier diameter, and the skirt of the well to be extended to 50 percent of the total clarifier depth. For the purpose of this analysis, a design SVI of 120 mL/g was adopted. Table 7-4 summarizes the CFD modeling results for this alternative, which indicate that the changes to the existing center wells can provide significant capacity benefits at the SWWRF.

Table 7-4. CFD Modeling Results for the Treatment process assessment for Optimized Center Well

Parameter	Existing Center Well	Optimized Center Well
Center Well Diameter (%)	12	30
Center Well Depth (%)	40	50
Average MLSS (g/L)	3.60	3.60
RAS (MGD)	18	18
Effluent SS (mg/L)	36	45
Sludge Blanket Height (% of total depth)	65	60
SLR (lb/d-fs)	36.35	40.50
SOR (gpd/fs)	792	931
Capacity [Average/ Peak] (MGD)	17/ 34	20/ 40

Figure 7-4 shows the velocity vector fields and the concentration distributions of the clarifier simulation with the optimized flocculator center well at a design SVI of 120 mL/g.

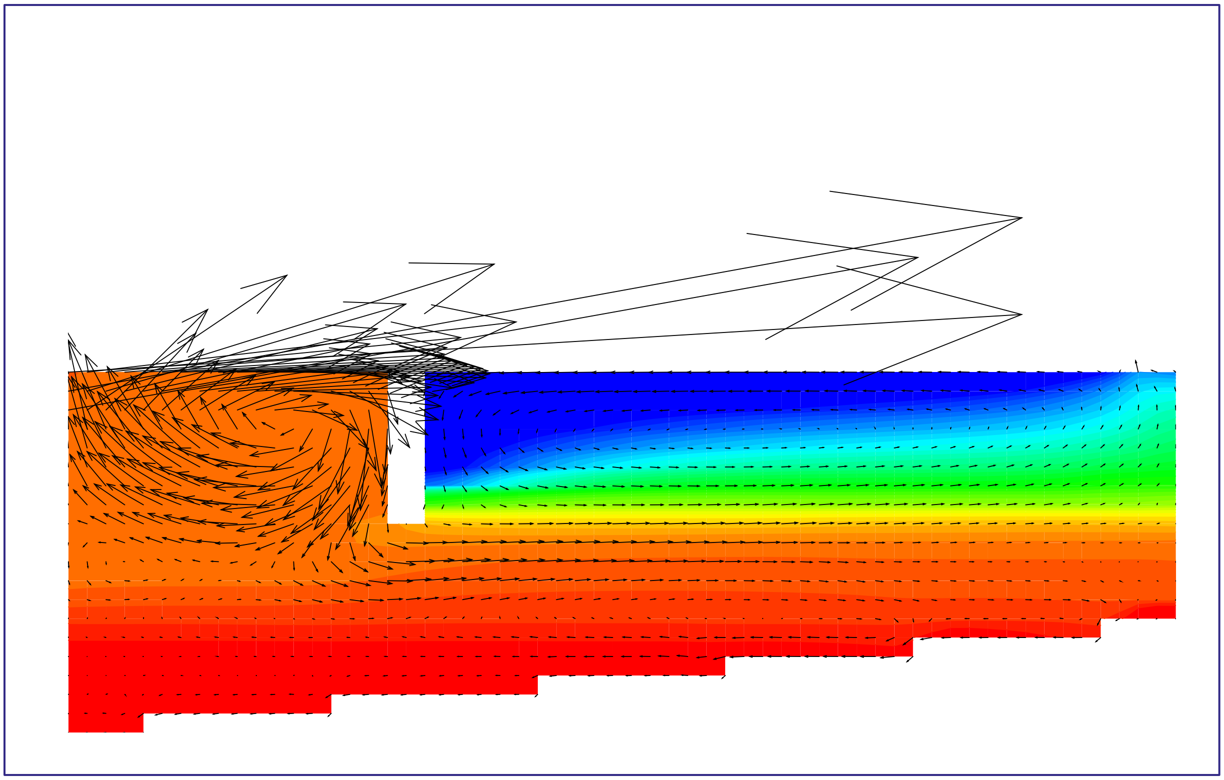


Figure 7-4. CFD model prediction for Secondary Clarifier # 1 with optimized center well and a design SVI of 120 mL/g - velocity vector field and solids suspended concentration at the maximum blanket level

7.1.3 Filtration and Disinfection

Filtration is provided by four multi-media filters with a total surface area of 5,624 square feet. Based on Brown and Caldwell's experience and information published in the MOP standards for tertiary filters, a filtration rate of less than 4.0 gpm/ft^2 based on the average design flow rate and 8.0 gpm/ft^2 at peak hourly flow is recommended. Applying these criteria, the filter complex capacity is adequate to handle the rated capacity of 20 MGD as annual average flow and 40 MGD as peak hour flow. This analysis assumes that all four filter basins are operational. The total capacity of the filtration system is approximately 30 MGD AADF and 60 MGD PHF. If one filter is out of service, the capacity of the filtration system is approximately 25 MGD AADF and 50 MGD PHF.

Disinfection is provided using sodium hypochlorite addition followed by two chlorine contact basins. For reclaimed water, Chapter 62-600 F.A.C. requires that the product of the chlorine residual and the contact time (or CT) be at least 120 mg-min/L. The required CT can be met using any combination of residual chlorine concentration and contact time provided that the chlorine residual is at least 1 mg/L. The chlorine feed system is operated to maintain a 4 mg/L residual in the contact basin effluent. Based on this residual chlorine concentration, a 30 minute detention time is required to meet the CT requirement. The volume of the existing basins provides 30 minutes of detention for a peak hourly flow in excess of 40 MGD. Therefore, the existing disinfection process at the SWWRF has adequate capacity to handle the plant's rated capacity of 20 MGD as annual average flow. The capacity of the disinfection system is approximately 23 MGD AADF and 46 MGD PHF.

7.1.4 Summary of Treatment Process Assessment

The results of the treatment process capacity evaluation are summarized in Figure 7-5. These results represent the capacity of the 'new plant" only.

This analysis indicated that the existing "new plant" has an overall capacity of approximately 17 MGD, and it is limited by the secondary clarification system. The aeration, filtration and disinfection facilities all have treatment capacities exceeding 20 MGD. In the case of the aeration system, this analysis assumes that the plant will be able to limit nitrification and operate as a carbonaceous removal plant only. If nitrification occurs, then the capacity of the aeration system will be de-rated from 20 MGD to approximately 15 MGD.

Plant modifications might be required to increase the capacity of the secondary clarifiers, which might include the addition of primary clarifiers, modification of the existing aeration basins to operate as a step-feed reactor, or the need to add additional secondary clarifiers. These alternatives were evaluated and are presented in Section 7.2 of this report.

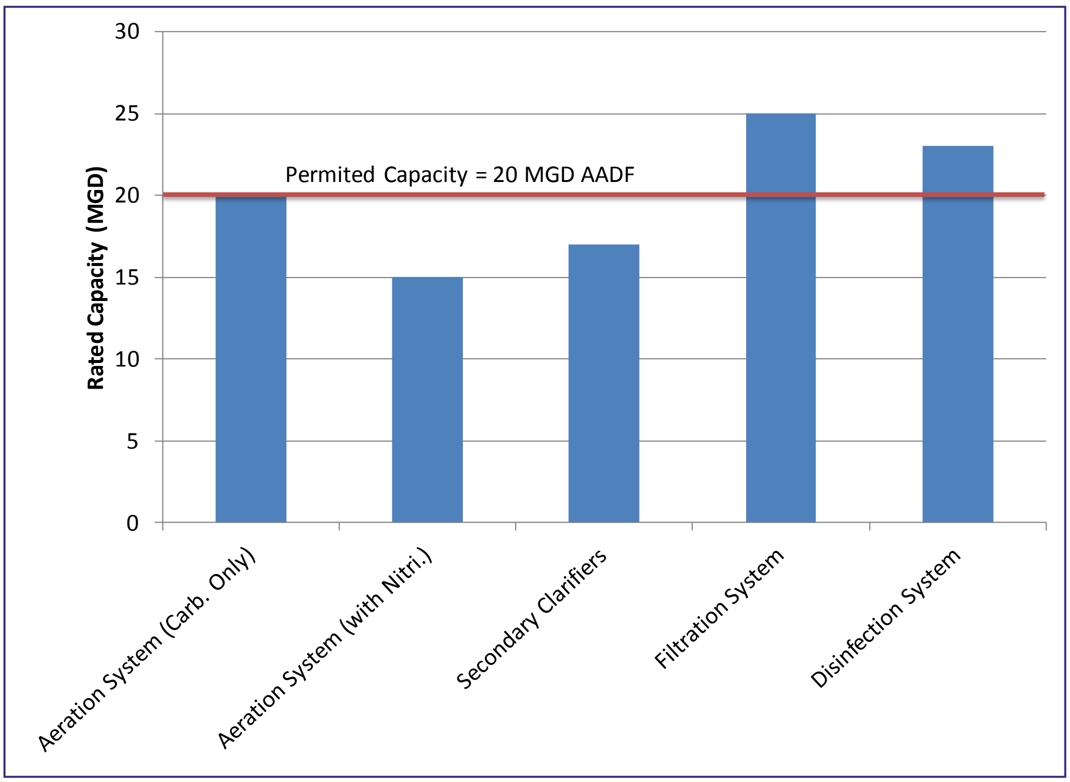


Figure 7-5. Summary of Process Capacity Evaluation for the SWWRF's new plant

7.2 Alternative Analysis

As part of this project, modifications to the existing SWWRF were evaluated in order to increase the through-put rates of those units limiting the overall capacity of the system. Based on the results presented in Section 7.1, the overall capacity of the SWWRF is limited at 17 MGD by the secondary clarification system. Therefore, the following alternatives were evaluated in order to increase the capacity to 20 MGD:

- Alternative 1: Step-feed system,
- Alternative 2: New primary clarifiers,
- Alternative 3: New secondary clarifiers.

All these alternatives aimed at reducing the loadings to the existing secondary clarifiers, which is the capacity-limiting process unit at the SWWRF.

7.2.1 Alternative 1: Step-Feed System

The process modifies the existing reactor configuration at the SWWRF by splitting the influent feed point along the length of the reactor. This process allows reduction of the overall mixed liquor concentrations applied to the secondary clarifiers, especially when the clarifiers need it the most, at high flow events. Figure 7-6 shows the BioWin process flow schematic for this configuration. Different step-feed proportions were evaluated with the BioWin model; however, the one that worked the best and is recommended during this analysis is that 50 percent of the influent flow is fed to the front-end of the reactor while the other 50 percent is diverted to the middle section of the reactor. During wet weather flows, influent flows in excess of 30 MGD will be directed to the last zone (3B) of the aeration basins for biological contact treatment. This step-feed configuration allows reducing the mixed liquor levels significantly compared to the current operation of the plant.

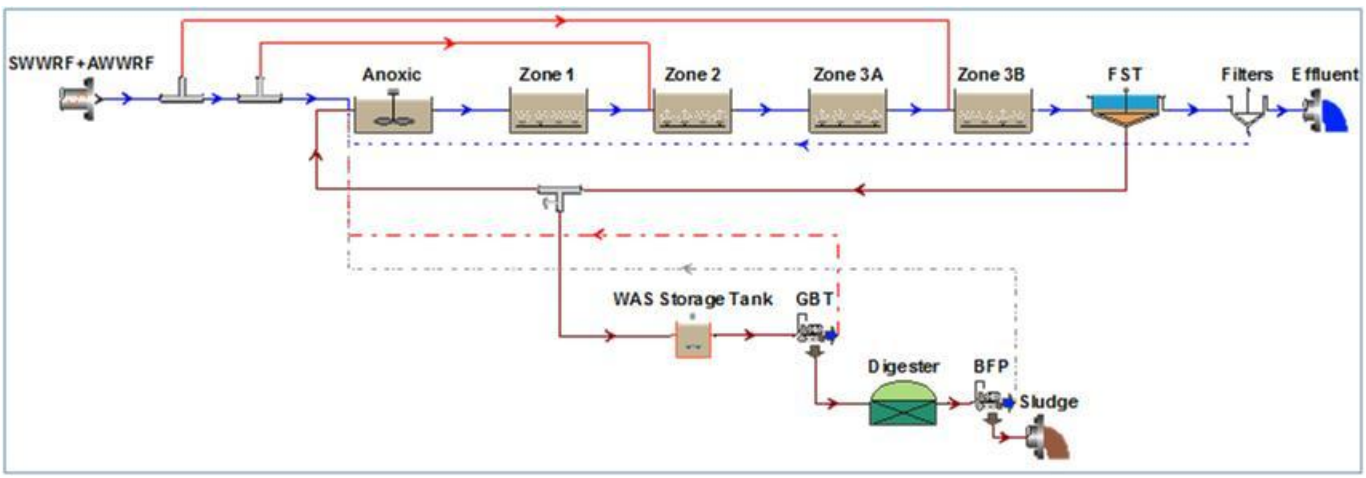


Figure 7-6. BioWin process flow schematic for the Step-Feed configuration

The BioWin model was used to determine the aeration requirements for this alternative. These values were used to estimate the aeration demands during average and peak hour conditions for the SWWRF and the results are summarized in Table 7-5. In general, the existing aeration system is capable of handling the aeration demands for carbonaceous removal only at 20 MGD; similar to the results presented previously. Similar to the results presented in Table 7-1, if nitrification occurs in the aeration basin, additional aeration capacity (diffusers and blowers) will be required.

Table 7-5. Summary of Aeration Requirements for Alternative 1

Parameter	Aeration Requirements for Carbonaceous Only Removal
Zone volume, MG	3.426 (aerobic volume)
Sidewater Depth, ft	17.0
AOR (average load/ peak load), lb O/d	1,400/ 2,180
DO (average load/ peak load), mg/L	2.0/ 1.0
Average Alpha	0.50
Average SOTE, %	32.5
Average Air Flow, scfm	11,660
Maximum Air Flow, scfm	17,650
Diffuser Type	Fine Bubble Diffuser
Diffuser Number	7,000
Diffuser density, At/Ad	10.51

The step-feed configuration was considered for the SWWRF since it will allow reduction of the loadings to the secondary clarifiers without major capital investment. Figure 7-7 shows an example of the mixed liquor profile in the aeration basins during the step feed operation predicted by the BioWin model. Based on these results, this configuration can reduce the loadings to the secondary clarifiers by about 61 percent.

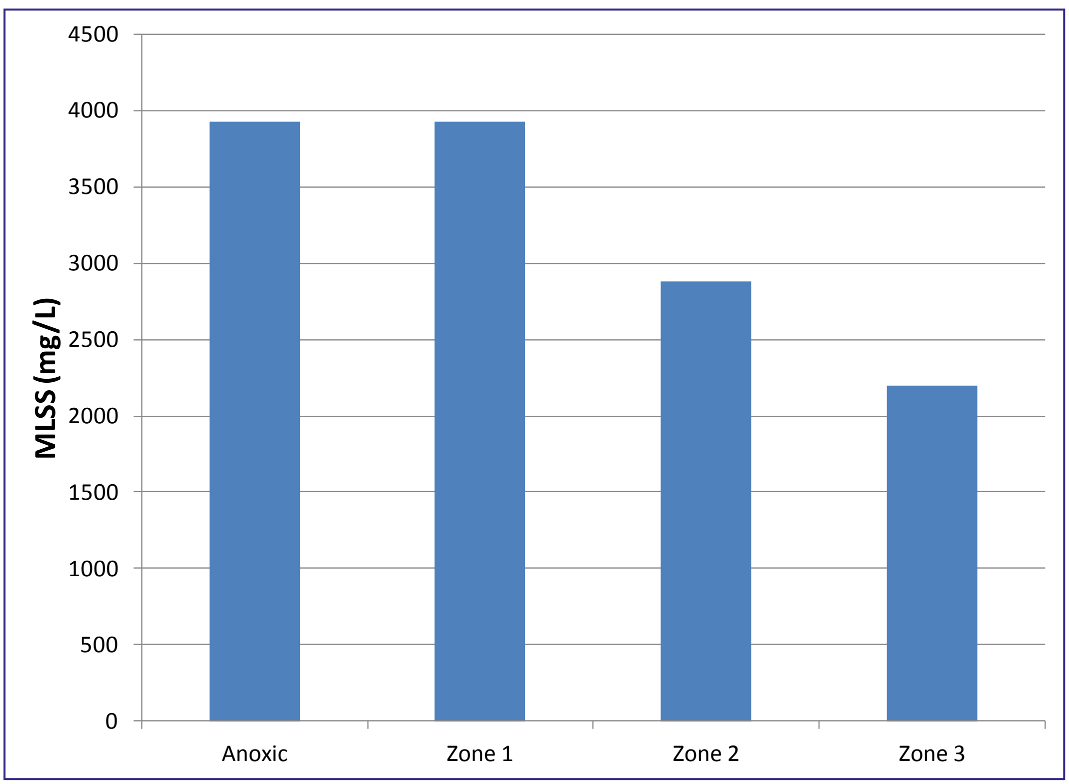


Figure 7-7. Mixed liquor profile in the aeration basins during the step-feed configuration at __-MGD flow

The hydrodynamic model was then used to estimate the capacity gains of the secondary clarification system due to the reduction in mixed liquor levels due to the step-feed configuration. For the purpose of this analysis and similar to the analysis conducted previously, two SVI values were used, 120 mL/g and 150 mL/g, and no improvements to the secondary clarifier internal structures were considered. Table 7-6 summarizes the CFD modeling results for Alternative 1. Based on the information presented in this table, the capacity of the secondary clarification system for Alternative 1 varies between 18 and 20 MGD. As the CFD modeling results indicate, the capacity of the secondary clarification system at the “new plant” can be increased to 20 MGD if the step-feed configuration is implemented.

Table 7-6. CFD Modeling Results for the Treatment process assessment for Alternative 1

Parameter	SVI of 150 mL/g	SVI of 120 mL/g
Average MLSS (g/L)	2.2	2.2
RAS (MGD)	18	18
Effluent SS (mg/L)	50	22
Sludge Blanket Height (% of total depth)	45	30
SLR (lb/d-fs)	31.46	33.80
SOR (gpd/fs)	838	931
Capacity [Average/ Peak] (MGD)	18/ 36	20/ 40

Figure 7-8 shows the velocity vector fields and the concentration distributions of the simulation with an average SVI of 120 mL/g for Alternative 1. From this figure, one can observe as the sludge blanket (defined by the dark orange color) stays below the center well skirt providing adequate clarification area during peak flow conditions. Short-circuiting is still observed below the center well’s skirt resulting in high blankets and high effluent suspended solids.

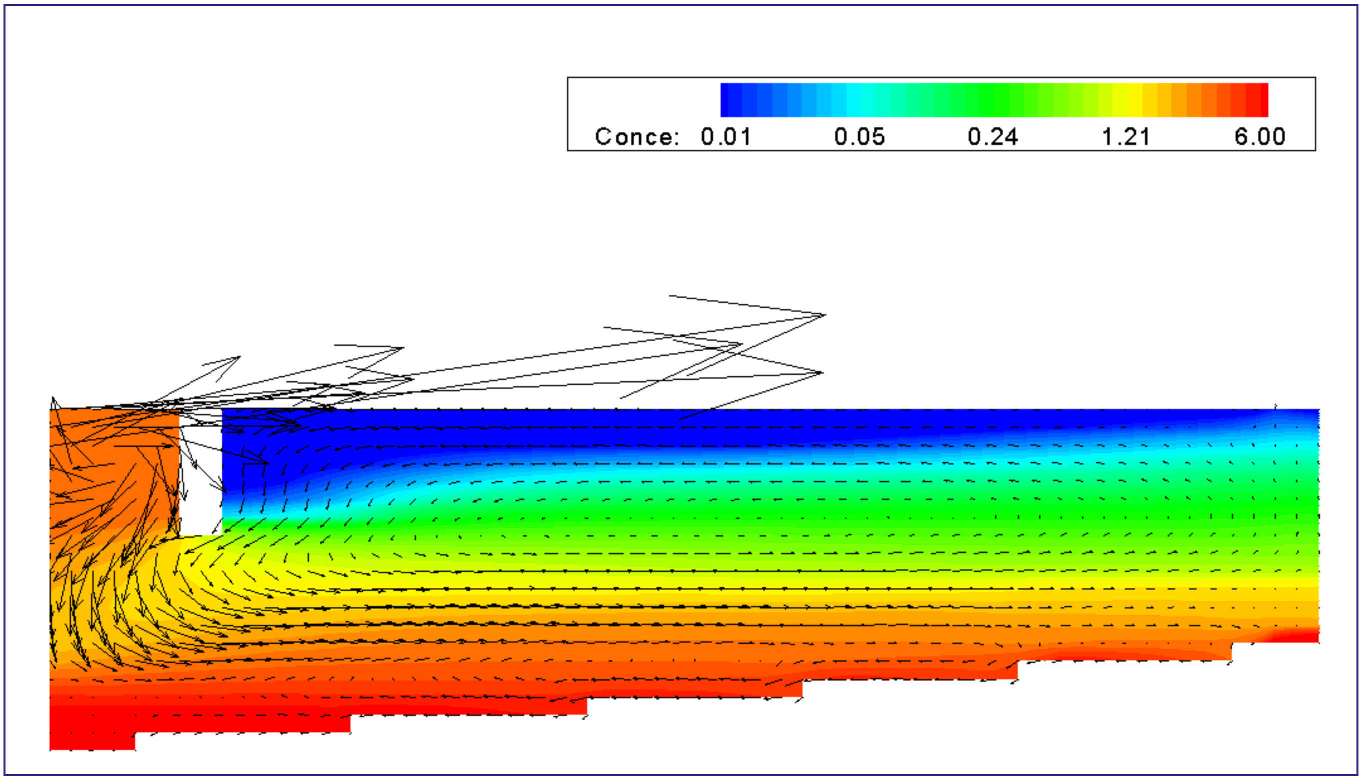


Figure 7-8. CFD model prediction for Alternative 1 at a design SVI of 120 mL/g - velocity vector field and solids suspended concentration at the maximum blanket level

7.2.2 Alternative 2: New Primary Clarifiers

Alternative 2 modifies the existing configuration at the SWWRF by the addition of a new primary clarification system to reduce the loadings to the activated sludge process. Primary sludge generated will be sent to the sludge holding tank where it will mix with the WAS prior to thickening, anaerobic digestion and dewatering. The BioWin process flow schematic is presented in Figure 7-9.

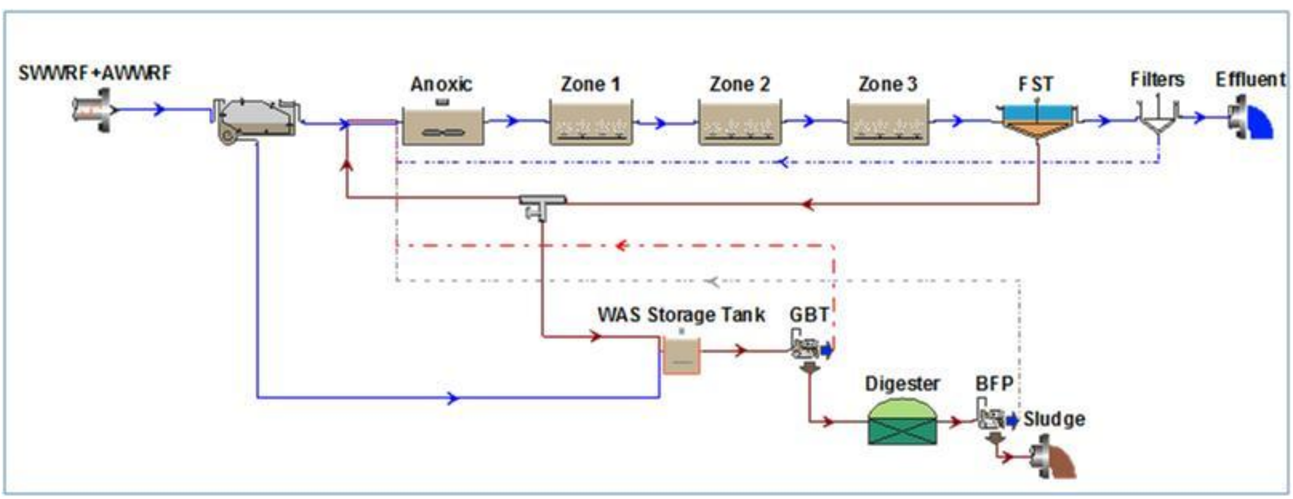


Figure 7-9. BioWin Process Flow Schematic for Alternative 2

For the purpose of this analysis, it was assumed that the new primary clarification system be located in the area of the property where the “old plant” currently sits. Based on preliminary calculations, two 100-foot diameter circular clarifiers will be required. For modeling purpose and based on data collected at bench-scale at the SWWRF, the potential TSS removal efficiency of the primary clarifiers, based on the wastewater characteristics, ranges from 60 to 72 percent. For modeling purposes, an average removal efficiency of 65 percent was selected.

The BioWin process model was used to estimate the average mixed liquor concentration in the aeration basins if primary clarification were in place at the SWWRF and the results are summarized in Figure 7-10. Because of the effectiveness of the primary clarifiers in removing the organic loads to the activated sludge process, the average mixed liquor concentration will be reduced from 3,600 mg/L to approximately 2,150 mg/L. This value is comparable to the mixed liquor reduction achieved in Alternative 1 by the step-feed configuration.

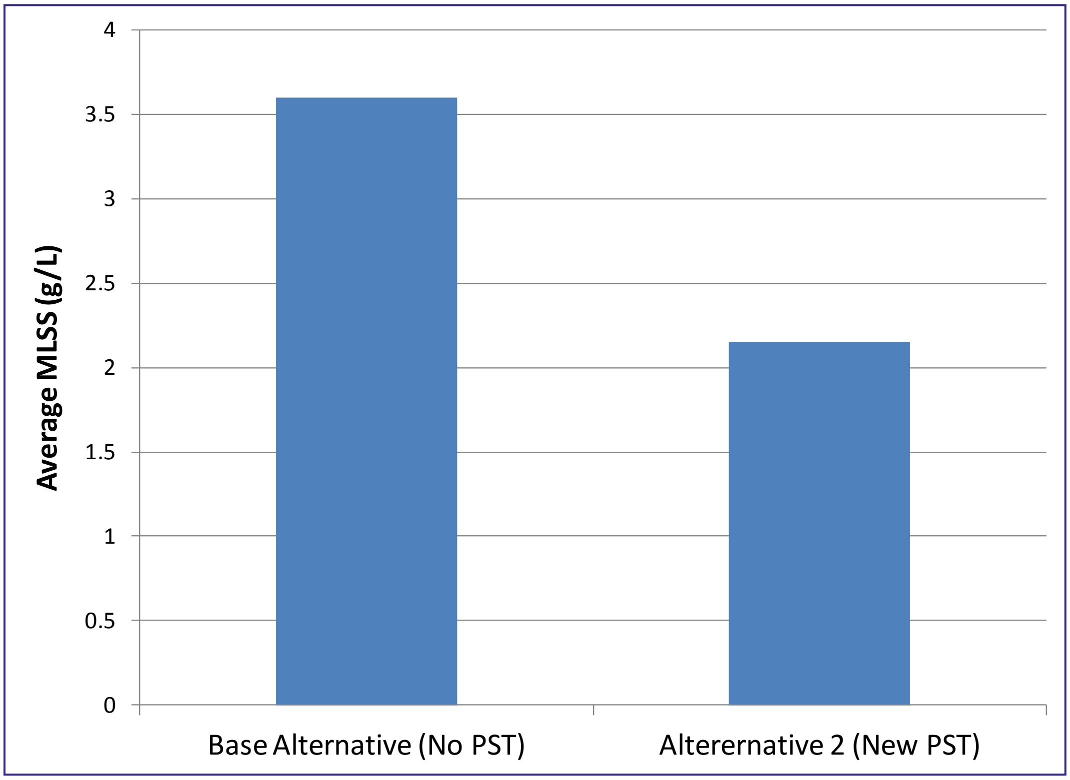


Figure 7-10. Effect of primary clarifiers on the average mixed liquor concentration for Alternative 2

Table 7-7 presents the summary of total aeration requirements for Alternative 2. This shows the existing aeration system has adequate capacity to handle the aeration demands at 20 MGD for carbonaceous only removal. In the case that nitrification is present; the existing aeration system will have adequate capacity to provide aeration for average conditions but not during peak aeration demands. Additional blower capacity and fine bubble diffusers will be required to handle the rated capacity if nitrification occurs at the SWWRF.

Table 7-7. Summary of Aeration Requirements for Alternative 3

Parameter	Carbonaceous Only Removal	Nitrification and Carbonaceous Removal
Zone volume, MG	3.426 (aerobic volume)	3.426 (aerobic volume)
Sidewater Depth, ft	17.0	17.0
AOR (average load/ peak load), lb O/d	1,010/ 1,585	1,800/ 2,850
DO (average load/ peak load), mg/L	2.0/ 1.0	2.0/ 1.0
Average Alpha	0.50	0.50
Average SOTE, %	33.1	32.9
Average Air Flow, scfm	8,000	13,200
Maximum Air Flow, scfm	11,150	20,050
Diffuser Type	Fine Bubble Diffuser	Fine Bubble Diffuser
Diffuser Number	7,000	10,700
Diffuser density, At/Ad	10.51	6.95

For this alternative, the capacity of the secondary clarification system is approximately 20 MGD for a design SVI of 120 mL/g as presented previously in Alternative 1.

7.2.3 Alternative 2: New Secondary Clarifiers

This alternative modifies the existing configuration at the SWWRF by providing additional secondary clarification capacity. Based on modeling results, one additional 135-foot diameter clarifier is required to increase the capacity of the SWWRF to 20 MGD. In addition, the RAS capacity of the plant needs to be increased from 18 MGD to 24 MGD or 6 MGD per clarifier.

7.3 Conclusions/Recommendations

Table 7-8 provides a summary of the results of the SWWRF simulations conducted for this plan with respect to treatment process assessment of the SWWRF. This table is used for comparison of results and provides a reference for the specific conclusions listed below for the SWWRF.

Table 7-8. Summary of the SWWRF Simulation Results				
Parameter	Base Scenario	Alternative 1	Alternative 2	Alternative 3
Operation Mode	Existing	Step Feed	Existing	Existing
Primary Clarifiers				
Number of Units	n/a	n/a	2	n/a
TSS Removal Efficiency, %	n/a	n/a	65	n/a
Aeration Basins				
Number of Units	2	2	2	2
Total Reactor Volume, MG	4.56	4.56	4.56	4.56
Average MLSS, mg/L	3,600	2,200	2,150	3,600
Total SRT, days	2.5	2.5	2.5	2.5
Aeration System	Fine bubble	Fine bubble	Fine bubble	Fine bubble
Number of Diffusers	7,000	7,000	7,000	7,000
Avg Airflow Rate, scfm	13,250	11,660	8,000	13,250
Peak Airflow Rate, scfm	16,600	17,650	11,150	16,600
Secondary Clarifiers				
Number of Units	3	3	3	4
Total Area, ft2	42,942	42,942	42,942	57,255
SVI, mL/g	120	120	120	120
RAS capacity, mgd	18	18	18	24
Peak SOR, gpd/ft2	792	931	931	700
Peak SLR, lbs/ft2.d	36.3	33.8	33.8	33.5
Deep Bed Multimedia Filters				
Number of Units	5	5	5	5
Total Filtration Area, ft^2	7,030	7,030	7,030	7,030
Filtration rate at peak hour flow, gpm/ft^2	8.0	8.0	8.0	8.0
Disinfection				
Number of CCT Tanks	2	2	2	2
Total Volume, ft^3	63,448	63,448	63,448	63,448
Detention Time at peak hour flow, min	34	34	34	34

Section 8

Solid Stream Process Evaluations

This section provides a summary of the results of solid stream process evaluations for the SWWRF. The main objective of the solids handling process assessment was to predict the amount of solids produced at the facility for the alternatives evaluated in Section 5. In addition, historical solids production data was evaluated in order to validate flow and solids data predicted by the simulation. Ultimately, the solids mass predictions will be used for the treatment process assessment of the existing solid process units at the SWWRF.

8.1 Sludge Production Rates

The calibrated process model was used to estimate the future sludge production rates at the SWWRF at a rated capacity of 20 MGD. Table 8-1 summarizes the predicted solid production rate of each alternative.

Table 8-1. Estimated Solid Production Rates

Condition	Base Option and Alternatives 1 and 3		Alternative 2	
	Primary Sludge (lb/d)	WAS (lb/d)	Primary Sludge (lb/d)	WAS (lb/d)
Average	n/a	35,000	25,000	15,000
Max Month	n/a	45,000	30,000	25,000
Max Day	n/a	60,000	43,400	31,700

8.2 Solid Stream Mass Balance

Predicted solid production rates presented in Table 8-1 were used to conduct mass balance analysis for the solid stream processes. Some assumptions have to be made in the calculations based on the historical data, wastewater characterization study and the information retrieved from previous reports. These assumptions are listed in Table 8-2.

Table 8-2. Assumptions used for Solid Mass Balances

Parameter	Assumption	Comment
Primary Sludge Solid Content	2.0% solids	Based on typical primary sludge solid content – for Alternative 2
GBT Solid Content	5% solids	Assumed based on historical data
GBT Removal Efficiency	90% based on TSS	Value from 2001 B&V Master Plan
GBT Filtrate Solid Content	0.1% solids	Model prediction for Condition 1
Anaerobic Digested Sludge Solid Content	2.5%	Based on historical data
Anaerobic Digested Sludge VSR	40%	Based on historical data for base option and Alternative 1
	60%	Assumption for Alternative 2 based on model prediction
BFP Filtrate Solid Content	0.75%	Model prediction
BFP Removal Efficiency	95%	Value from 2001 B&V Master Plan
Cake Solid Content	15%	Based on historical data

The mass balance results are summarized in Table 8-3 for the maximum month condition. Overall, the mass balance would be used to evaluate the capacity of the existing solid stream unit processes and to determine the upgrades required to handle additional flows and loads.

Table 8-3. Mass Balance for Solid Processes at Average Solid Production Rate

Parameter		Unit	Base Scenario & Alternatives 1 and 3	Alternative 2
Primary Sludge				
	Flow	gpm	n/a	125
	Solid Mass	lb/d	n/a	30,000
	% Solids	%	n/a	2.00
WAS				
	Flow	gpm	535	297
	Solid Mass	lb/d	45,000	25,000
	% Solids	%	0.70	0.70
GBT Sludge				
	Flow	gpm	67	82
	Solid Mass	lb/d	40,500	49,500
	% Solids	%	5	5
	Removal Efficiency	%	90	90
GBT Filtrate				
	Flow	gpm	467	340
	Solid Mass	lb/d	4,500	5,500
	% Solids	%	0.10	0.10
Anaerobic Digested Sludge				
	Flow	gpm	67	91
	Solid Mass	lb/d	28,350	27,225
	% Solids	%	2.50	2.50
	VS Red	%	40%	60%
BFP Cake (Final Cake)				
	Flow	gpm	15	14
	Solid Mass	lb/d	26,933	25,850
	% Solids	%	15.00	15.00
	Removal Efficiency	%	95%	95%
	Solid Production	wet tons/d	89.8	86.2

8.3 Treatment process assessment of Solid Handling Facilities

Solids treatment and handling at the SWWRF consist of four main unit processes: sludge holding tank, gravity belt thickening, anaerobic digestion and belt filter press dewatering. The capacity of each of these unit processes was evaluated using the information provided in Tables 8-1, 8-2 and 8-3. Table 8-4 summarizes the limiting factors for each process units.

The holding tank currently provides a temporary storage for WAS prior to thickening in order to decrease the number of hours required to operate the thickening operation. It is recommended to have 12 hours of storage if it is aimed to operate the GBT's only two shifts per day.

One two-meter GBT is used to thicken WAS from approximately 1 percent to 5 percent solids before anaerobic digestion. The process currently operates 12 hours per day for 7 days per week. Recommended loadings for GBT are limited to a maximum solids loading rate (SLR) of 1,000 pph/m of belt width with maximum hydraulic loading rate (HLR) of 250 gpm/m belt width.

Table 8-4. Limiting Factors for Solid Handling Facilities

Process	Description	Limiting Factor
Holding Tank	110,000 gal capacity, one tank	12 hour detention
Gravity Belt Thickener	1 unit , 2-m belt	SLR=1,000 pph/m of belt
	12 hrs and 7 days of operation	HLR=250 gpm/m of belt
Anaerobic Digesters	3 tanks (2 online)	Min SRT = 15 days
	100 ft diameter	
	max water level=22.5 ft - min water level=14 ft	
	Volume=1.3 MG/each	
Belt Filter Presses	2 units, 2 m belt	SLR=700 pph/m of belt
	14 hrs and 4 days of operation	HLR=75 gpm/m of belt

In the SWWRF, thickened WAS is stabilized through mesophilic anaerobic digestion in order to meet Class B pathogen requirements. Based on the regulations, the detention time in the digesters must be a minimum of 15 days at 35°C. The SWWRF has three anaerobic digestion tanks; however, only two tanks are currently operational and in service. The third tank was taken off-line due to structural concerns. Each 100 ft diameter digesters are equipped with floating covers, draft tube mixing and heating. The maximum volume of each digester is approximately 1.3 MG.

The facility has two 2-meter BFP's to remove the water from digested biosolids and to produce a sludge cake. Historical data indicates that currently the digested biosolids at the SWWRF has approximately 2.5% solid and the final dewatered cake has a solids concentrations of approximately 15% solids. The BFPs are currently operated 14 hours per day and 4 days per week. BFP's are limited to a maximum SLR of 700 pph/m of belt width and maximum HLR of 75 gpm/m of belt width.

8.3.1 Base Option and Alternatives 1 and 3

For base option and Alternatives 1 and 3, the detention time in the sludge holding tank would be around 3.9 hours at maximum month production rates. Although not limiting the capacity, additional sludge storage volume might be necessary if the facility would like to operate the GBT more efficiently. Table 8-5 summarizes the operating conditions of existing process units for the Base Option and Alternatives 1 and 3 at average, max month, and max day solid production rates.

At the rated capacity of 20 MGD, the solids and hydraulic loadings to the existing single GBT at the SWWRF will exceed the limiting SLR and HLR values presented in Table 8-4 based on the current operation of 12 hrs per day, 7 days per week. Based on these results, the existing GBT has a capacity to handle future sludge production up to 11.5 MGD as annual average flow. Since it is not prudent to increase to increase the time that the GBT is operation, Brown and Caldwell recommends adding an additional 2-meterGBT.

Table 8-5 shows the anaerobic digesters' detention time for annual average, maximum month and maximum day solid production with two units operational. Based on these results, the existing anaerobic digesters have adequate capacity to handle the projected sludge production at the SWWRF.

As Table 8-5 indicates, the solids and hydraulic loading rates to the existing two BFPs at the SWWRF will exceed the recommended values presented in Table 8-4 at 20 MGD capacity based on current operation of 14 hours, 4 days per week. Based on the data presented in Table 8-4, the existing BFPs have adequate capacity to handle the secondary sludge production up to 16 MGD as annual average flow if the existing schedule operation is maintained. In order to increase the capacity of the BFPs at the SWWRF, Brown and Caldwell recommends increasing the operation time to 12 hours for 6 days per week. This change will allow the existing BFP to handle the secondary sludge produce at 20 MGD as annual average flow.

Table 8-5. Operating Conditions of Existing Units at 20 MGD AADF for Base Option and Alternatives 1 and 3

	Annual Average	Max Month	Max Day
Holding Tank			
Detention Time, hr	6.3	4.9	3.7
GBT (operated 12 hrs X 7 days/week)			
SLR, pph/m of belt width	1,458	1,875	2,500
HLR, gpm/m of belt width	291	375	500
Anaerobic Digestion (2 units on-line)			
SRT,days	25	20	19
BFP (operated 14 hrs x 4 days/week)			
SLR, pph/m of belt width	689	886	1,181
HLR, gpm/m of belt width	55	71	94

8.3.2 Alternative 2

Table 8-6 reviews the operating conditions of existing units for Alternative 2 when primary clarifiers are included to the existing plant.

For Alternative 2, the detention time in the sludge holding tank would be around 4.0 hours at maximum month production rates. Similar to the previous scenario, this limited detention time will not limit the capacity of the plant.

At the rated capacity of 20 MGD, the solids and hydraulic loadings to the existing single GBT at the SWWRF will exceed the limiting SLR and HLR values. Brown and Caldwell recommends adding an additional 2-meterGBT. The existing anaerobic digesters have adequate capacity to handle the projected sludge production at the SWWRF. As Table 8-6 indicates, the solids and hydraulic loading rates to the existing two BFPs at the SWWRF will exceed the recommended values. Brown and Caldwell recommends increasing the operation time to 12 hours for 6 days per week.

Table 8-6. Operating Conditions of Existing Units at 20 MGD AADF for Alternative 2			
	Annual Average	Max Month	Max Day
Holding Tank			
Detention Time, hr	5.50	4.00	3.0
GBT (operated 12 hrs X 7 days/week)			
SLR, pph/m of belt width	1,667	2,292	3,129
HLR, gpm/m of belt width	333	458	625
Anaerobic Digestion (2 units on-line)			
SRT, days	27	18	13
BFP (operated 14 hrs x 4 days/week)			
SLR, pph/m of belt width	619	967	1,320
HLR, gpm/m of belt width	49	77	106

8.4 Conclusions/Recommendations

The treatment process assessment of the solids processing facilities at the SWWRF indicated that the existing GBT will limit the overall capacity of the plant at 11.5 MGD. The sludge holding tank would have very short detention times which might eliminate the function of the tank; however, the holding tank is not considered to be a "real" limiting process for the SWWRF. The existing anaerobic digesters (two units) have adequate capacity to handle the projected sludge values presented in Table 8-1.

Based on the results presented herein, at least one additional GBT unit must be installed and the operating schedule of the GBTs be extended in order to handle the projected sludge production at 20 MGD.

Table 8-7 summarizes recommended action items to handle 20 mgd flow and loading for each alternative.

Table 8-7. Recommended Modifications for Solid Handling Facilities to Treat 20 MGD AADF		
Solid Handling Process	Base Scenario and Alternatives 1 and 3	Alternative 2
GBT	Install one 2-m belt width GBT and operate continuously	Install one 2-m belt width GBT and operate continuously
Anaerobic Digesters	No action necessary	No action necessary
BFP	Change operating schedule to 14 hours for 7 days/week	Change operating schedule to 12 hours for 7 days/week

It must be noted that the evaluations presented above includes no consideration of redundancy. It is recommended to take account of the redundancy requirements especially for thickening and dewatering processes since these units are required to be in operation for long hours-almost continuously. At least one thickening and one dewatering unit or additional storage capacity (likely upstream of the GBTs) should be considered for redundancy purposes.

Section 9

Summary and Recommendations of the Treatment Process Assessment

This section provides a summary of the results of treatment process assessment of the SWWRF. A combination of historical data analysis, special sampling data collection, process modeling, and mass balances were used to assess the treatment capacity of the liquid and solids processing units at the SWWRF to investigate the capabilities of this plant to handle the combined flows from the SWWRF's and AWWRF's service areas.

9.1 Summary of Treatment process assessment of the SWWRF

The results of the treatment process capacity evaluation are summarized in Figure 9-1. The treatment capacity values presented in this figure do not include the potential capacity of the "old plant", which is currently permitted for 4 MGD AADF. For the purpose of this analysis, it was assumed that the existing mechanical aerators in the “new plant” were replaced by a fine-bubble aeration system as recommended in the 2009 PDR prepared by Boyle Engineering Corporation.

The hydraulic modeling evaluation of the SWWRF shows the facility to be capable of hydraulically passing the projected peak hour flow of 40 MGD if all the process units present in the “new plant” are operational. In the case that one secondary clarifier is out of service, the hydraulic capacity of the plant will reduce to approximately 37 MGD PHF. At flows exceeding 37 MGD, wastewater would still pass through the plant; however, the Clarifier weirs would be submerged, increasing the amount of solids passing into the secondary effluent and carrying into the filters.

The treatment process evaluation indicates that the existing treatment capacity of "new plant" facilities is limited by the secondary clarification system at approximately 17 MGD AADF. The aeration system (for carbonaceous removal only), filtration and disinfection facilities have treatment capacities exceeding 20 MGD AADF. In the case that nitrification occurs at the facilitiy, the capacity of the aeration system will be limited to 15 MGD AADF.

Evaluation of the sludge handling facilities at the 20-MGD design flow showed that the WAS holding tank and the gravity belt thickener apparently limit the overall plant capacity. The WAS holding tank is used to minimize the operating schedule of the GBT, and if the GBT hours of operation are increased, the WAS holding requirements decrease. Consequently, the W AS holding tank is not considered to be a true limiting process. The limiting process is the GBT. Based on the results of this plan, the treatment capacity of the GBT will be exceeded when the average flow reaches approximately 11.5 MGD based on a schedule of 12 hours per day, 7 days week. Even though, the treatment capacity of the GBT could be increased by extending the run time of the unit, it was not prudent to assume this since it is already almost continuously. The existing anaerobic digesters (two units) and BFPs have adequate capacity to stabilize and dewatered the thickened sludge produced at the plant when influent flow reaches 20 MGD AADF.

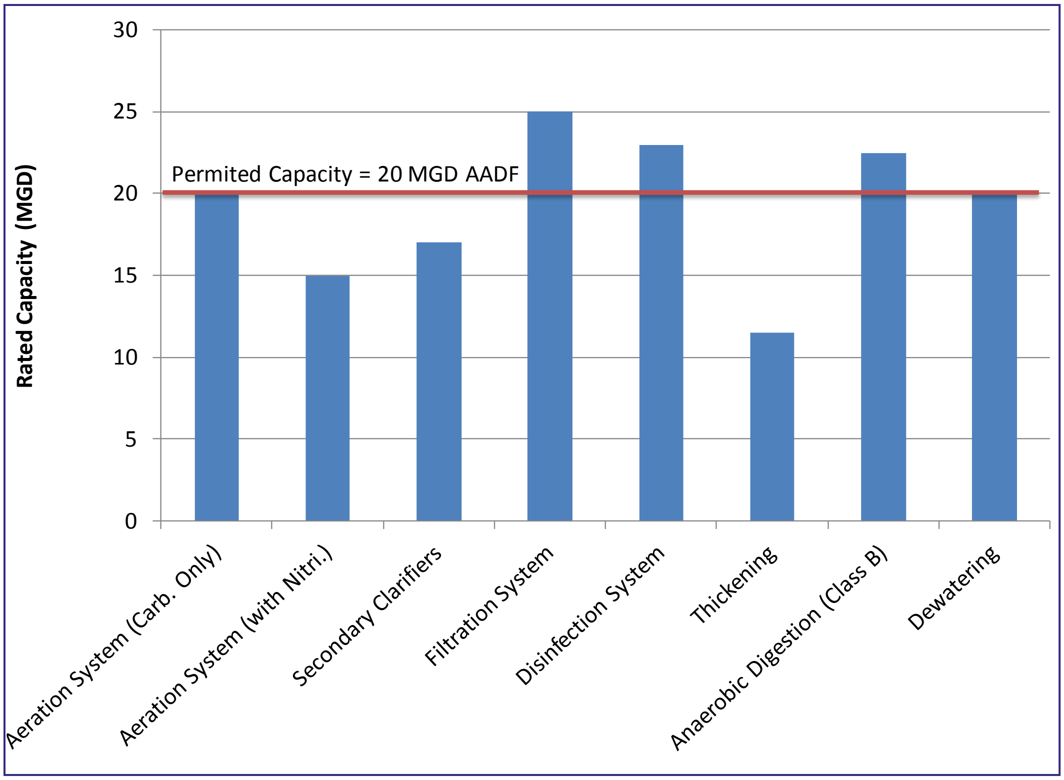


Figure 9-1. Summary of Treatment Process Capacity Evaluation Results

9.2 Recommendations

Based on the treatment process results presented in this plan, the treatment capacity of the facility is limited by the secondary clarification capacity of the "new plant" at 17 MGD AADF. The treatment capacity is limited by the combination of the projected peak hour flow and the high mixed liquor concentration. Therefore, treatment process alternatives were evaluated to reduce the operating mixed liquor in the aeration basins. Two possible treatment modifications were analyzed, being the step-feed process and the addition of a new primary clarification system. Both alternatives will increase the capacity of the existing secondary clarification process at the SWWRF. The addition of new primary clarifiers will not only increase the capacity of the plant but it will significantly reduce the aeration requirements in the biological process and will increase the biogas production in the anaerobic digesters. Therefore, even though the addition of the new primary clarifiers might have a larger capital investment, it will reduce the overall costs associated with operation by decreasing the aeration requirements and increasing the biogas production, which could subsequently be used to produce energy at the plant.

The new fine-bubble aeration system was designed for carbonaceous removal only. Brown and Caldwell recommends effective automatic DO control and online ammonia analyzers be implemented at the SWWRF to control the aeration system. This will provide operational flexibility to minimize nitrification and to produce good-quality settling sludge, which was an important assumption during this plan.

The sludge processing facilities are limited by the GBT. Therefore, one additional GBT is required at the SWWRF to handle the sludge production at flows of 20 MGD AADF.

Appendix A: SWWRF Historical Operational Data from January 2007 through November 2011

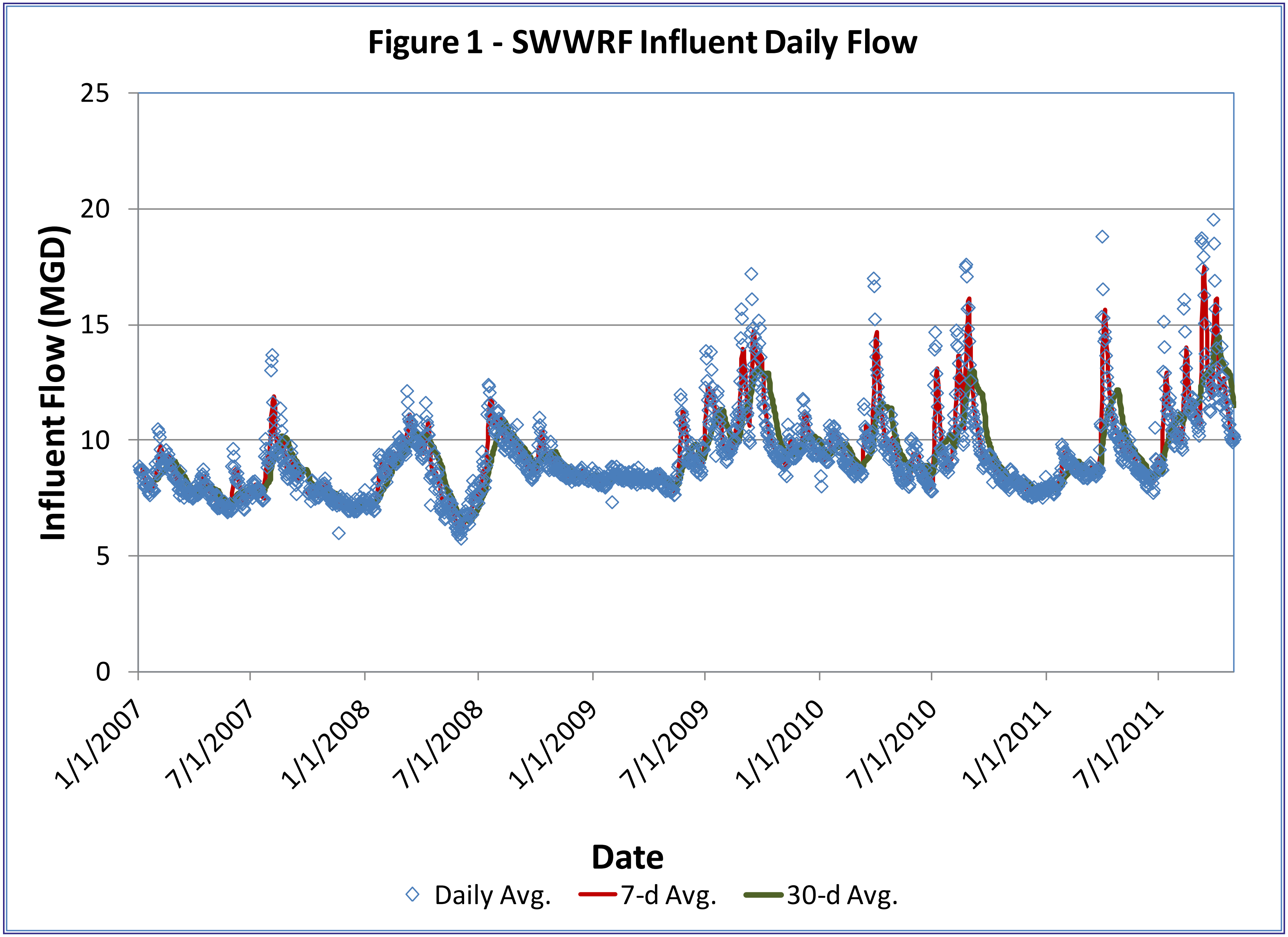


Figure 1 - SWWRF Influent Daily Flow
25
20
15
10
5
0
Influent Flow (MGD)
1/1/2007
7/1/2007
1/1/2008
7/1/2008
1/1/2009
7/1/2009
1/1/2010
7/1/2010
1/1/2011
7/1/2011
Date
Daily Avg.
7-d Avg.
30-d Avg.

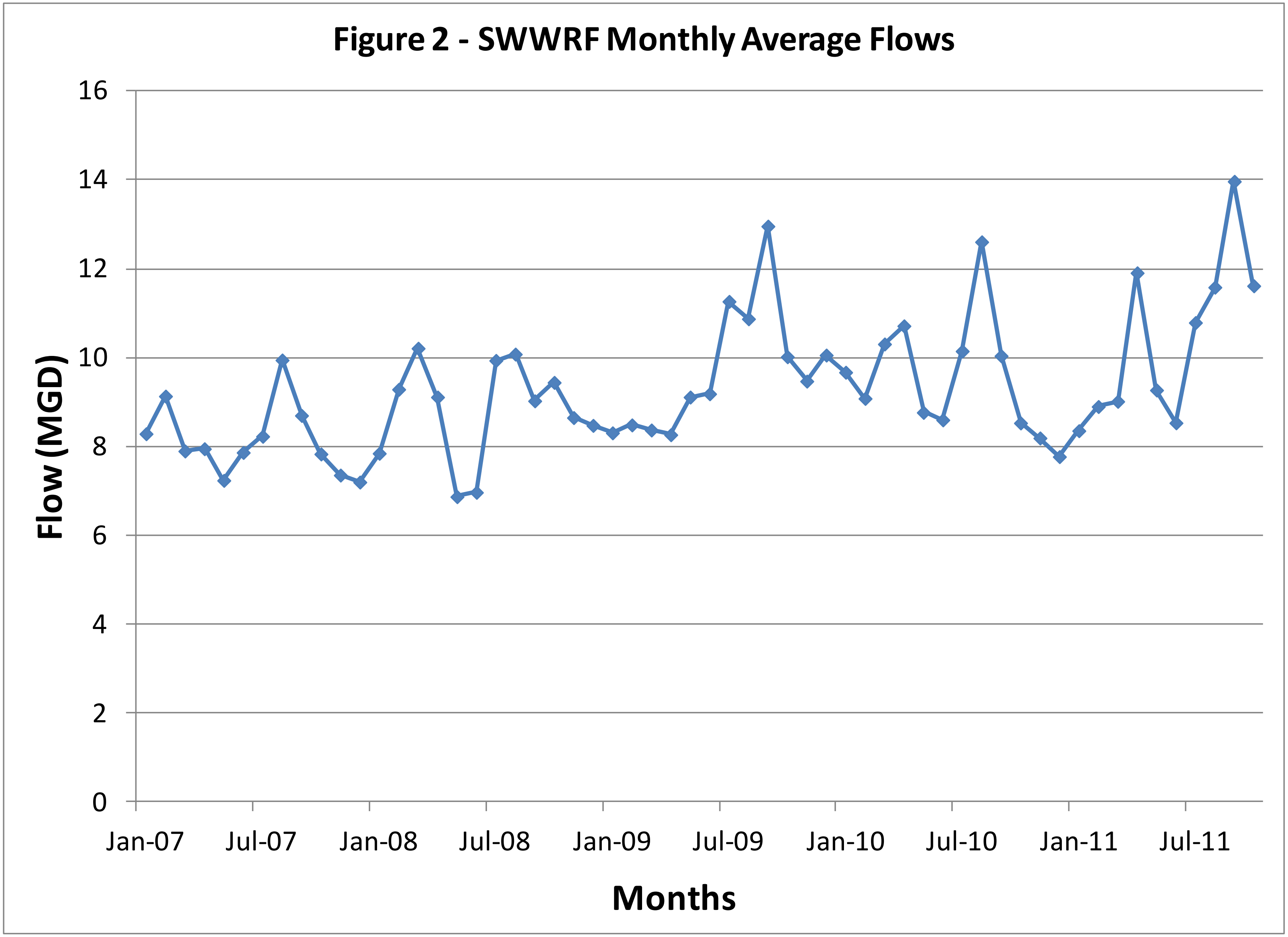


Figure 2 - SWWRF Monthly Average Flows
16
14
12
10
8
6
4
2
0
Flow (MGD)
Jan-07
Jul-07
Jan-08
Jul-08
Jan-09
Jul-09
Jan-10
Jul-10
Jan-11
Jul-11
Months

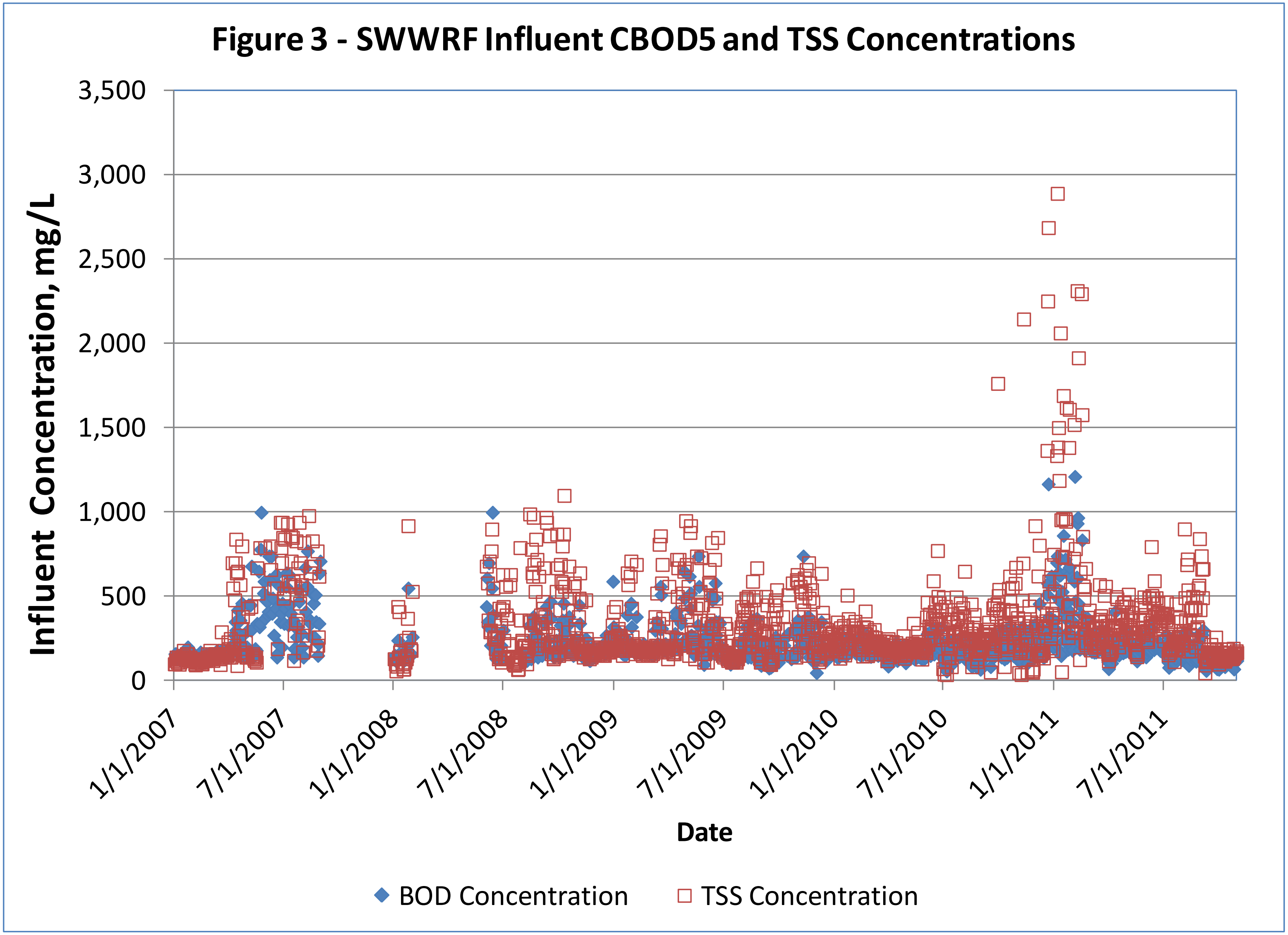


Figure 3 - SWWRF Influent CBOD5 and TSS Concentrations
Influent Concentration, mg/L
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/2007
7/1/2007
1/1/2008
7/1/2008
1/1/2009
7/1/2009
1/1/2010
7/1/2010
1/1/2011
7/1/2011
Date
BOD Concentration
TSS Concentration

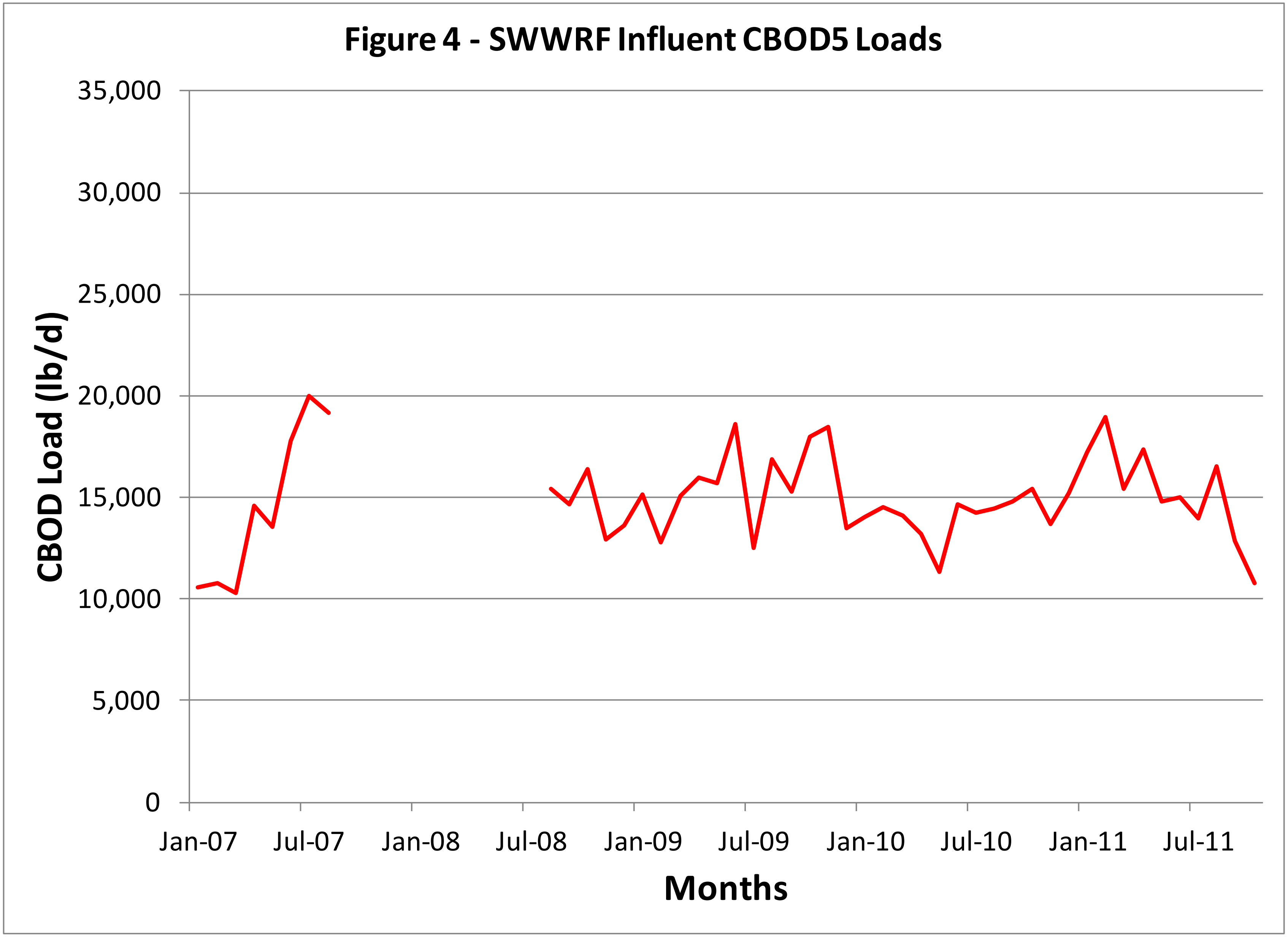


Figure 4 - SWWRF Influent CBOD5 Loads
CBOD Load (lb/d)
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
Jan-07
Jul-07
Jan-08
Jul-08
Jan-09
Jul-09
Jan-10
Jul-10
Jan-11
Jul-11
Months

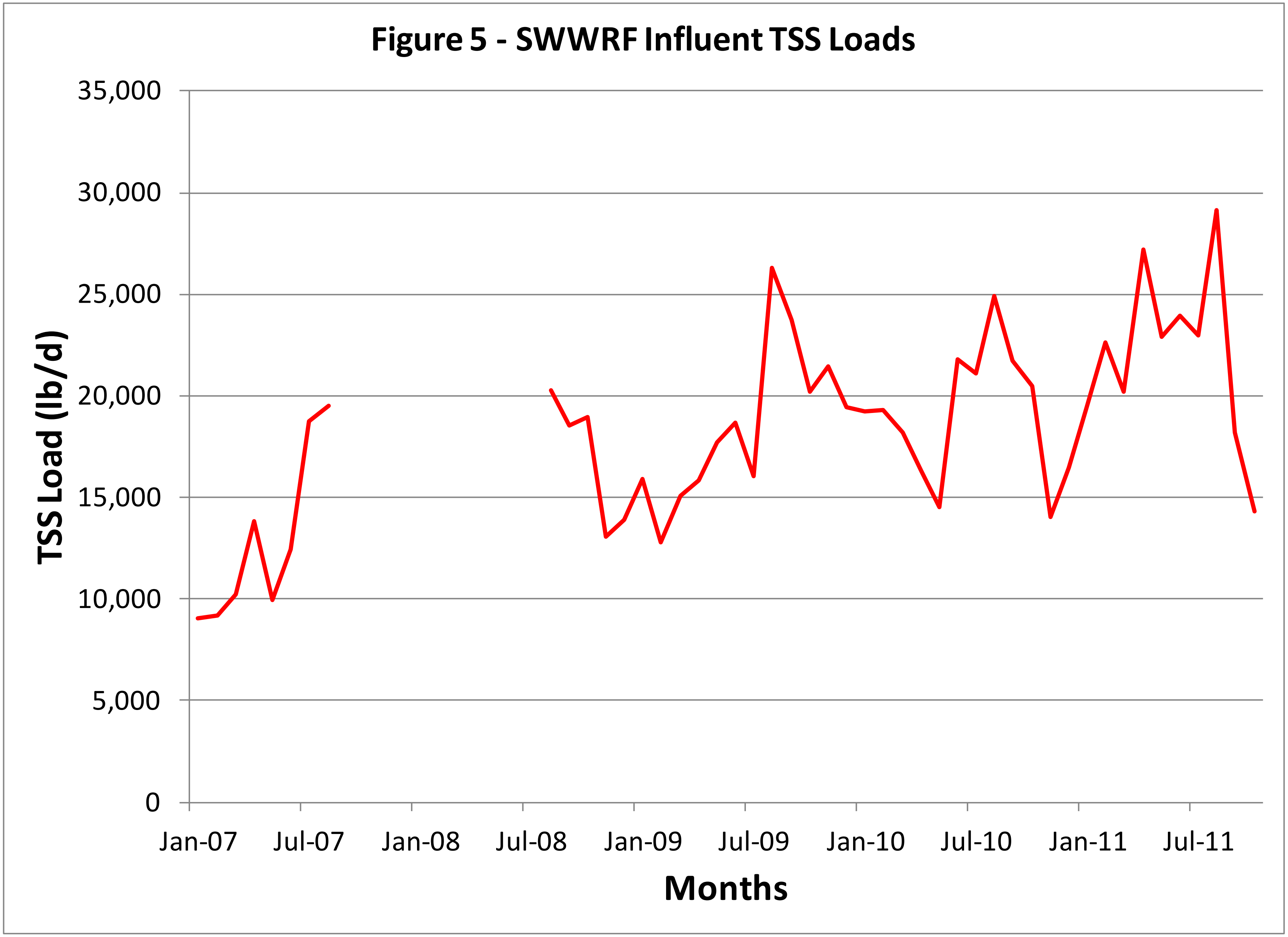


Figure 5 - SWWRF Influent TSS Loads
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
TSS Load (lb/d)
Jan-07
Jul-07
Jan-08
Jul-08
Jan-09
Jul-09
Jan-10
Jul-10
Jan-11
Jul-11
Months

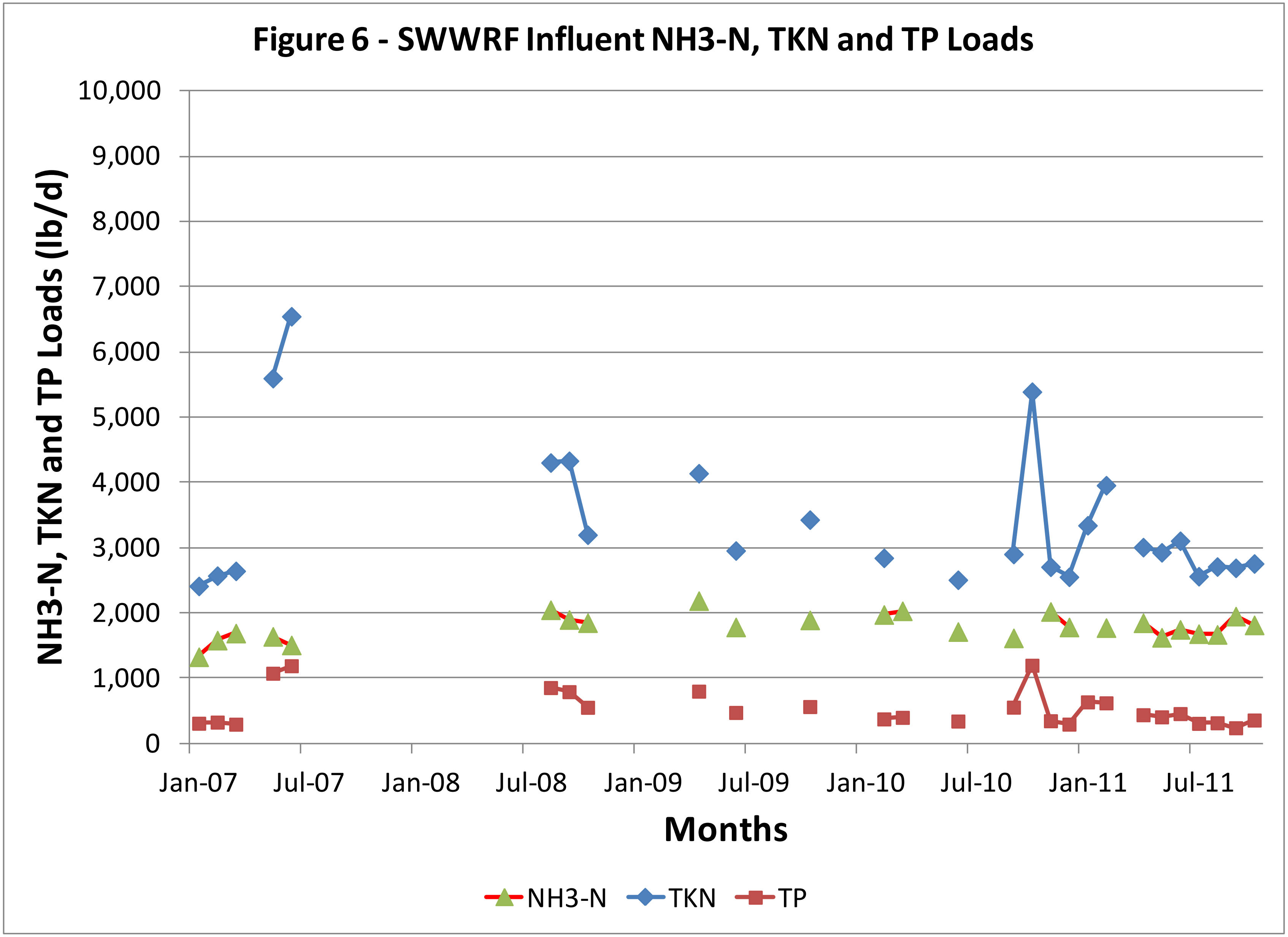


Figure 6 - SWWRF Influent NH3-N, TKN and TP Loads
NH3-N, TKN and TP Loads (lb/d)
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
Jan-07
Jul-07
Jan-08
Jul-08
Jan-09
Jul-09
Jan-10
Jul-10
Jan-11
Jul-11
Months
NH3-N
TKN
TP

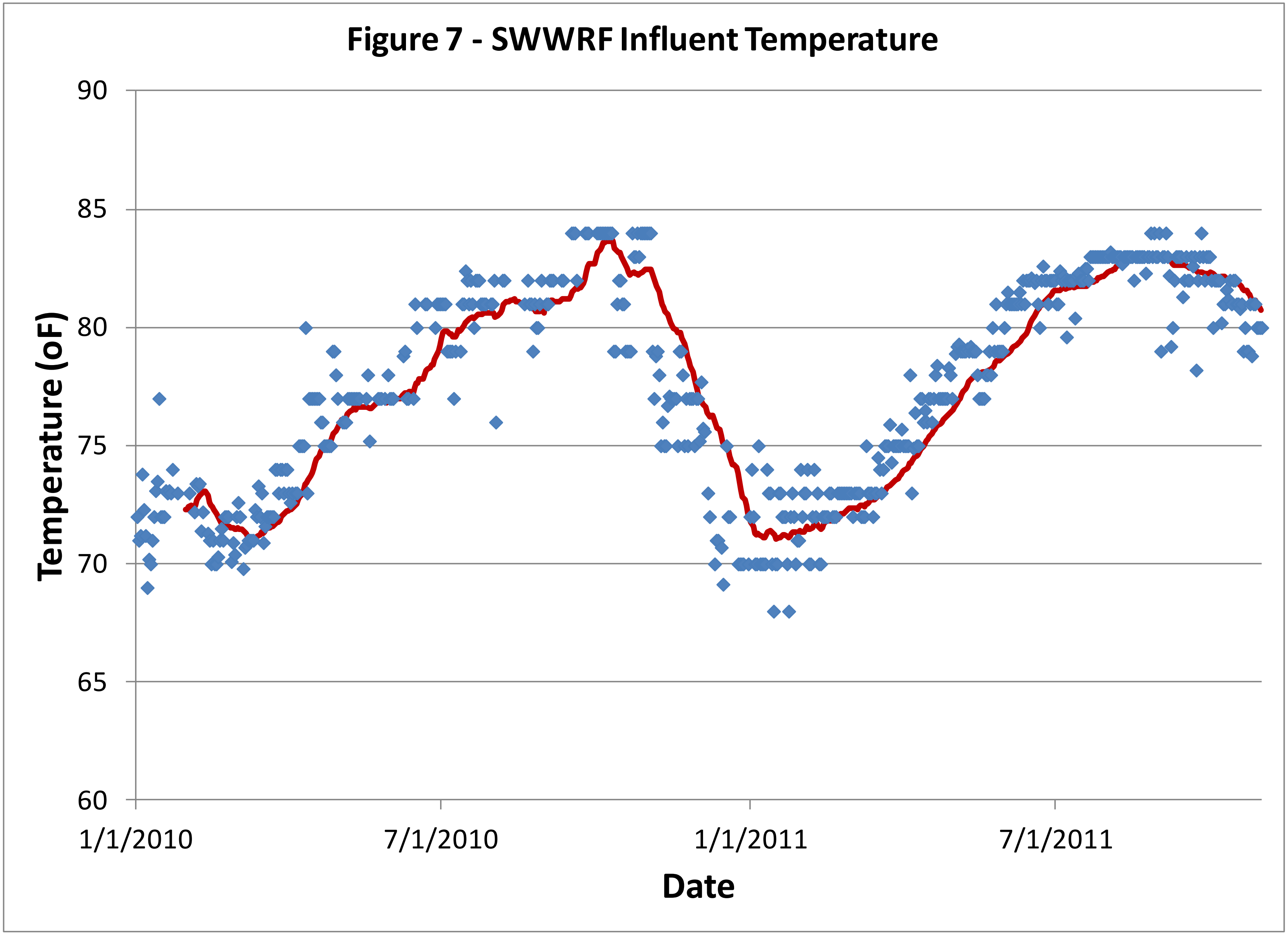


Figure 7 - SWWRF Influent Temperature
Temperature (oF)
90
85
80
75
70
65
60
1/1/2010
7/1/2010
1/1/2011
7/1/2011
Date

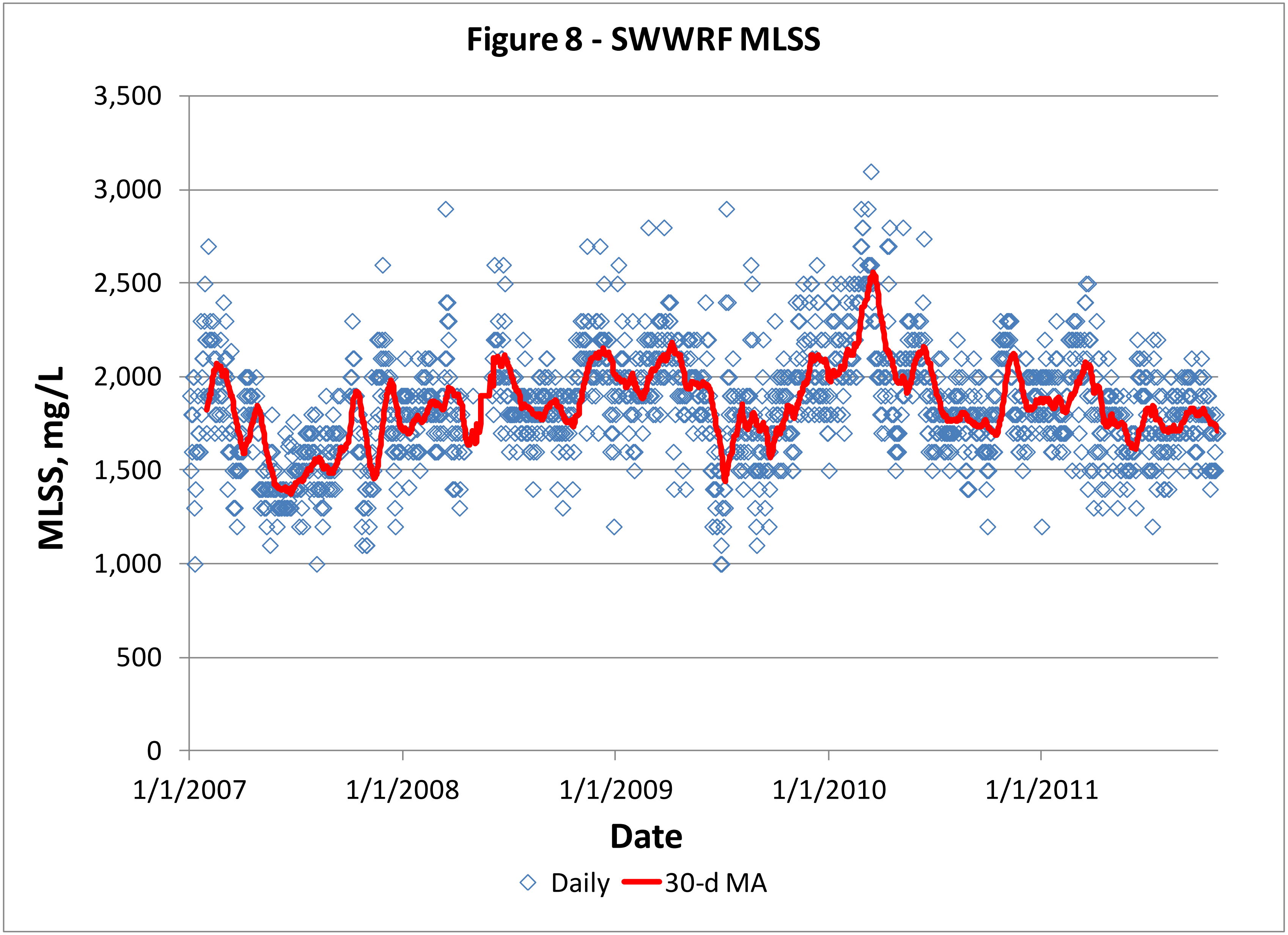


Figure 8 - SWWRF MLSS
MLSS, mg/L
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/2007
1/1/2008
1/1/2009
1/1/2010
1/1/2011
Date
Daily
30-d MA

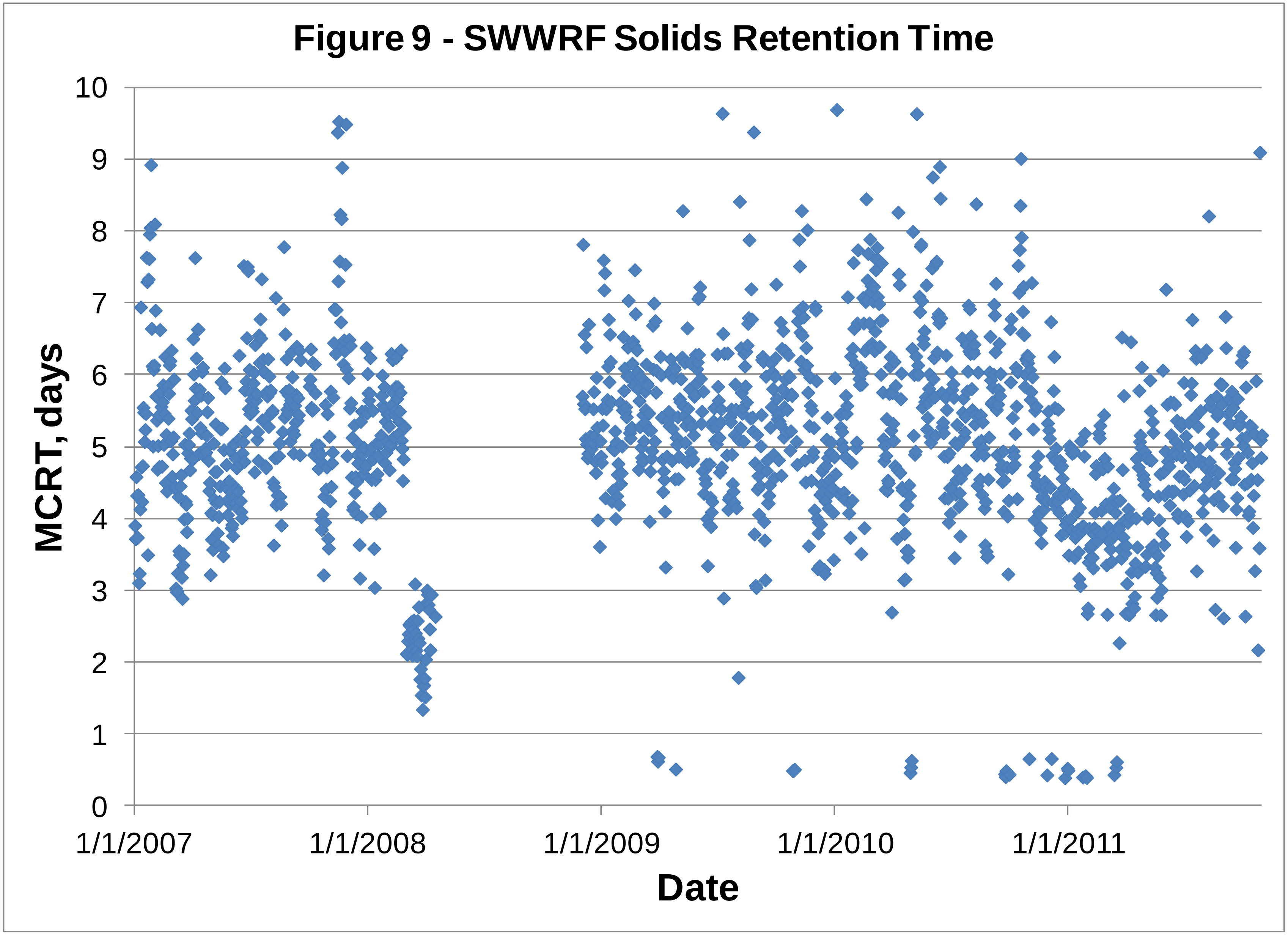


Figure 9 - SWWRF Solids Retention Time
MCRT, days
Date
0
1
2
3
4
5
6
7
8
9
10
1/1/2007
1/1/2008
1/1/2009
1/1/2010
1/1/2011

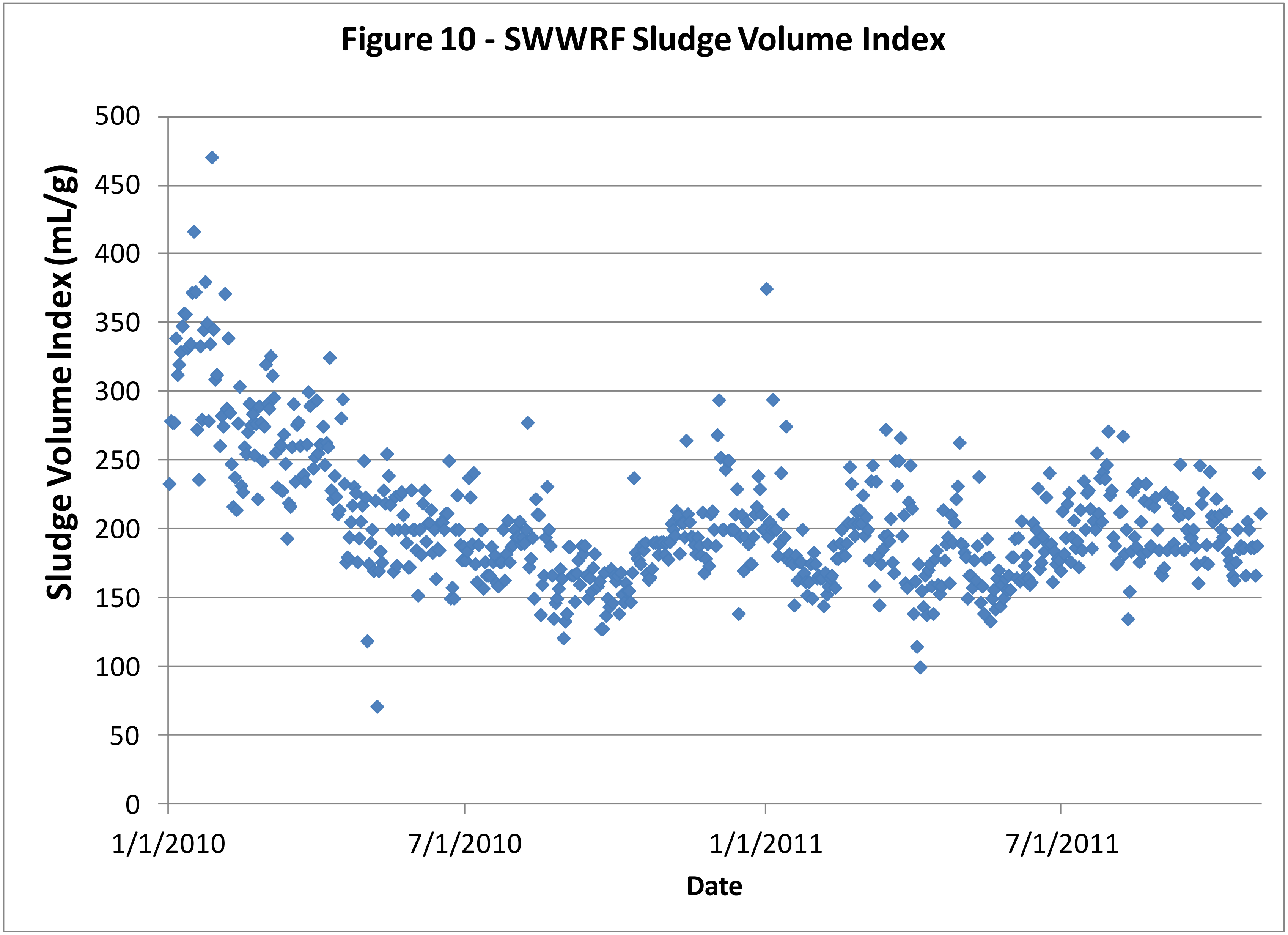


Figure 10 - SWWRF Sludge Volume Index
Sludge Volume Index (mL/g)
500
450
400
350
300
250
200
150
100
50
0
1/1/2010
7/1/2010
1/1/2011
7/1/2011
Date

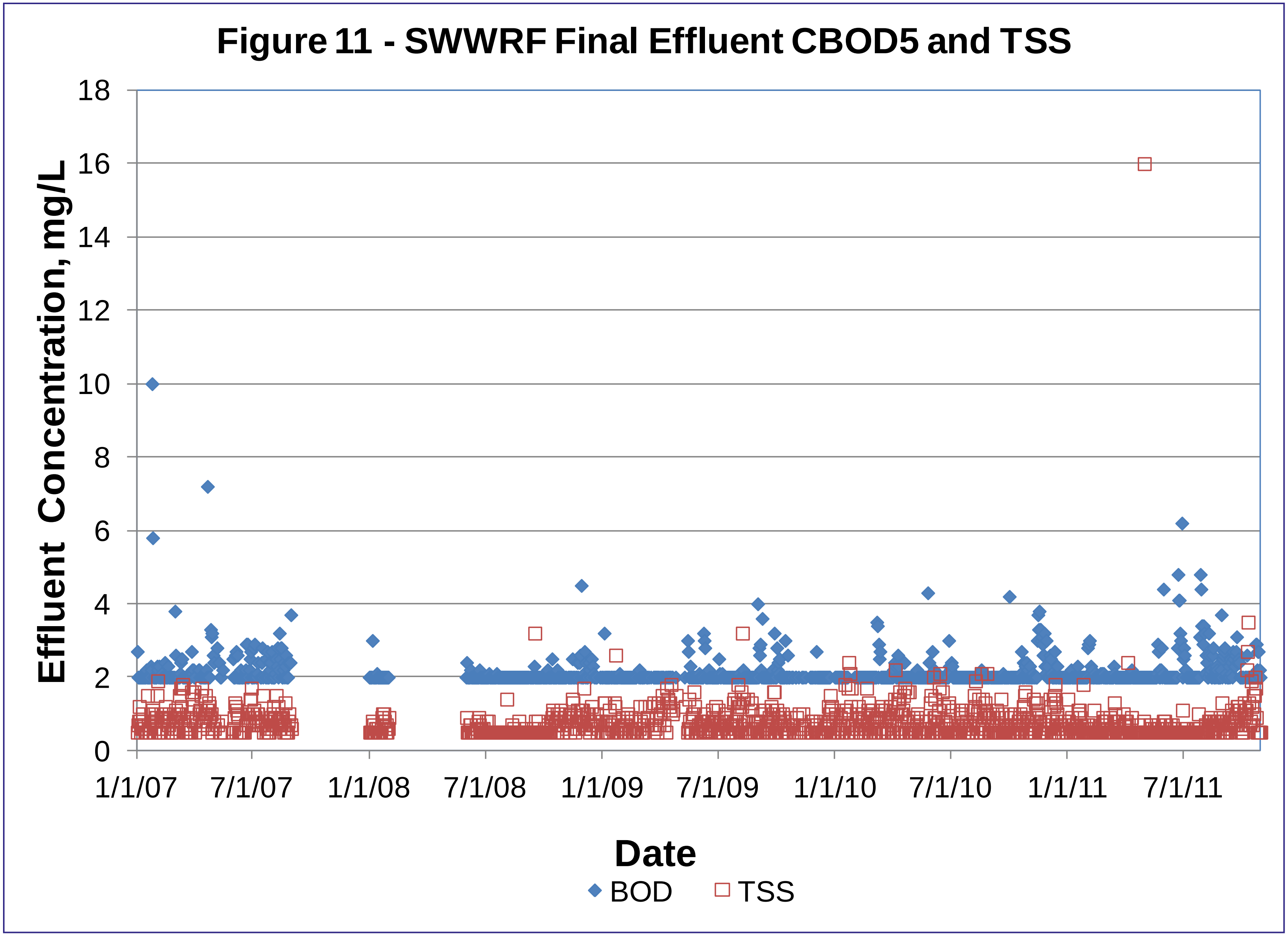


Figure 11 - SWWRF Final Effluent CBOD5 and TSS
Effluent Concentration, mg/L
0
2
4
6
8
10
12
14
16
18
1/1/07
7/1/07
1/1/08
7/1/08
1/1/09
7/1/09
1/1/10
7/1/10
1/1/11
7/1/11
Date
BOD
TSS

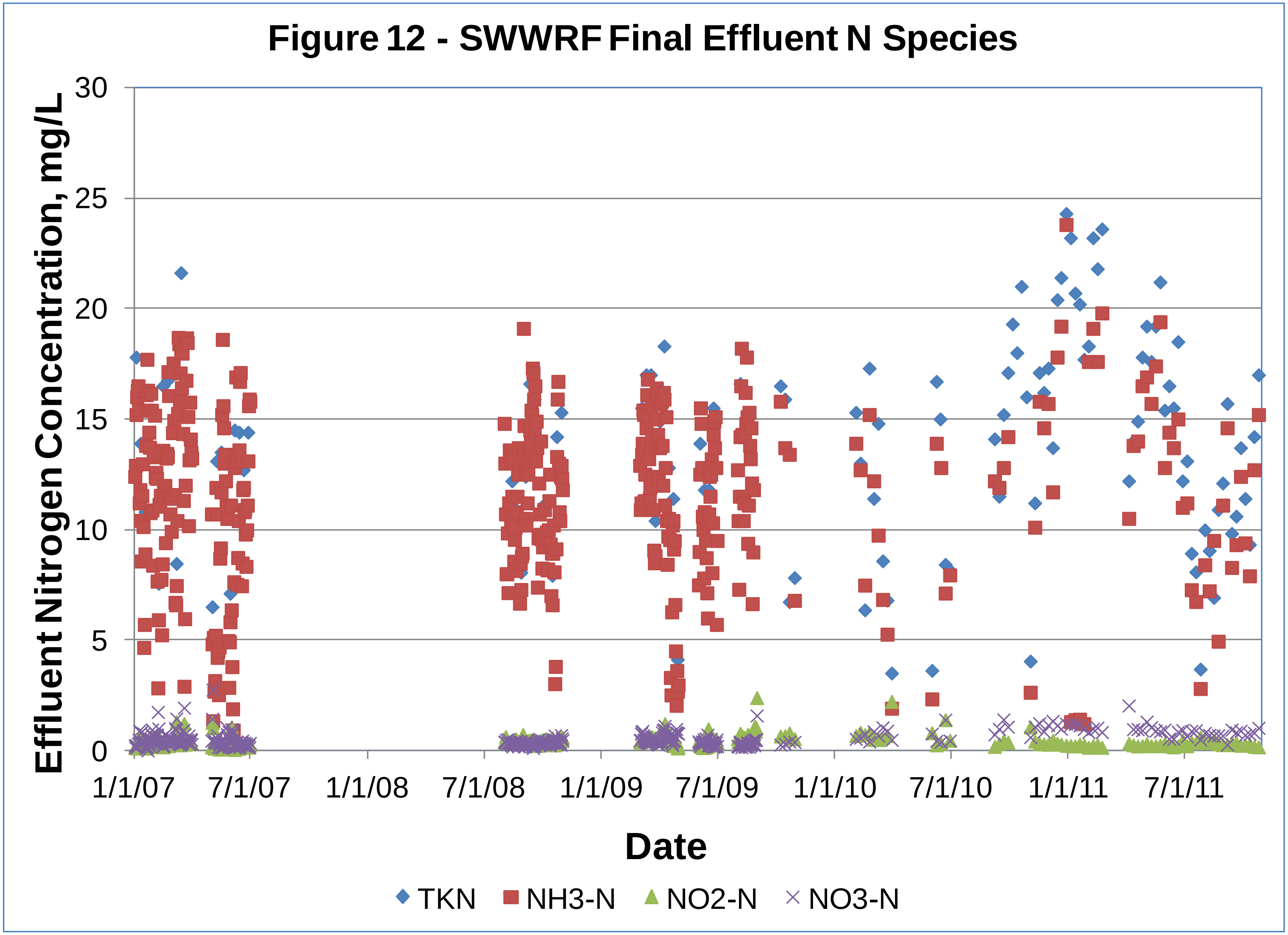


Figure 12 - SWWRF Final Effluent N Species
Effluent Nitrogen Concentration, mg/L
30
25
20
15
10
5
0
1/1/07
7/1/07
1/1/08
7/1/08
1/1/09
7/1/09
1/1/10
7/1/10
1/1/11
7/1/11
Date
TKN
NH3-N
NO2-N
NO3-N

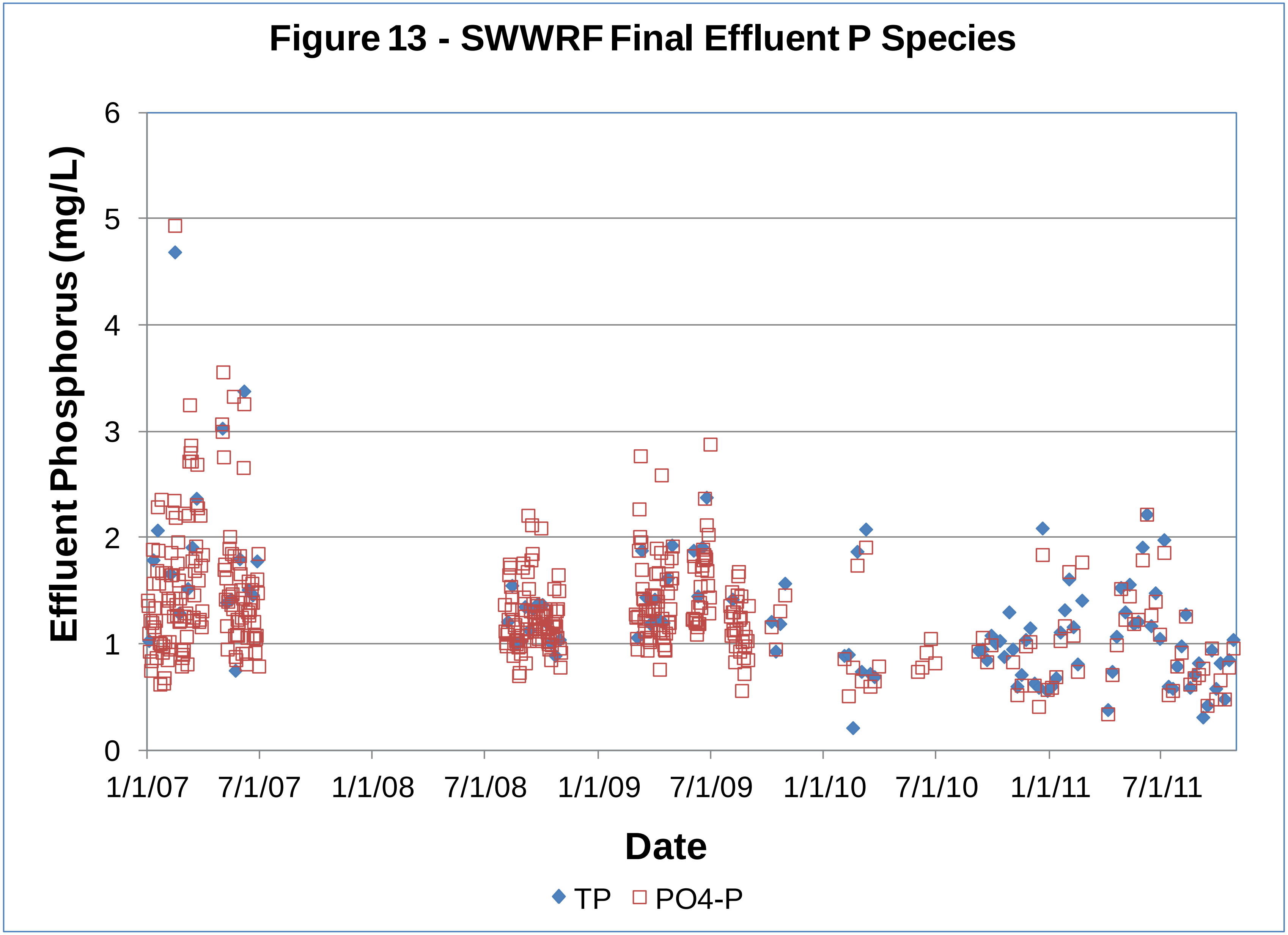


Figure 13 - SWWRF Final Effluent P Species
Effluent Phosphorus (mg/L)
0
1
2
3
4
5
6
1/1/07
7/1/07
1/1/08
7/1/08
1/1/09
7/1/09
1/1/10
7/1/10
1/1/11
7/1/11
Date
TP
PO4-P

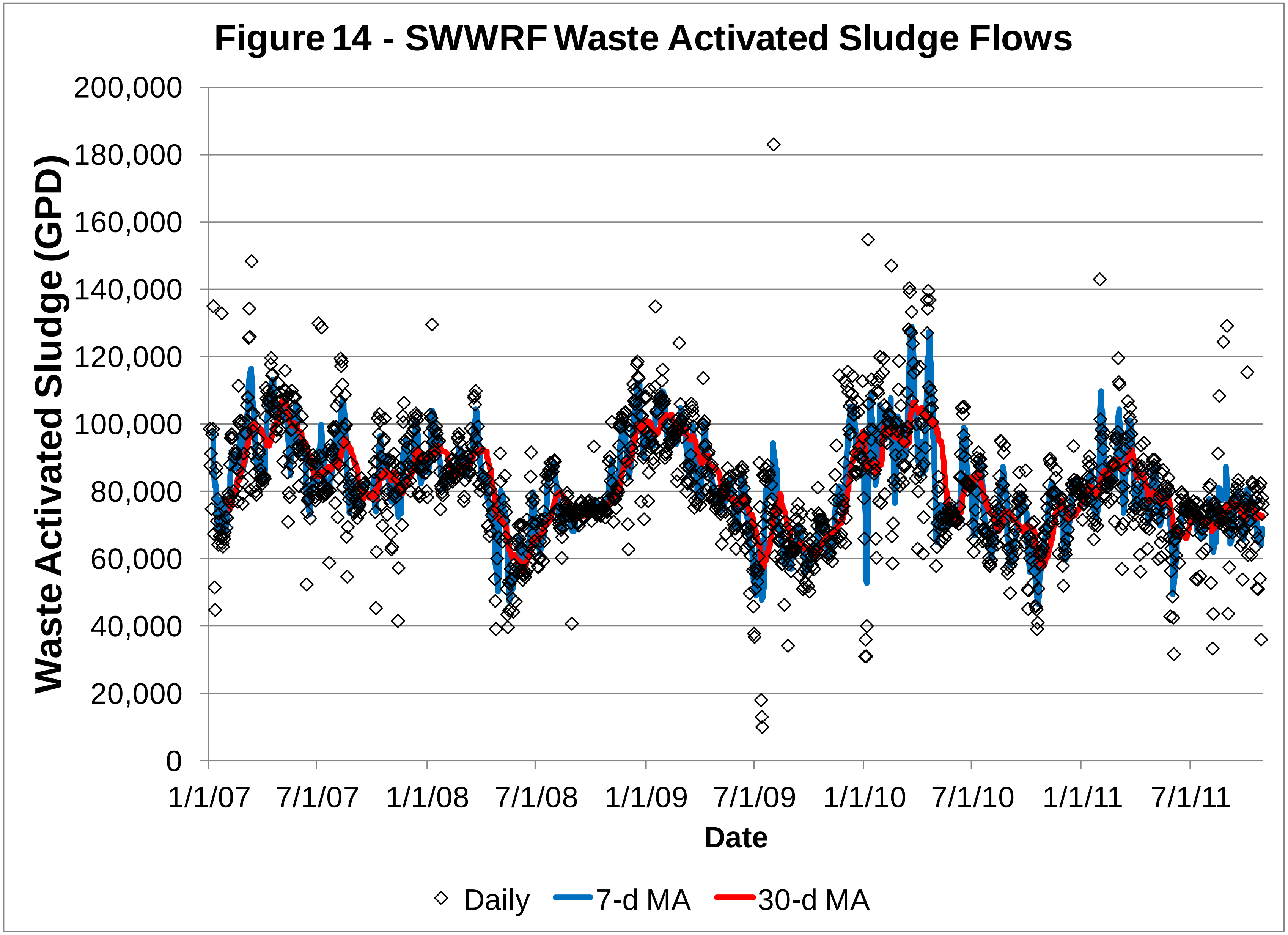


Figure 14 - SWWRF Waste Activated Sludge Flows
Waste Activated Sludge (GPD)
200,000
180,000
160,000
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
1/1/07
7/1/07
1/1/08
7/1/08
1/1/09
7/1/09
1/1/10
7/1/10
1/1/11
7/1/11
Date
Daily
7-d MA
30-d MA

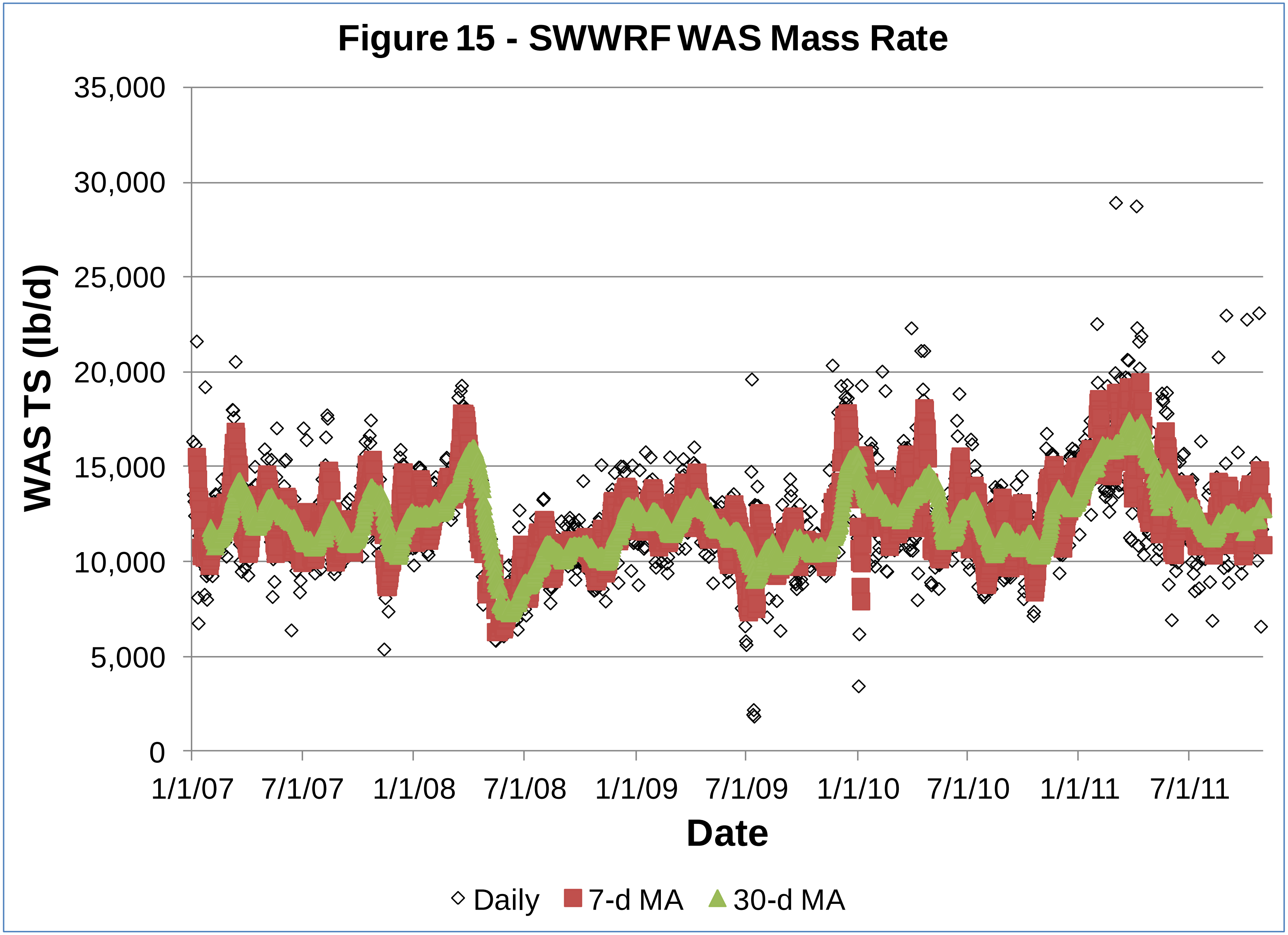


Figure 15 - SWWRF WAS Mass Rate
WAS TS (lb/d)
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
1/1/07
7/1/07
1/1/08
7/1/08
1/1/09
7/1/09
1/1/10
7/1/10
1/1/11
7/1/11
Date
Daily
7-d MA
30-d MA

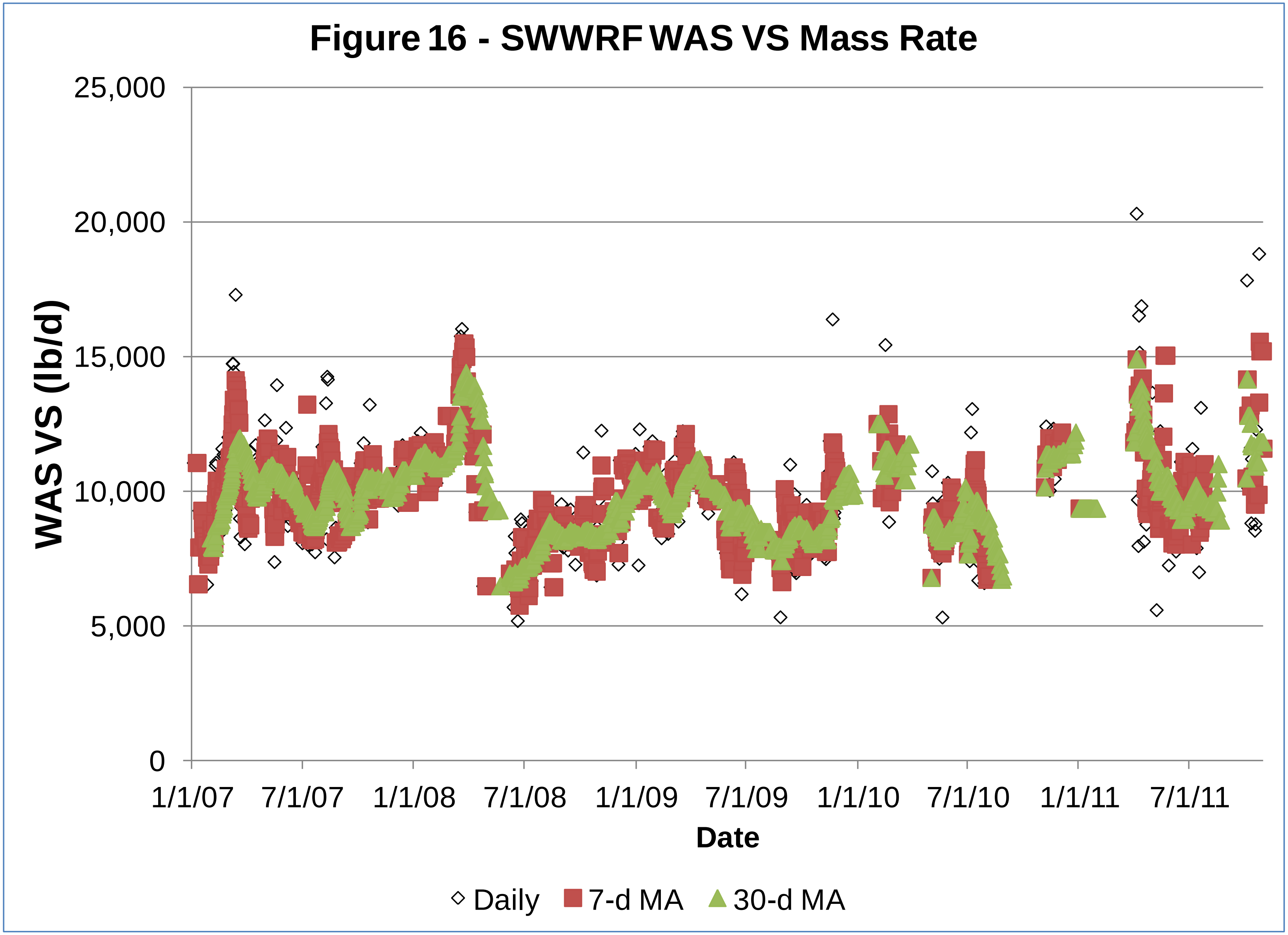


Figure 16 - SWWRF WAS VS Mass Rate
WAS VS (lb/d)
25,000
20,000
15,000
10,000
5,000
0
1/1/07
7/1/07
1/1/08
7/1/08
1/1/09
7/1/09
1/1/10
7/1/10
1/1/11
7/1/11
Date
Daily
7-d MA
30-d MA

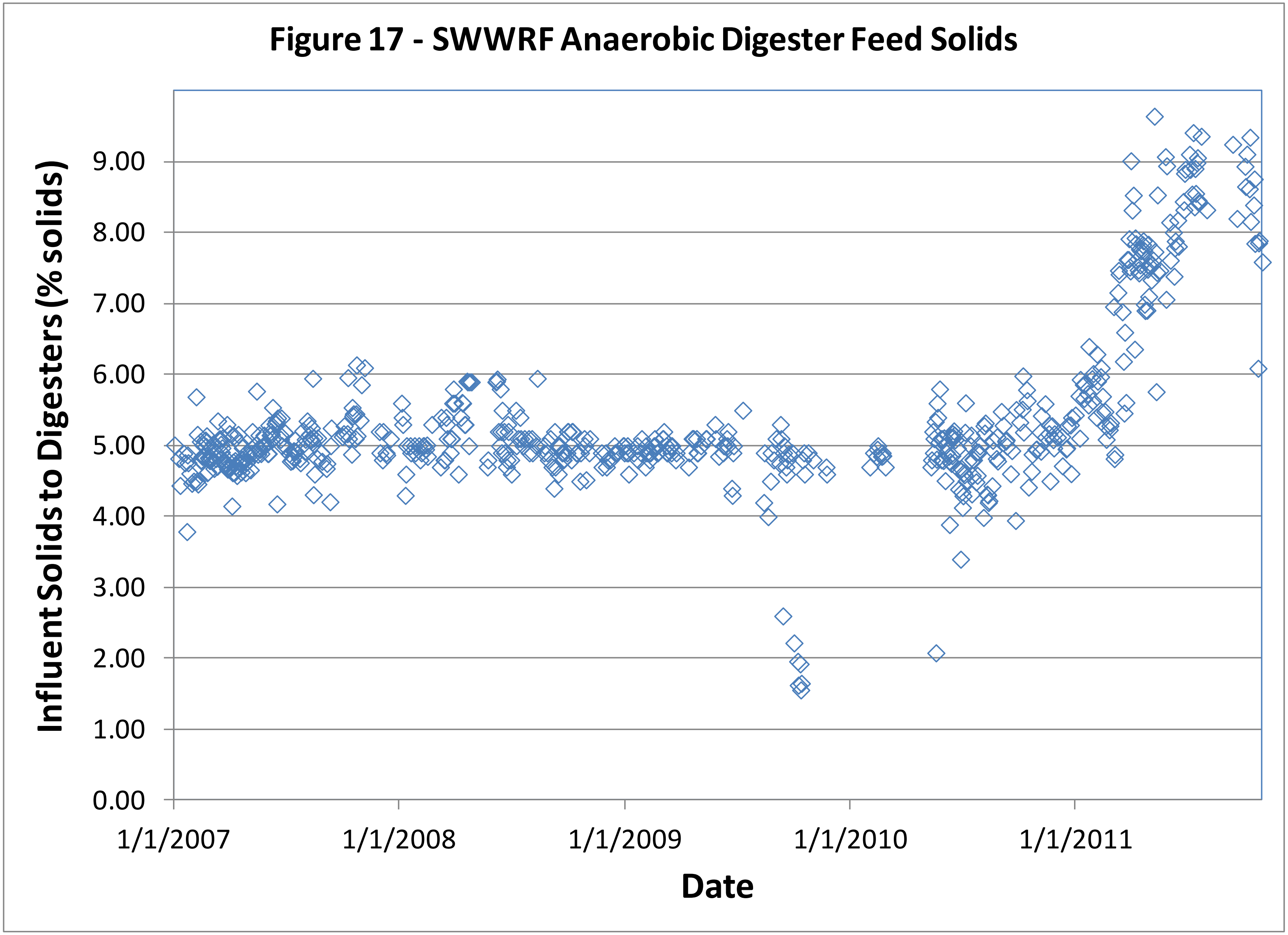


Figure 17 - SWWRF Anaerobic Digester Feed Solids
Influent Solids to Digesters (% solids)
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
1/1/2007
1/1/2008
1/1/2009
1/1/2010
1/1/2011
Date

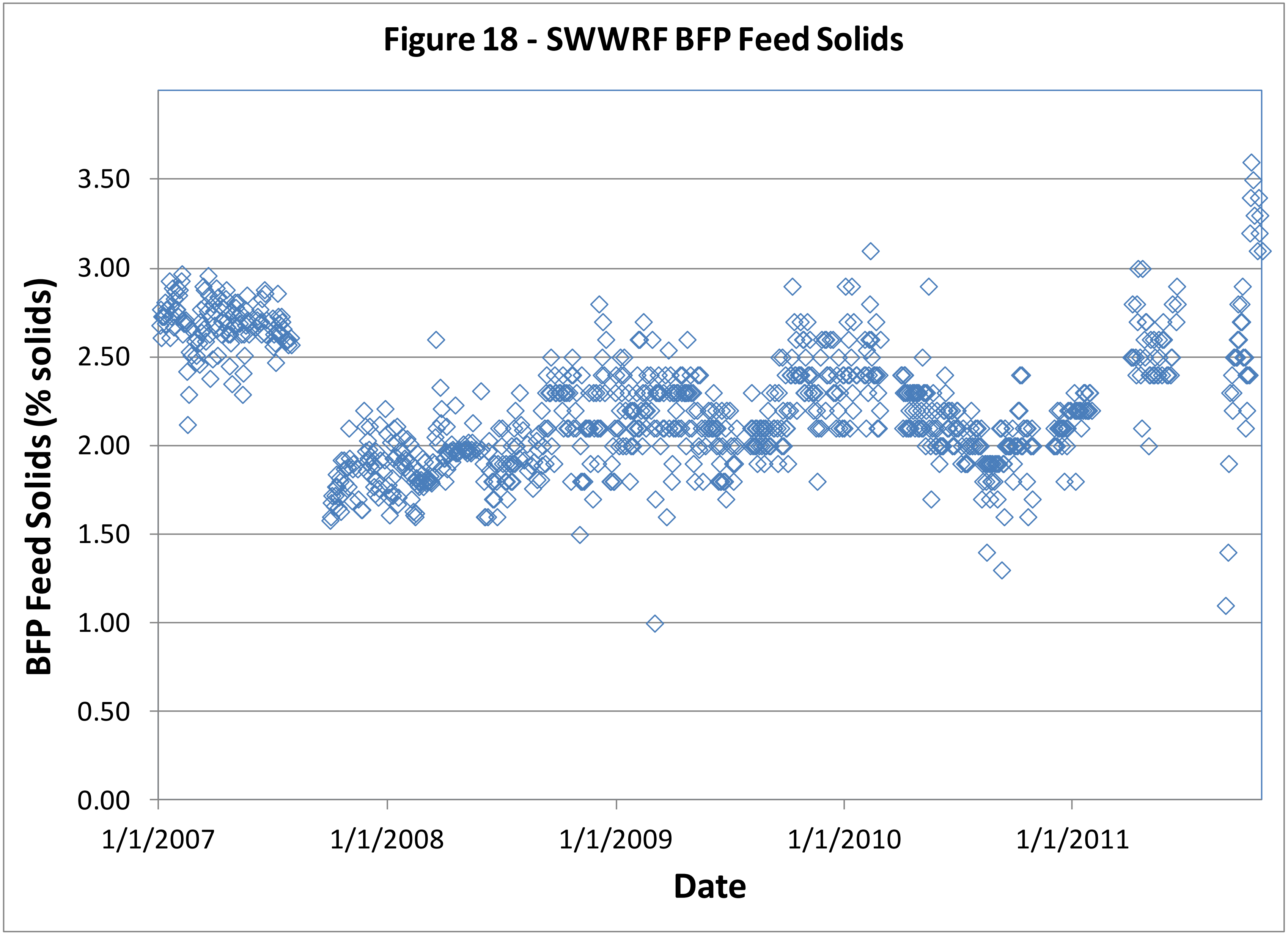


Figure 18 - SWWRF BFP Feed Solids
BFP Feed Solids (% solids)
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
1/1/2007
1/1/2008
1/1/2009
1/1/2010
1/1/2011
Date

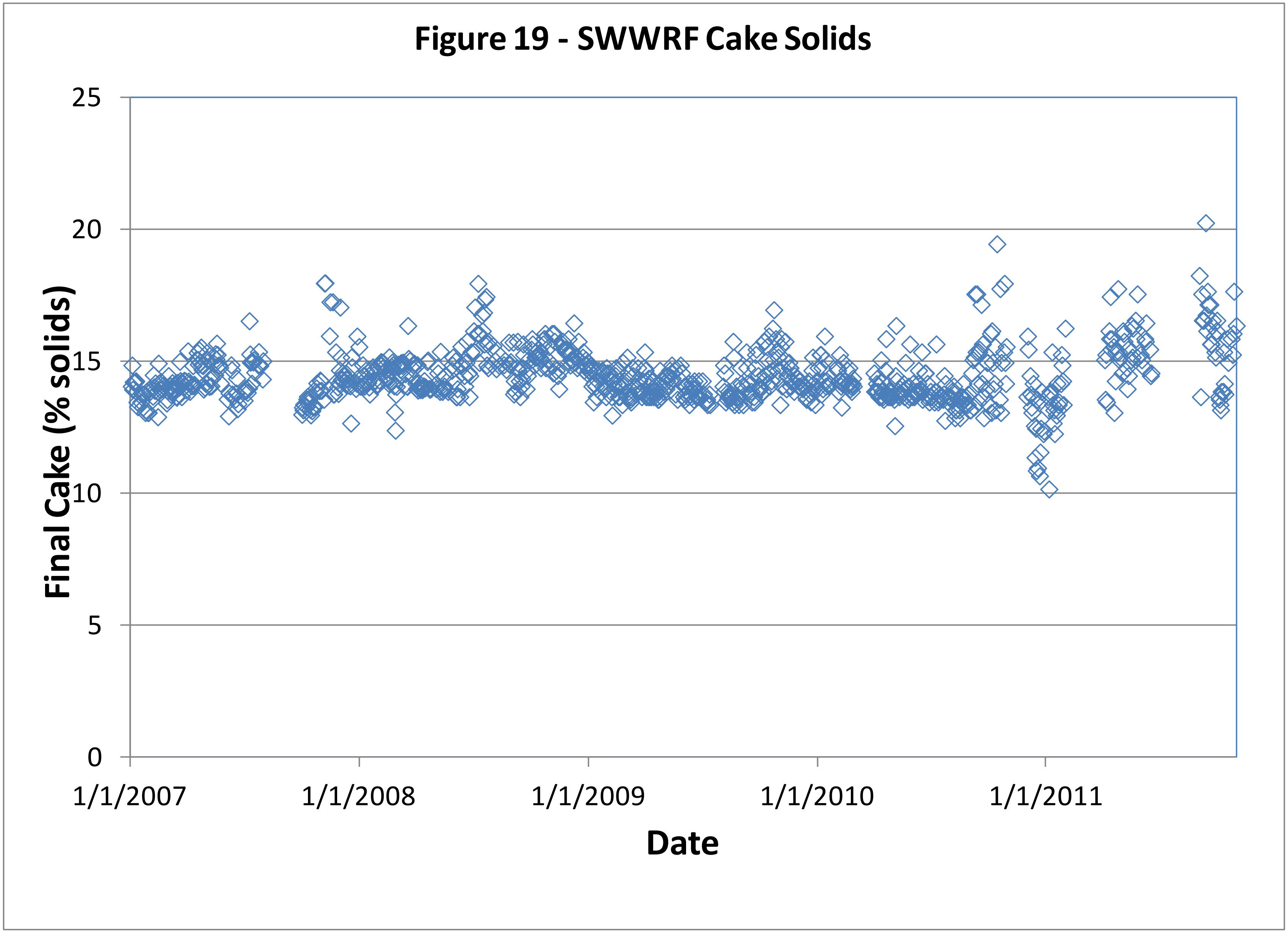


Figure 19 - SWWRF Cake Solids
Final Cake (% solids)
25
20
15
10
5
0
1/1/2007
1/1/2008
1/1/2009
1/1/2010
1/1/2011
Date

Appendix B: Sampling Plan

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Draft Technical Memorandum

Prepared for: City of St. Petersburg

Project Title: Engineering Services for the Southwest Water Reclamation Facility (SWWRF) Capacity Assessment Sampling Plan

Project No: 141555

Technical Memorandum

Subject: Sampling Plan (Draft)

Date: August 18, 2011

To: David Abbaspour, City of St. Petersburg

From: Jose A. Jimenez, Brown and Caldwell

Copy to: Todd Bosso, Brown and Caldwell
Robert Long, City of St. Petersburg
Randy Weatherspoon, City of St. Petersburg
Lane Longley, City of St. Petersburg
Kimberly Meyer, City of St. Petersburg
Paul Zimmermann, City of St. Petersburg

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1. Introduction

The City of St. Petersburg (CITY) is considering removing the Albert Whitted Water Reclamation Facility (AWWRF) from service and diverting that wastewater to the Southwest Water Reclamation Facility (SWWRF). In order to assess the impact of the wastewater flows from AWWRF to SWWRF and to determine the overall capacity of the facility, Brown and Caldwell has been tasked with developing a sampling plan to develop a calibrated BioWin® process model of the SWWRF. This model will be used in a separate task order to determine the process capacity of the SWWRF and to identify required improvements to allow the raw wastewater from both WRFs to be processed at the SWWRF.

Because every wastewater is different, the purpose of the wastewater characterization step is to measure the chemical oxygen demand (COD) in the influent to the activated sludge system and to partition the COD into the following fractions:

- unbiodegradable particulate COD,
- unbiodegradable soluble COD,
- biodegradable particulate COD,
- readily biodegradable (soluble) COD, and
- readily biodegradable COD that is volatile fatty acids.

A similar partitioning of the influent nitrogen and phosphorus species also are performed. The relative amount of these COD and nitrogen fractions dictates the kinetics of microbial growth in the activated sludge system.

In order to determine the specific wastewater characteristics, a special sampling plan will be conducted at the AWWRF and SWWRF. Samples will be collected and analyzed by the City staff. The City's personnel will decide whether the City's laboratory staff will conduct the sampling and analysis, or whether private laboratories will be contracted for this work. In the latter case, the City will coordinate with private laboratories to complete this work.

2. Sampling Plan

There are three elements to the special sampling plan recommended for the SWWRF: composite sampling, diurnal sampling and grab sampling.

2.1 Composite Sampling

Daily (24-hour) composite sampling will be performed over a three-week period. Because of the relatively small number of special sampling events, QA/QC procedures must be rigorously followed for both sampling and analytical methods. Table 1 summarizes the composite sampling matrix for the AWWRF and SWWRF. For each sampling day, samples will be collected at the following locations:

- AWWRF's raw influent: five days of flow-weighted composite samples will be collected after the existing bar screen with the existing composite sampler in that location. It is imperative that accurate flow monitoring is in place during the sampling study.
- SWWRF's raw influent: currently, influent to the SWWRF is pumped to the facility via two different force mains. Therefore, for the purpose of this analysis, five days of flow-weighted composite samples will be collected at each individual force main with the existing composite samplers. It is imperative that accurate flow monitoring is in place during the sampling study.
- SWWRF's combined influent with return streams: three days of composite samples will be collected after the existing bar screens and grit removal using the existing sampler currently being used for the BCR investigation. Since no flow measurements are in place at this location, the samples will be time proportion samples rather than flow-weighted samples. These will be as general backup information for the raw influent samples.
- SWWRF's secondary effluent: five days of flow-weighted composite samples will be collected at the SWWRF. A combined secondary effluent samples from the secondary clarifiers will be collected at the filtration facility's influent distribution chamber. Since no flow measurements are in place at this location, the samples will be time proportion samples rather than flow-weighted samples.
- SWWRF's final effluent: five days of flow-weighted composite samples will be collected at the chlorine contact tank with the existing composite sampler at that location. It is imperative that accurate flow monitoring is in place during the sampling study.

Table 1 – Composite Sampling Matrix

Parameters	AWWRF Raw Influent	SWWRF Raw Influent	SWWRF Combined Influent	SWWRF Secondary Effluent	SWWRF Final Effluent
Flow	X	X			
Total suspended solids	X	X	X	X	X
Volatile suspended solids	X	X			
Total COD	X	X	X	X	X
Soluble COD (0.45 micron filtrate)	X	X			X
Soluble COD (glass fiber filtrate)					X
"Floc" COD as per test	X	X			X
Carbonaceous 5-day BOD	X	X	X	X	X
Total TKN	X	X		X	X
Soluble TKN (0.45 micron filtrate)	X	X			X

Table 1 – Composite Sampling Matrix					
Parameters	AWWRF Raw Influent	SWWRF Raw Influent	SWWRF Combined Influent	SWWRF Secondary Effluent	SWWRF Final Effluent
Total ammonia	X	X	X		X
Nitrate					X
Nitrite					X
Total P	X	X	X	X	X
Soluble TP (0-.45 micron filtrate)	X	X			X
Soluble TP (glass fiber filtrate)	X	X			X
Orthophosphate P	X	X			X
Alkalinity (as CaCO3)	X	X			X
pH	X	X			X

2.2 Diurnal Sampling

During, or after the three-week period, grab samples will be collected at three-hour intervals during a 24-hour period. Three separate diurnal sampling events are normally sufficient. As for the composite sampling, because of the relatively small number of special sampling events, QA/QC procedures must be rigorously followed for both sampling and analytical methods. The samples will be analyzed for the constituents presented in Table 2. For each sampling day, samples will be collected at the following locations:

- AWWRF's raw influent: three days of diurnal samples will be collected at three-hour intervals (8 samples per day) with the existing influent sampler. It is imperative that accurate flow monitoring is in place during the sampling study.
- SWWRF's raw influent: three days of diurnal samples will be collected at three-hour intervals at each force main (8 samples per day per sampling location) with the existing influent samplers. It is imperative that accurate flow monitoring is in place during the sampling study.
- SWWRF's final effluent: three days of diurnal samples will be collected at three-hour intervals (8 samples per day) with the existing effluent sampler. It is imperative that accurate flow monitoring is in place during the sampling study.

Table 2 – Diurnal Sampling Matrix			
Parameters	AWWRF Raw Influent	SWWRF Raw Influent	SWWRF Final Effluent
Flow	X	X	X
Total suspended solids	X	X	X
Volatile suspended solids			
Total COD	X	X	
Soluble COD (0.45 micron filtrate)			X
Soluble COD (glass fiber filtrate)			
"Floc" COD as per test			
Total 5-day BOD (uninhibited)			
Carbonaceous 5-day BOD			

Table 2 – Diurnal Sampling Matrix			
Parameters	AWWRF Raw Influent	SWWRF Raw Influent	SWWRF Final Effluent
Total TKN			
Soluble TKN (0.45 micron filtrate)			
Total ammonia	X	X	X
Nitrate			
Nitrite			
Total P	X	X	
Soluble TP (0-.45 micron filtrate)			
Soluble TP (glass fiber filtrate)			
Orthophosphate P			X
Alkalinity (as $CaCO_3$)	X	X	X
pH	X	X	X

2.3 Grab Sampling

During the composite and diurnal sampling events, grab samples will be collected around important process units at the SWWRF in order to understand the performance and operation of the facility during the sampling events. Grab samples will be collected at least once per day and analyzed for a limited number of parameters as presented in Table 3. For each sampling day, samples will be collected at the same locations where the samples are currently collected by plant's personnel.

Table 3 – Grab Sampling Matrix								
Parameters	Aeration Basin Mixed Liquor	Return Activated Sludge	Waste Activated Sludge	GBT Overflow	GBT Underflow	Anaerobic Digester Effluent	BFP Centrate	BFP Cake
Flow		X	X	X	X	X	X	X
Total suspended solids	X	X	X	X	X	X	X	X
Volatile suspended solids	X					X		X
Total COD							X	
Soluble COD (0.45 micron filtrate)							X	
Soluble COD (glass fiber filtrate)								
"Floc" COD as per test								
Carbonaceous 5-day BOD								
Total TKN								
Soluble TKN (0.45 micron filtrate)							X	
Total ammonia							X	
Nitrate							X	
Nitrite							X	
Total P							X	
Soluble TP (0.45 micron filtrate)								

Table 3 – Grab Sampling Matrix								
Parameters	Aeration Basin Mixed Liquor	Return Activated Sludge	Waste Activated Sludge	GBT Overflow	GBT Underflow	Anaerobic Digester Effluent	BFP Centrate	BFP Cake
Soluble TP (glass fiber filtrate)								
Orthophosphate P							X	
Alkalinity (as $CaCO_3$)							X	
pH							X	

2.4 Number of Samples

Table 4 presents a summary of the number of analysis required per sampling day. This table also includes the total number of analysis needed to complete the sampling plan assuming five days of composite sampling and three days of diurnal sampling are completed.

Table 4 – Number of Analysis Required per Sampling Day			
Parameter	Composite Sampling Day [2]	Diurnal Sampling Day [2]	Total Number of Analysis
Flow	17	38	199
Total suspended solids	20	46	238
Volatile suspended solids	5	2	31
Total COD	8	26	118
Soluble COD (0.45 micron filtrate)	6	10	60
Soluble COD (glass fiber filtrate)	1	0	5
"Floc" COD as per test	4	0	20
Carbonaceous 5-day BOD	6	0	30
Total TKN	5	0	25
Soluble TKN (0.45 micron filtrate)	6	2	36
Total ammonia	7	34	137
Nitrate	3	2	21
Nitrite	3	2	21
Total P	8	26	118
Soluble TP (0-.45 micron filtrate)	4	0	20
Soluble TP (glass fiber filtrate)	4	0	20
Orthophosphate P	6	10	60
Alkalinity (as $CaCO_3$)	6	3	39
pH	6	6	48
			1047[1]

[1] Total number of analyses does not include flow measurements
[2] Composite and Diurnal Sampling Days include individual grab samples listed in Table 3

3. Recommended Methods for Analysis of Samples

Table 5 presents a summary list of the recommended methods for sample analysis.

Table 5 – List of Recommended Methods for Analysis

Parameter	Method [1]
TSS	EPA 160.2 SM 2540 D
VSS	EPA 160.4 SM 2540 E
COD	EPA 410.4 SM 5220 C or D HACH Method 8000 TNT 821-822
sCOD(0.45 micron) [2]	EPA 410.4 SM 5220 D HACH Method 8000 TNT 821-822
Floc filtered COD [3]	EPA 410.4 SM 5220 D HACH Method 8000 TNT 821-822
CBOD5	EPA 405.1 SM 5210 B by inclusion of step 5210B.4e6
TKN	EPA 351.2 SM 4500-Norg B. or C. HACH Method 10072
sTKN (0.45 micron) [2]	EPA 351.2 SM 4500-Norg B. or C. HACH Method 10072
NH3-N	EPA 350.1 SM 4500NH3-B and G HACH Method 10031 or 8155
NO3-N	EPA 353.2 SM 4500NO3-D or F HACH Method 8324
NO2-N	EPA 353.2 SM 4500NO3-B HACH Method 10207
TP	EPA 365.2 SM 4500P-E HACH Method 8190
PO4-P (0.45 micron) [2]	EPA 365.2 SM 4500P-E HACH Method 8048
Alk	EPA 310.1 SM 2320 B
pH	EPA 150.1 SM 4500-H+ B

[1] EPA: Environmental Protection Agency

SM: Standard Methods

[2] filtered through 0.45 micron glass fiber filter

[3] Floc Filtered COD: sample preparation per Mamais, D; Jenkins, D; and Pitt, P. "A Rapid Physical-Chemical Method for the Determination of Readily Biodegradable Soluble COD in Municipal Wastewater", Water Research, 27(1), pp. 195 - 197, 1993.

4. Specific Notes for Analysis

- It is imperative that the special sampling plan be programmed for a period when relatively stable operating conditions are expected – for example, taking units on or off line during this period should be avoided if at all possible.
- For this special sampling plan, sample bottles should be thoroughly washed and cleaned. All sampler tubing should be brand new and replaced for the campaign.
- Blending of samples: Samples for COD (total), TKN (total) and TP (total) should be thoroughly homogenized in a blender prior to analysis. This applies to samples of raw influent and secondary effluent.
- GF filtrate: the raw influent sampling includes COD, TKN and TP on "GF filtrate". This is the filtrate from 1.5-µm glass-fiber TSS filtration.
- Hach COD: presumably COD analyses will be conducted using the Hach Test-in-tube spectrophotometric method. High range tubes (0 – 1,500 mg/L) will be appropriate for the raw influent.
- Floc Filtered COD (ffCOD): Influent readily biodegradable COD concentration (RBCOD) will be measured using the ffCOD (flocculated and filtered COD) method of Mamais et al. (1993). The method is based on a physical separation, which involves pre-flocculation of the sample followed by filtration (referred to as the flocCODsol test or "ffCOD"). It is assumed that the flocculation step removes the colloidal material, resulting in a filtrate that contains only "truly soluble" material. The procedure is outlined briefly below:
  - 1 mL of 100 g/L zinc sulfate solution is added to 100 mL of wastewater;
  - the sample is then mixed vigorously for approximately 1 minute;
  - the sample pH is adjusted to approximately 10.5 using 6 M sodium hydroxide solution;
  - the sample is then allowed to settle, and a sample of the supernatant is withdrawn;
  - the supernatant sample is filtered using a 0.45 microns membrane filter, and the filtrate COD is analyzed.

References

Mamais, D., D. Jenkins and P. Pitt (1993) *A Rapid Physical-chemical Method for the Determination of Readily Biodegradable Soluble COD in Municipal Wastewater.* Water Res., 27(1):195-197.

Appendix C: Wastewater Characterization Study Results

Table 1. Influent Composite Sample Results at AWWRF					
Parameters	9/12/11	9/13/11	9/14/11	9/15/11	9/16/11
Alkalinity	194	195	202	207	214
CBOD	110	110	140	130	140
COD	296	311	414	325	399
Floc Filtered COD	74.4	79.4	94.1	96.3	94
Soluble COD (0.45 um)	91.1	103	111	119	113
Ammonia-N	16.1	17.6	18.5	19.3	19
Total Kjeldahl Nitrogen	26.4	26.2	29.1	29.3	29.9
Soluble TKN (0.45 um)	19.1	20.2	21.5	22.5	22
Soluble TKN (GF)	20	21.2	22.4	23.8	23.4
Total Phosphorous	6	3.96	7	4.44	6.72
Soluble Total Phosphorus (0.45 um)	4.12	2.26	5.16	2.73	4.56
Soluble Total Phosphorus (GF)	4.24	2.52	5.16	2.9	5.08
Soluble Orthophosphate (0.45 micron)	3.87	2.16	4.8	2.44	4.7
TSS	130	130	150	140	160
ISS	10	10	14	10	20
VSS	120	120	136	130	140
Field pH	7.3	7.27	7.27	7.22	7.22

Table 2. Influent Composite Wastewater Fraction Results for AWWRF					
Ratios	9/12/11	9/13/11	9/14/11	9/15/11	9/16/11
COD/ CBOD	2.69	2.83	2.96	2.50	2.85
CBOD/TSS	0.85	0.85	0.93	0.93	0.88
NH4/ TKN	0.61	0.67	0.64	0.66	0.64
NH4/ COD	0.05	0.06	0.04	0.06	0.05
NH4/ CBOD	0.15	0.16	0.13	0.15	0.14
TKN/ COD	0.09	0.08	0.07	0.09	0.07
TKN/ CBOD	0.24	0.24	0.21	0.23	0.21
sTKN/TKN (GF)	0.76	0.81	0.77	0.81	0.78
TP/ COD	0.02	0.01	0.02	0.01	0.02
TP/ CBOD	0.05	0.04	0.05	0.03	0.05
TP/ NH3	0.37	0.23	0.38	0.23	0.35
TP/ TKN	0.23	0.15	0.24	0.15	0.22
sTP/TP (GF)	0.71	0.64	0.74	0.65	0.76
sO-P/TP	0.65	0.55	0.69	0.55	0.70
VSS/ TSS	0.92	0.92	0.91	0.93	0.88
VSS/COD	0.41	0.39	0.33	0.40	0.35
VSS/PCOD	0.59	0.58	0.45	0.63	0.49

Table 3. Average Composite Sampling Results at Albert Whitted WRF	
Parameters	Average*, mg/L
Alkalinity	202.4
CBOD	126
COD	349
Floc Filtered COD	88
Soluble COD (0.45 um)	107
Ammonia-N	18.1
Total Kjeldahl Nitrogen	28.18
Soluble TKN (0.45 um)	21.06
Soluble TKN (GF)	22.16
Total Phosphorous	5.62
Soluble Total Phosphorus (0.45 um)	3.77
Soluble Total Phosphorus (GF)	3.98
Soluble Orthophosphate (0.45 micron)	3.59
TSS	142
ISS	12.8
Field pH	7.26

**Average of five day composite sampling*

Table. 4 – Diurnal Sampling Results for the AWWRF																		
Parameter	Alkalinity (mg/L)			Ammonia-N (mg/L)			COD (mg/L)			Total Phosphorous (mg/L)			TSS (mg/L)			Field pH (SU)		
Hours/Date	9/12/2011	9/13/2011	9/14/2011	9/12/2011	9/13/2011	9/14/2011	9/12/2011	9/13/2011	9/14/2011	9/12/2011	9/13/2011	9/14/2011	9/12/2011	9/13/2011	9/14/2011	9/12/2011	9/13/2011	9/14/2011
2:00	184	182	188	14.2	14.5	15.9	323	307	326	2.56	2.97	3.02	86	100	120	7.30		
5:00	165	165	164	10.8	10.7	11.0	154	187	243	1.75	2.49	2.24	50	54	86			
8:00	167	169	168	11.4	12.1	12.4	129	141	194	3.66	2.02	2.10	46	54	58			
11:00	214	210	216	21.8	22.2	22.8	356	352	339	9.56	5.36	8.44	190	190	150			
14:00	225	229	232	22.0	23.4	24.2	375	383	404	9.52	5.64	11.90	200	160	180	7.34	7.18	7.31
17:00	209	214	217	19.6	19.6	20.6	309	322	369	7.08	4.52	8.60	130	130	150			
20:00	204	204	210	18.0	18.6	19.3	330	306	371	7.30	4.04	9.04	170	100	140			
23:00	190	195	203	16.3	17.0	17.6	324	352	354	3.58	3.86	4.04	150	120	120			
Avg	194.75	196	199.75	16.7625	17.2625	17.975	287.5	293.75	325	5.62625	3.8625	6.1725	127.75	113.5	125.5	7.32	7.18	7.31

Table 5. Diurnal Flow Variation at the AWWRF					
Hours/Date	9/12/2011	9/13/2011	9/14/2011	9/15/2011	9/16/2011
0:00	7.25	7.24	8.86	7.08	6.90
1:00	7.27	7.22	9.63	7.08	6.90
2:00	10.12	8.70	6.74	6.98	6.81
3:00	9.32	9.19	6.30	6.92	6.75
4:00	7.11	6.29	5.86	5.97	5.97
5:00	6.95	5.98	5.57	5.34	5.03
6:00	6.58	6.19	5.82	5.50	5.29
7:00	7.11	6.80	6.45	6.08	6.13
8:00	7.24	6.99	6.88	6.83	6.74
9:00	10.87	10.28	8.35	9.33	6.87
10:00	9.69	8.93	8.94	8.36	8.35
11:00	10.76	10.33	10.51	9.13	10.24
12:00	10.98	9.40	9.45	8.91	8.91
13:00	10.64	9.15	9.32	8.75	8.72
14:00	10.01	8.89	9.22	8.77	8.57
15:00	9.62	8.81	8.88	8.39	8.24
16:00	9.50	8.74	8.67	8.15	7.98
17:00	9.30	8.56	8.61	8.04	7.87
18:00	9.10	8.41	8.62	7.91	7.80
19:00	9.13	8.47	8.54	7.97	7.76
20:00	9.03	8.60	8.63	8.08	7.75
21:00	9.14	8.79	8.59	8.31	7.64
22:00	7.65	7.35	7.34	7.32	6.67
23:00	7.12	6.90	7.02	6.85	6.56
AVG	8.81	8.18	8.03	7.59	7.35

Table 6. Diurnal Influent Patterns for the AWWRF *						
Hour	Alkalinity	NH3-N	COD-N	TP	TSS	Flow
	ratio	ratio	ratio	ratio	ratio	ratio
2:00	0.92	0.83	1.03	0.54	0.80	0.99
5:00	0.62	0.46	0.47	0.31	0.37	0.75
8:00	0.80	0.63	0.47	0.46	0.39	0.94
11:00	1.27	1.48	1.34	1.67	1.66	1.19
14:00	1.29	1.47	1.41	1.81	1.60	1.13
17:00	1.13	1.19	1.14	1.28	1.15	1.06
20:00	1.09	1.10	1.14	1.27	1.12	1.05
23:00	0.88	0.85	0.99	0.65	0.91	0.89

** Ratios are calculated based on loads*

Table. 7 – Influent Composite Sampling Results for the SWWRF													
		INFLUENT A						INFLUENT B					
Parameters	Unit	9/26/2011	9/27/2011	9/28/2011	9/29/2011	9/30/2011	Avg	9/26/2011	9/27/2011	9/28/2011	9/29/2011	9/30/2011	Avg
Alkalinity	mg/L	246	199	192	199	214	210.00	249	217	217	204	236	224.60
CBOD	mg/L	130	100	82	86	88	97.20	95	55	40	40	49	55.80
COD	mg/L	413	279	344	284	377	339.40	312	389	352	360	365	355.60
Floc Filtered COD	mg/L	108	84.5	69.5	85.9	69.0	83.38	74.3	53	48.4	52.2	47.8	55.14
Soluble COD (0.45 micron)	mg/L	137	108	137	105	92.4	115.88	86.8	69.8	67.4	73.3	66.9	72.84
Soluble COD (GF)	mg/L	--	--	--	--	--	--	--	--	--	--	--	--
Ammonia-N	mg/L	23.4	13.2	12.8	14.3	15.1	15.76	13	8.53	8.26	9.53	11	10.06
Total Kjeldahl Nitrogen	mg/L	32.5	20.8	18.6	20.9	21.7	22.90	19.9	13.8	12	12.9	13.6	14.44
Soluble TKN (0.45 micron)	mg/L	24.3	15.8	13.8	15.3	15.9	17.02	13.6	9.56	8.8	10.2	10.8	10.59
Soluble TKN (GF)	mg/L	24.9	16.4	14.4	15.7	16.6	17.60	14.2	9.88	9.08	10.6	11.6	11.07
Soluble Nitrate+Nitrite-N (0.45 micron)	mg/L	--	--	--	--	--	--	--	--	--	--	--	--
Soluble Nitrate-N (0.45 micron)	mg/L	--	--	--	--	--	--	--	--	--	--	--	--
Soluble Nitrite-N (0.45 micron)	mg/L	--	--	--	--	--	--	--	--	--	--	--	--
Soluble Orthophosphate (0.45 micron)	mg/L	1.95	0.98	1.05	1.22	1.27	1.29	1.2	0.72	0.79	0.93	1.06	0.94
Total Phosphorous	mg/L	4.12	2.50	2.14	2.68	2.97	2.88	2.96	2.02	1.4	1.77	1.86	2.00
Soluble Total Phosphorus (0.45 micron)	mg/L	2.01	1.10	1.03	1.28	1.44	1.37	1.31	0.86	0.63	1.04	1.04	0.98
Soluble Total Phosphorus (GF)	mg/L	2.04	1.15	1.10	1.35	1.49	1.43	1.32	0.84	0.72	1.02	1.04	0.99
TSS	mg/L	190	550	100	130	120	218.00	210	140	92	78	90	122.00
ISS	mg/L	36	80	20	21	22	35.80	72	54	33	34	34	45.40
VSS	mg/L	154	470	80	109	98	182.20	138	86	59	44	56	76.60
Field pH	SU	7.14		7.30	7.38	7.28	7.28	7.24	7.23	7.61	7.83	7.77	7.54
(Particulate COD)pCOD	mg/L	276	171.00	207.00	179.00	284.60	223.52	225.2	319.2	284.6	286.7	298.1	282.76
(Colloidal COD) CODs - ffCOD	mg/L	29.00	23.50	67.50	19.10	23.40	32.50	12.50	16.80	19.00	21.10	19.10	17.70
Fus		0.10	0.14	0.13	0.13	0.09	0.12	0.13	0.10	0.13	0.10	0.09	0.11
Fbs		0.16	0.17	0.07	0.17	0.10	0.13	0.11	0.04	0.01	0.04	0.04	0.05

Table. 8 – Secondary and Final Effluent Composite Results for the SWWRF													
		SECONDARY EFFLUENT						FINAL EFFLUENT					
Parameters	Unit	9/26/2011	9/27/2011	9/28/2011	9/29/2011	9/30/2011	Avg	9/26/2011	9/27/2011	9/28/2011	9/29/2011	9/30/2011	Avg
Alkalinity	mg/L	--	--	--	--	--	--	240	219	200	199	208	213.20
CBOD	mg/L	3.9	3.7	2.7	2.8	2.6	3.14	2.9	2.5	2.4	2	2	2.36
COD	mg/L	59.6	50.5	52.6	48	48.9	51.92	47	55.2	50.5	48	41.4	48.42
Floc Filtered COD	mg/L	--	--	--	--	--	--	40.7	37.9	46.3	37.4	32.9	39.04
Soluble COD (0.45 micron)	mg/L	--	--	--	--	--	--	47	46.8	56.8	52.2	45.6	49.68
Soluble COD (GF)	mg/L	--	--	--	--	--	--	47	46.8	52.6	48	45.6	48.00
Ammonia-N	mg/L	--	--	--	--	--	--	13	12.4	8.41	7.79	9.02	10.12
Total Kjeldahl Nitrogen	mg/L	15.4	14.8	10.9	10.3	-	12.85	13.9	13.7	9.42	8.48	9.56	11.01
Soluble TKN (0.45 micron)	mg/L	--	--	--	--	--	--	13	12.9	9	8.28	9.24	10.48
Soluble TKN (GF)	mg/L	--	--	--	--	--	--	13.7	13.4	9.22	8.3	9.68	10.86
Soluble Nitrate+Nitrite-N (0.45 micron)	mg/L	--	--	--	--	--	--	0.72	1.12	0.96	1.33	0.96	1.02
Soluble Nitrate-N (0.45 micron)	mg/L	--	--	--	--	--	--	0.14	0.23	0.21	0.39	0.27	0.25
Soluble Nitrite-N (0.45 micron)	mg/L	--	--	--	--	--	--	0.58	0.89	0.74	0.94	0.69	0.77
Soluble Orthophosphate (0.45 micron)	mg/L	--	--	--	--	--	--	0.91	0.48	0.34	0.42	0.67	0.56
Total Phosphorous	mg/L	1.15	0.62	0.42	0.65	-	0.71	1.04	0.58	0.37	0.49	0.68	0.63
Soluble Total Phosphorus (0.45 micron)	mg/L	--	--	--	--	--	--	0.99	0.55	0.33	0.48	0.69	0.61
Soluble Total Phosphorus (GF)	mg/L	--	--	--	--	--	--	1.06	0.57	0.39	0.51	0.68	0.64
TSS	mg/L	3.6	36	10	2.8	3.2	11.12	1.1	-	1.1	0.7	0.5	0.85
ISS	mg/L	--	--	--	--	--	--	--	--	--	--	--	--
VSS	mg/L												
Field pH	SU	--	--	--	--	--	--	7.42	7.48		7.34	7.45	7.42
								0.0	8.4	-6.3	-4.2	-4.2	-1.26
								6.30	8.90	10.50	14.80	12.70	10.64

Table 9. Average Composite Sampling Results at Southwest WRF*					
Parameter	Influent A	Influent B	Combined Influent	Secondary Effluent	Final Effluent
	Average*, mg/L	Average*, mg/L	Average*, mg/L	Average*, mg/L	Average*, mg/L
Alkalinity	210.00	224.60	--	--	213.20
CBOD	97.20	55.80	85.20	3.14	2.36
COD	339.40	355.60	285.20	51.92	48.42
Floc Filtered COD	83.38	55.14	--	--	39.04
Soluble COD (0.45 micron)	115.88	72.84	--	--	49.68
Soluble COD (GF)	--	--	--	--	48.00
Ammonia-N	15.76	10.06	20.58	--	10.12
Total Kjeldahl Nitrogen	22.90	14.44	27.78	12.85	11.01
Soluble TKN (0.45 micron)	17.02	10.59	--	--	10.48
Soluble TKN (GF)	17.60	11.07	--	--	10.86
Soluble Nitrate+Nitrite-N (0.45 micron)	--	--	--	--	1.02
Soluble Nitrate-N (0.45 micron)	--	--	--	--	0.25
Soluble Nitrite-N (0.45 micron)	--	--	--	--	0.77
Soluble Orthophosphate (0.45 micron)	1.29	0.94	--	--	0.56
Total Phosphorous	2.88	2.00	4.27	0.71	0.63
Soluble Total Phosphorus (0.45 micron)	1.37	0.98	--	--	0.61
Soluble Total Phosphorus (GF)	1.43	0.99	--	--	0.64
TSS	218.00	122.00	125.20	11.12	0.85
ISS	35.80	45.40	--	--	--
Field pH	7.28	7.54	--	--	7.42

**Average of five day composite sampling*

Table 10. Average Wastewater Fractions at Southwest WRF					
Ratios	Influent A	Influent B	Combined Influent	Secondary Effluent	Final Effluent
	Average*, mg/L	Average*, mg/L	Average*, mg/L	Average*, mg/L	Average*, mg/L
COD/ CBOD	3.55	7.12	3.34	16.87	20.81
CBOD/TSS	0.62	0.47	0.75	0.65	2.92
NH4/ TKN	0.68	0.70	0.74	--	0.92
NH4/ COD	0.05	0.03	0.07	--	0.21
NH4/ CBOD	0.16	0.19	0.24	--	4.27
TKN/ COD	0.07	0.04	0.09	0.24	0.23
TKN/ CBOD	0.23	0.27	0.32	3.92	4.64
sTKN/TKN (0.45)	0.74	0.74	--	--	0.96
sTKN/TKN (GF)	0.77	0.77	--	--	0.99
TP/ COD	0.01	0.01	0.01	0.01	0.01
TP/ CBOD	0.03	0.04	0.05	0.21	0.27
TP/ NH3	0.18	0.20	0.21	--	0.06
TP/ TKN	0.13	0.14	0.15	0.05	0.06
sTP/TP (0.45)	0.47	0.49	--	--	0.96
sTP/TP (GF)	0.49	0.50	--	--	1.02
sO-P/TP	0.45	0.48	--	--	0.89
VSS/ TSS	0.82	0.62	--	--	--
VSS/COD	0.31	0.22	--	--	--
VSS/PCOD	0.47	0.29	--	--	--

Table 11. Diurnal Influent Variation at Southwest WRF*											
Sample		Influent A					Influent B				
Hours	Flow	Alkalinity	Ammonia-N	COD	TP	TSS	Alkalinity	Ammonia-N	COD	TP	TSS
2:00	0.92	0.86	0.80	0.89	0.83	0.89	0.92	0.85	1.04	0.85	1.01
5:00	0.79	0.72	0.63	0.59	0.61	0.58	0.79	0.71	0.83	0.72	0.85
8:00	0.97	0.91	0.84	0.64	0.78	0.71	0.92	0.77	0.73	0.74	0.73
11:00	1.08	1.11	1.25	0.94	1.21	1.17	1.01	1.03	0.75	0.92	0.74
14:00	1.06	1.13	1.31	1.23	1.28	1.14	1.09	1.37	0.99	1.27	1.04
17:00	1.02	1.08	1.09	1.24	1.13	1.14	1.08	1.26	1.24	1.25	1.10
20:00	1.07	1.10	1.06	1.22	1.08	1.10	1.09	1.04	1.22	1.17	1.47
23:00	1.08	1.09	1.02	1.25	1.08	1.26	1.09	0.98	1.20	1.07	1.06

** Ratios are calculated based on loads, 71% of the total influent flow assumed to be originated from influent A and 29% of the total influent assumed to be coming from influent B*

Table. 12 – Hourly Influent and Effluent Flow Data during the Diurnal Sampling at the SWWRF								
Sample	TOTAL INFLUENT FLOW						EFFLUENT FLOW	
Hours/Date	10/3/2011	10/11/2011	10/12/2011	10/13/2011	11/16/2011	11/17/2011	11/16/2011	11/17/2011
0:00	14.09	14.36	13.55	12.85	9.80	9.66	6.65	8.87
1:00	13.18	13.24	12.64	11.95	8.97	8.94	7.88	10.96
2:00	11.86	12.37	11.63	11.02	7.81	7.86	8.34	11.46
3:00	11.23	11.45	10.67	10.24	6.98	6.87	7.47	11.07
4:00	10.67	10.87	10.29	9.76	7.06	6.80	7.32	10.69
5:00	10.17	10.55	9.83	9.50	6.81	7.12	5.49	9.94
6:00	10.60	10.79	10.10	9.51	6.84	6.76	5.99	7.69
7:00	11.64	11.83	11.12	10.76	8.30	8.20	11.45	5.70
8:00	12.92	13.11	12.29	11.83	9.71	9.79	23.41	5.22
9:00	14.22	14.02	13.52	12.88	10.44	10.28	23.71	1.99
10:00	13.89	14.15	13.57	13.12	10.43	10.33	9.66	6.18
11:00	14.58	14.35	14.01	13.22	11.08	10.91	18.75	0.00
12:00	14.52	14.25	14.02	13.44	10.57	10.61	19.44	1.89
13:00	14.18	14.24	13.75	13.20	10.31	10.36	11.64	9.47
14:00	14.05	14.12	13.31	12.90	10.35	10.32	11.53	9.46
15:00	13.87	13.96	13.47	13.10	9.68	9.79	11.61	9.45
16:00	13.36	13.44	12.94	12.57	10.07	9.72	5.21	9.44
17:00	13.46	13.38	12.78	12.62	9.55	9.58	2.39	9.45
18:00	13.22	13.56	13.28	12.83	10.77	10.17	2.49	9.44
19:00	13.49	13.72	13.45	12.95	10.56	10.39	1.48	9.45
20:00	13.98	14.26	13.80	13.27	10.76	10.51	5.17	7.55
21:00	14.24	14.61	14.07	13.48	10.88	10.66	8.11	2.82
22:00	14.39	14.53	13.98	13.37	10.90	10.73	5.39	3.25
23:00	14.22	14.56	13.87	13.59	10.33	10.20	6.82	5.74
AVG	13.17	13.32	12.75	12.25	9.54	9.44	9.47	7.38

Appendix D: BioWin Influent Specifier for COD Fraction Determination

Southwest WRF – Influent A COD Fractions [Data based on average composite results from Special Sampling]

Measurements	Value	Unit
Main influent concentrations		
Flow	12.4	mgd or m3/d
Total COD	299.4	mgCOD/L
Total Kjeldahl Nitrogen	22.9	mgN/L
Total P	2.9	mgP/L
Other influent concentrations		
Nitrate N	0.0	mgN/L
pH	7.3	
Alkalinity (CaCO3 equivalent)	210.0	mgCaCO3/L
Calcium	80.0	mg/L
Magnesium	15.0	mg/L
Dissolved oxygen	0.0	mgO2/L
Other measurements		
Effluent filtered COD	39.0	mgCOD/L
Influent filtered COD (GFC)	120.0	mgCOD/L
Influent FF COD	83.4	mgCOD/L
Influent acetate	7.0	mgCOD/L
Influent ammonia	15.8	mgN/L
Influent ortho-phosphate	1.3	mgP/L
Influent carbonaceous BOD5	125.0	mgO2/L
Influent filtered cBOD5		mgO2/L
Influent VSS	117.3	mgVSS/L
Influent TSS	138.0	mgTSS/L

GUIDE

- Enter measured lab data in column on left (**BOLD**)
  (If data is missing, estimate. May need to repeat after Step 2)
- Check resulting fractions **(BOLD)**

Parameter	Value	Unit	Typical range
Alkalinity (molar)	**4.2**	**meq/L**	**2 - 6**
Fus	**0.13**	-	**0.03 - 0.08**
CODp	**179.4**	**mgCOD/L**	
Fbs	**0.15**	-	**0.12 - 0.25**
Fac	**0.15**	-	**0.0 - 0.3**
Fna	**0.69**	-	**0.5 - 0.8**
Fpo4	**0.45**	-	**0.3 - 0.6**
COD/BOD5	**2.40**	-	**1.9 - 2.2**
Fcv	**1.53**	**mgCODp/mgVSS**	**1.5 - 1.7**
ISS	**20.7**	**mgISS/L**	**15 - 45**

Influent COD fractions	Default	Estimate	*Notes*
Fbs	0.160	0.153	*from Step 1*
Fus	0.050	0.125	*from Step 1*
Fup	0.130	**0.180**	*affects BOD, VSS*
Fzbh	0.000	**0.000**	*from separate method*
Fxs	0.660	0.542	*by difference (must be > 0!!)*
Fxsp	0.750	**0.770**	*affects VSS, scale: 0 to 1*

GUIDE

- Change COD fractions **(BOLD)** until match is achieved

Suggestion:
Inhibited cBOD5 = 0.84 x "true" cBOD5

Influent values	Measured (From Step 1)	Calculated (Based on fractions above)	*Match Status*
CODt	299	299	***Excellent***
Soluble COD (GFC)	120	121	***Excellent***
FF COD	83	83	***Excellent***
cBOD5	125	125	***Excellent***
fcBOD5	0	59	***Unacceptable***
VSS	117	117	***Excellent***
TSS	138	138	***Excellent***

Important fractions	(can be used as a check)	
Fraction	Value	Typical range
COD/cBOD5	2.40	1.9-2.2
Sol. COD fraction	0.40	0.3-0.5
VSS/TSS	0.85	0.75-0.85

Calculated concentrations (from CODt & fractions)		
Sus	38	
Xi	54	
Sbs	46	
Xs (c+p)	162	
Zbh	0	
Xsc	37	Added to Ss for BOD calcs
Xsp	125	

Southwest WRF – Influent B COD Fractions [Data based on average composite results from Special Sampling]

Measurements	Value	Unit
Main influent concentrations		
Flow	4.4	mgd or m3/d
Total COD	195.6	mgCOD/L
Total Kjeldahl Nitrogen	14.4	mgN/L
Total P	2.0	mgP/L
Other influent concentrations		
Nitrate N	0.0	mgN/L
pH	7.5	
Alkalinity (CaCO3 equivalent)	224.6	mgCaCO3/L
Calcium	80.0	mg/L
Magnesium	15.0	mg/L
Dissolved oxygen	0.0	mgO2/L
Other measurements		
Effluent filtered COD	39.0	mgCOD/L
Influent filtered COD (GFC)	79.0	mgCOD/L
Influent FF COD	55.8	mgCOD/L
Influent acetate	2.8	mgCOD/L
Influent ammonia	10.1	mgN/L
Influent ortho-phosphate	0.9	mgP/L
Influent carbonaceous BOD5	85.0	mgO2/L
Influent filtered cBOD5		mgO2/L
Influent VSS	85.0	mgVSS/L
Influent TSS	100.0	mgTSS/L

GUIDE

- Enter measured lab data in column on left (**BOLD**)
  (If data is missing, estimate. May need to repeat after Step 2)
- Check resulting fractions **(BOLD)**

Parameter	Value	Unit	Typical range
Alkalinity (molar)	4.5	meq/L	2 - 6
Fus	0.19	-	0.03 - 0.08
CODp	116.6	mgCOD/L	
Fbs	0.09	-	0.12 - 0.25
Fac	0.15	-	0.0 - 0.3
Fna	0.70	-	0.5 - 0.8
Fpo4	0.47	-	0.3 - 0.6
COD/BOD5	2.30	-	1.9 - 2.2
Fcv	1.37	mgCODp/mgVSS	1.5 - 1.7
ISS	15.0	mgISS/L	15 - 45

Influent COD fractions	Default	Estimate	*Notes*
Fbs	0.160	0.094	*from Step 1*
Fus	0.050	0.192	*from Step 1*
Fup	0.130	0.040	*affects BOD, VSS*
Fzbh	0.000	0.000	*from separate method*
Fxs	0.660	0.675	*by difference (must be > 0!!)*
Fxsp	0.750	0.830	*affects VSS, scale: 0 to 1*

GUIDE

- Change COD fractions **(BOLD)** until match is achieved

Suggestion:
Inhibited cBOD5 = 0.84 x "true" cBOD5

Influent values	Measured (From Step 1)	Calculated (Based on fractions above)	*Match Status*
CODt	196	196	*Excellent*
Soluble COD (GFC)	79	78	*Excellent*
FF COD	56	56	*Excellent*
cBOD5	85	87	*Excellent*
fcBOD5	0	29	*Unacceptable*
VSS	85	86	*Excellent*
TSS	100	101	*Excellent*

Important fractions	(can be used as a check)	
Fraction	Value	Typical range
COD/cBOD5	2.26	1.9-2.2
Sol. COD fraction	0.40	0.3-0.5
VSS/TSS	0.85	0.75-0.85

Calculated concentrations (from CODt & fractions)		
Sus	38	
Xi	8	
Sbs	18	
Xs (c+p)	132	
Zbh	0	
Xsc	22	Added to Ss for BOD calcs
Xsp	110	

Albert Whitted WRF – Influent COD Fractions [Data based on average composite results from Special Sampling]

Measurements	Value	Unit
Main influent concentrations		
Flow	6.0	mgd or m3/d
Total COD	350.0	mgCOD/L
Total Kjeldahl Nitrogen	28.2	mgN/L
Total P	5.6	mgP/L
Other influent concentrations		
Nitrate N	0.0	mgN/L
pH	7.3	
Alkalinity (CaCO3 equivalent)	202.0	mgCaCO3/L
Calcium	80.0	mg/L
Magnesium	15.0	mg/L
Dissolved oxygen	0.0	mgO2/L
Other measurements		
Effluent filtered COD	39.0	mgCOD/L
Influent filtered COD (GFC)	140.0	mgCOD/L
Influent FF COD	87.9	mgCOD/L
Influent acetate	7.5	mgCOD/L
Influent ammonia	18.1	mgN/L
Influent ortho-phosphate	3.6	mgP/L
Influent carbonaceous BOD5	156.0	mgO2/L
Influent filtered cBOD5		mgO2/L
Influent VSS	120.7	mgVSS/L
Influent TSS	142.0	mgTSS/L

GUIDE

- Enter measured lab data in column on left (**BOLD**)
  (If data is missing, estimate. May need to repeat after Step 2)
- Check resulting fractions **(BOLD)**

Parameter	Value	Unit	Typical range
Alkalinity (molar)	**4.0**	**meq/L**	**2 - 6**
Fus	**0.11**	**-**	**0.03 - 0.08**
CODp	**210.0**	**mgCOD/L**	
Fbs	**0.14**	**-**	**0.12 - 0.25**
Fac	**0.15**	**-**	**0.0 - 0.3**
Fna	**0.64**	**-**	**0.5 - 0.8**
Fpo4	**0.64**	**-**	**0.3 - 0.6**
COD/BOD5	**2.24**	**-**	**1.9 - 2.2**
Fcv	**1.74**	**mgCODp/mgVSS**	**1.5 - 1.7**
ISS	**21.3**	**mgISS/L**	**15 - 45**

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Influent COD fractions	Default	Estimate	*Notes*
Fbs	0.160	0.144	*from Step 1*
Fus	0.050	0.107	*from Step 1*
Fup	0.130	**0.140**	*affects BOD, VSS*
Fzbh	0.000	**0.000**	*from separate method*
Fxs	0.660	0.609	*by difference (must be > 0!!)*
Fxsp	0.750	**0.760**	*affects VSS, scale: 0 to 1*

GUIDE

- Change COD fractions **(BOLD)** until match is achieved

Suggestion:
Inhibited cBOD5 = 0.84 x "true" cBOD5

Influent values	Measured (From Step 1)	Calculated (Based on fractions above)	*Match Status*
CODt	350	350	***Excellent***
Soluble COD (GFC)	140	139	***Excellent***
FF COD	88	88	***Excellent***
cBOD5	156	157	***Excellent***
fcBOD5	0	72	***Unacceptable***
VSS	121	121	***Excellent***
TSS	142	143	***Excellent***

Important fractions	(can be used as a check)	
Fraction	Value	Typical range
COD/cBOD5	2.22	1.9-2.2
Sol. COD fraction	0.40	0.3-0.5
VSS/TSS	0.85	0.75-0.85

Calculated concentrations (from CODt & fractions)		
Sus	38	
Xi	49	
Sbs	50	
Xs (c+p)	213	
Zbh	0	
Xsc	51	Added to Ss for BOD calcs
Xsp	162	

Combined Southwest and Albert Whitted WRFs – Influent Fractions based on average composite results from Special Sampling

SWWRF - Inf. A	31025	lb/d	12.4	mgd
SWWRF - Inf. B	7192	lb/d	4.4	mgd
AWWRF	17514	lb/d	6	mgd

		SWWRF					
Name	Default	Influent A	Influent B	L. W. Avg.	F.W. Avg.	AWWRF	Composite Flows
Fbs	0.16	0.153	0.094	0.142	0.138	0.144	0.1426
Fac	0.15	0.150	0.150	0.150	0.150	0.150	0.1500
Fxsp	0.75	0.770	0.830	0.781	0.786	0.760	0.7746
Fus	0.05	0.125	0.192	0.138	0.143	0.107	0.1281
Fup	0.13	0.180	0.040	0.154	0.143	0.140	0.1494
Fna	0.66	0.688	0.697	0.690	0.690	0.642	0.6748
Fnox	0.5	0.500	0.500	0.500	0.500	0.500	0.5000
Fnus	0.02	0.020	0.020	0.020	0.020	0.020	0.0200
FupN	0.035	0.035	0.035	0.035	0.035	0.035	0.0350
Fpo4	0.5	0.449	0.470	0.453	0.454	0.643	0.5126
FupP	0.011	0.011	0.011	0.011	0.011	0.011	0.0110
FZbh	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZbm	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZaob	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZnob	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZamob	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZbp	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZbpa	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
Fzbam	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
FZbhm	1.00E-04	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
COD:TSS		2.1660	1.96	2.127	2.112	2.45	2.2283
COD:TKN		12.5	12.5	12.500	12.500	12.23	12.4153
COD:NH4		19.14	18.70	19.05	19.02	19.04	19.05
TP:COD		0.0100	0.01	0.010	0.010	0.02	0.0119
COD:cBOD		2.4	2.3	2.381	2.374	2.24	2.3368
cBOD:TSS		0.9	0.85	0.891	0.887	1.1	0.9564

Appendix E: BioWin Process Model Calibration and Validation

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Technical Memorandum

Project Title: Capacity Assessment of St. Petersburg, Southwest Water Reclamation Facility

Project No: 142081

Technical Memorandum [No. 4]

Subject: Calibration of BioWin Process Simulator at Southwest WRF, St. Petersburg, FL

Date: January 14, 2011

From: FILE

To: Derya Dursun and Jose Jimenez

Limitations:

This is a draft memorandum and is not intended to be a final representation of the work done or recommendations made by Brown and Caldwell. It should not be relied upon; consult the final report.

1. Introduction

This Technical Memo (TM) provides a summary of results of the calibration for the BioWin biological process simulator used for modeling the activated sludge processes for the Southwest Water Reclamation Facility (SWWRF) at St. Petersburg, FL. Detailed wastewater characteristics obtained from special sampling campaigns were utilized for calibration of the simulator in September-October 2011.

A model for the activated sludge secondary treatment processes at the SWWRF was created using the BioWin simulator, developed by EnviroSim Associates Ltd. of Flamborough, Ontario, Canada. BioWin allows the prediction of complex biological interactions using various mechanistic and empirical models to represent material transformations and pollutant removals in the plant for both liquid and solids process streams. It enables the user to simulate carbonaceous oxidation and the fate of nutrients in activated sludge treatment facilities.

The intent of the calibration was to ensure that the model is accurate when used to simulate future flow and loading conditions for the capacity assessment. Two types of verification exercises were conducted to verify the BioWin model predictions: (1) calibration of the model from September 26 thru October 17, 2011, which includes the special sampling period; and, (2) validation of the model using plant daily historical data from September 1, 2011 through October 31, 2011.

2. Calibration

Figure 1 depicts the SWWRF process flow schematic as created in the BioWin simulator. Shown are two plug flow reactors with four aerobic zones each, the secondary clarifiers, filters, the RAS and WAS streams and the solid unit processes including WAS storage tank, gravity belt thickener (GBT), anaerobic digesters, and belt filter presses (BFP).

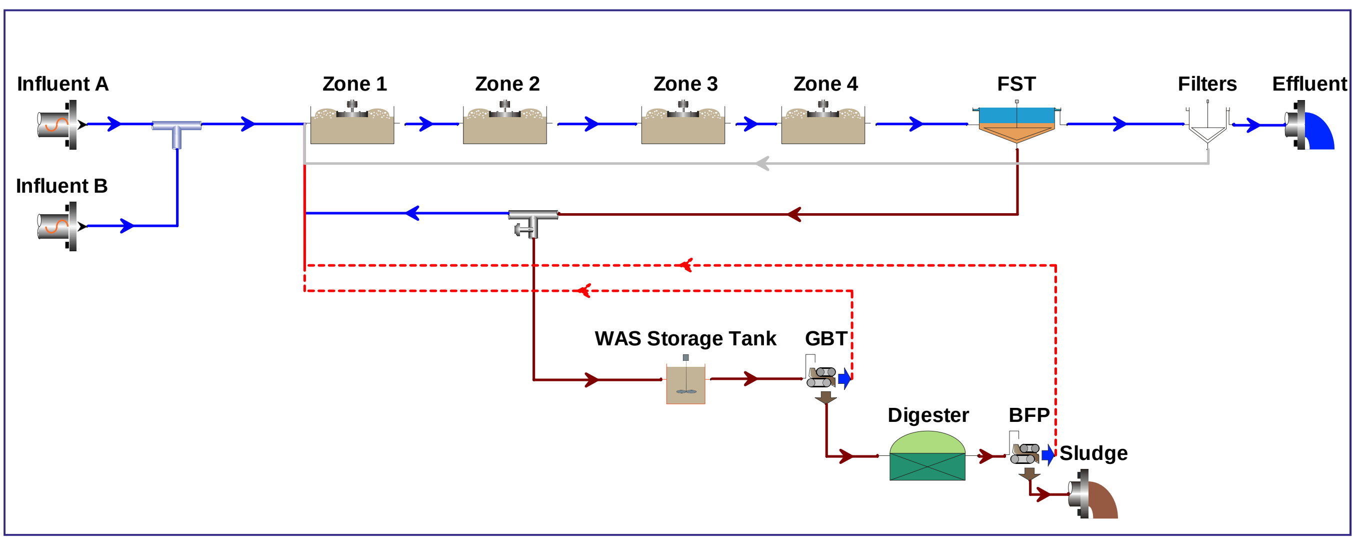

Figure 1. SWWRF Process Flow Schematic in BioWin Simulator

2.1 BioWin Simulation for Calibration

The wastewater characterization data presented in TM-1 were used to calibrate a dynamic-state simulation of the SWWRF over the 22-day period (from September 26 to October 17, 2011). The graphical representation of the plant layout and flow scheme was created as shown in Figure 1, in which physical data such as tank volumes and clarifier areas were specified (Table 1). Process data such as influent flow rates and compositions; recycle rates, and typical operating DO concentrations were also entered into the model. The actual aeration basin temperature was maintained at average 27.6°C, same as measured during the wastewater characterization period used for calibration.

Table 1. Process Physical Data

Element Name	Description	Volume, mgal	Area, ft^2	Depth, ft
Zone 1	¼ th of aeration tank 1+¼ th of aeration tank 2	1.03	9,180	15
Zone 2	¼ th of aeration tank 1+¼ th of aeration tank 2	1.03	9,180	15
Zone 3	¼ th of aeration tank 1+¼ th of aeration tank 2	1.03	9,180	15
Zone 4	¼ th of aeration tank 1+¼ th of aeration tank 2	1.03	9,180	15
FST	3- secondary clarifiers	4.18	42,942	13
Filters	Tertiary filters	n/a	n/a	n/a
Was Storage tank	Sludge storage tank	0.11	996	14.76
GBT	Gravity Belt Thickener	n/a	n/a	n/a
Digester	1-Anaerobic Digester	1.0	7,854	17
BFP	2-Belt Filter Presses	n/a	n/a	n/a

Figure 2 – Current Reactor Configuration at the SWWRF

Because all recycle flows (centrate from BFP, GBT), and the RAS were included in the process simulation, the contribution of centrate to the influent would be included in the simulations. Each aerobic zone was configured as a separate cell in the model. Two aeration tanks were configured as four aerated zones operating in series, with mechanical aeration. Actual DO values were input into the simulation, since this might significantly impact the results of simulations

Simulator calibration was achieved by matching, as closely as possible, the predicted effluent characteristics with the measured daily plant performance data during September 26 to October 17, 2011 sampling period. The dynamic model was then checked against operating data from the wastewater characterization period to produce a final calibrated dynamic model that accurately depicts the conditions in the SWWRF. The results of the dynamic simulations were compared to the effluent and mixed liquor characteristics measured during the wastewater characterization period to verify that the model was properly calibrated.

2.2 Calibration Simulation Results

Table 2 summarizes the average input and output values from the dynamic simulation and compares the model predictions with the average values for plant measurements over the simulation period. Diurnal conditions were modeled and comparisons of the model predictions with plant performance for the simulation period are shown on Figures 3, 4, 5 and 6.

For the calibration period, the WAS flow rate was modified to match the measured mixed liquor suspended solid concentration, measured WAS mass rate and the effluent parameters, within a reasonable margin of error. The WAS flow rate was maintained approximately at 0.4 mgd to maintain an SRT of 4.5 days and MLSS of around 1,800 mg/L. Plant reported WAS flow rates did not seem accurate and therefore were not used for calibration purpose.

Calibration results of the dynamic model is listed on Table 2 below, where the third column lists those parameters observed at the facility during the wastewater characterization period, and the last column of the table lists those values as predicted by BioWin. The table shows that the BioWin-predicted parameters are in close agreement with the plant-measured values for SRT, MLSS and all measured effluent parameters.

Table 2. BioWin Calibration Results Summary

Parameter	Units	Measured	BioWin Calibration
Aeration Basins			
MLSS	mg/L	1,800	1,760
MLVSS	mg/L	--	1,337
SRT	days	4.9	4.5
Return Activated Sludge (RAS)			
Flow	MGD	11.1	--
Concentration	mg/L	4,036	3,900
Waste Activated Sludge (WAS)			
Flow	MGD	0.0745	0.4
Concentration	mg/L	17,300	3,900
Mass Rate	lbs/d	10,750	11,915
Final Effluent			
TSS	mg/L	0.98	0.97
COD	mg/L	48.4	--
Soluble COD	mg/L	44.6	42.5

$CBOD_5$	mg/L	2.12	1.35
TKN	mg/L	10.8	11.8
NH_3-N	mg/L	9.3	10.3
NO_3-N	mg/L	0.25	0.17
NO_2-N	mg/L	0.75	0.74
TP	mg/L	0.64	0.75
Sludge Handling			
An. Digesters VSS in	%	2.8	2.5
An. Digesters VSS out	%	1.8	1.6
Filtrate NH_3-N	mg/L	734	799
Filtrate TP	mg/L	200	276
Cake	%	13.98	14.0
Total Mass of Sludge	lb/d	6,799	7,726

The dynamic simulation results for influent COD and cBOD provide good agreement as shown in Figure 3. Although the influent COD is an input to the model, influent cBOD is calculated by using the influent characteristics and kinetic parameters built in the BioWin simulator. Having a close match between actual cBOD vs predicted cBOD indicates the accuracy of wastewater characterization. The model also very closely predicts the MLSS concentration in reactors (Figure 4). Figure 5 presents the agreement between predicted values and plant-measured values for effluent TSS. Although, there is some variation between the measured effluent TSS and that predicted by the model, but the model provides a reasonable estimate and gives similar average results. Effluent COD and cBOD concentrations, also shown on Figure 6, are modeled well using the BioWin simulator. Because there were only minor differences between the plant-measured daily values and those predicted by the model, no further modifications to flows, influent characteristics, or biological kinetics were required from the simulations. The calibrated BioWin simulator for the SWWRF will be used in the plant capacity assessment and to determine the capacity of the existing facility.

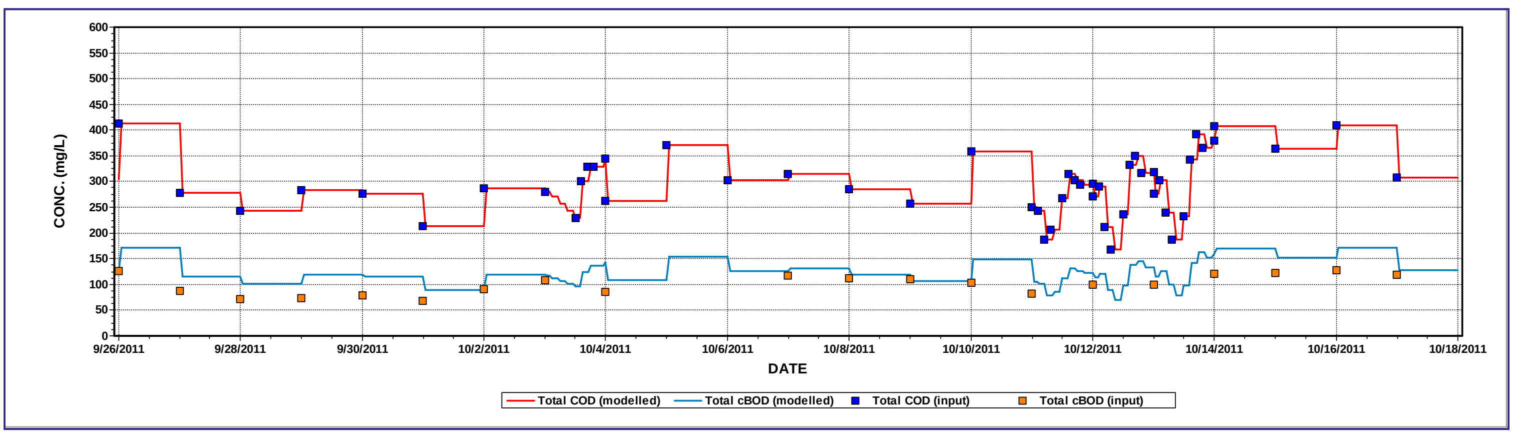

Figure 3 – BioWin Input for Influent COD and cBOD

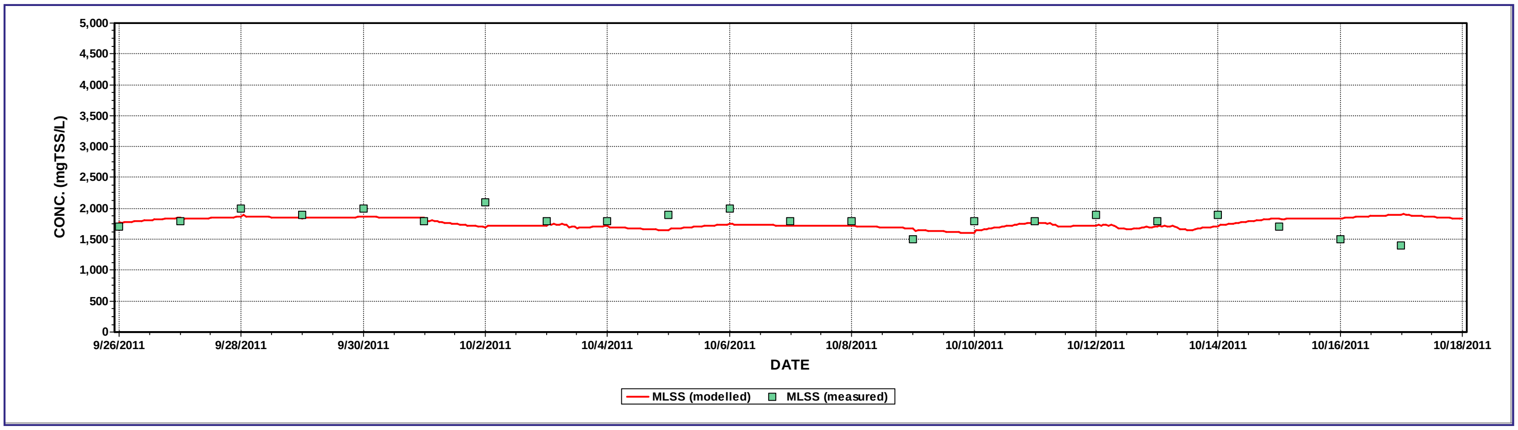

Figure 4 – Dynamic Simulation of MLSS in Reactors

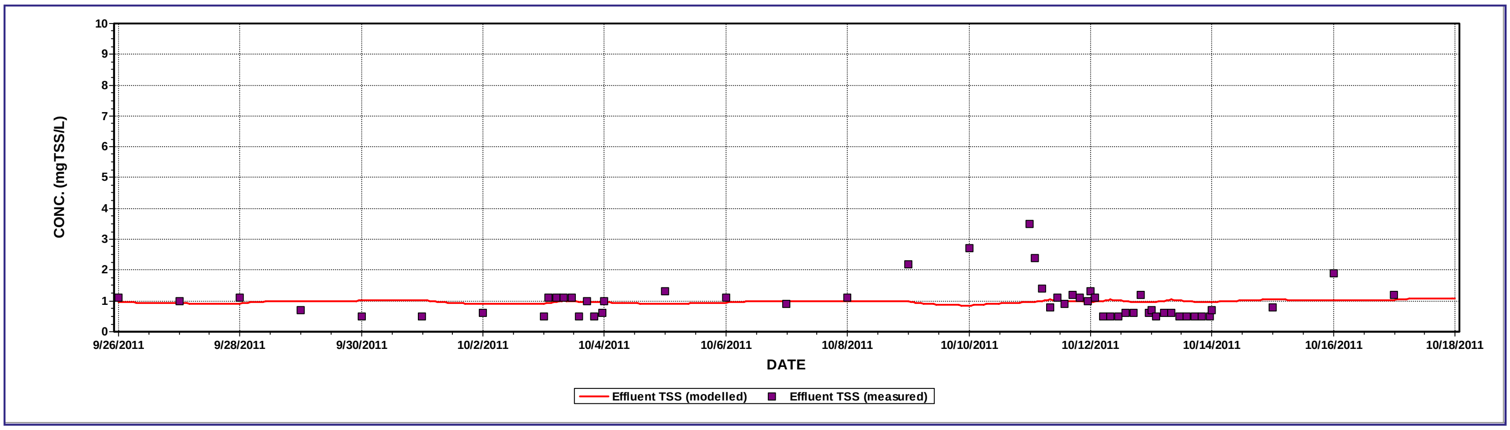

Figure 5 – Dynamic Simulation of Effluent TSS Concentrations

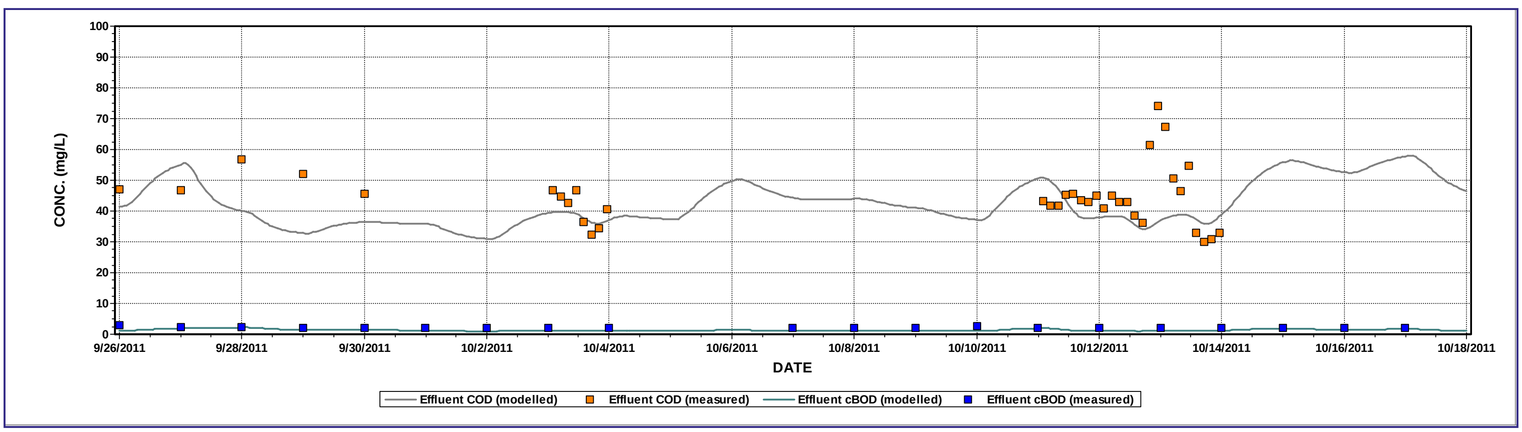

Figure 6 – Dynamic Simulation of Effluent COD and cBOD Concentrations

3. Validation

After calibration to the special sampling data, the same BioWin model (provided in Section 2.1) was used to simulate an extended period from September 1, 2011 through October 31, 2011 to ensure the validity of the results predicted by the model. Historical plant data in the form of daily averages was used for the purpose of this analysis.

The input information for BioWin was based on historical influent TSS, NH_3-N and total P for the period specified above. The following influent fractions were used to estimate the influent COD, TKN and ISS concentration for modeling purposes:

- COD to TSS ratio of 2.13
- COD to TKN ratio of 12.5
- ISS to COD ratio of 0.07

3.1 Validation Simulation Results

Similar to the calibration, daily average operational parameters (temperature, DO etc) were input to the model. Daily RAS flow values were entered as well in the model based on data provided by plant staff. Figure 7 shows the predicted and observed mixed liquor concentration over the validation period. As seen in this Figure 7, the BioWin model predicts the MLSS concentrations relatively well. However, similar to calibration, WAS rates were manipulated to match the MLSS concentrations, and reported WAS mass rates. Figure 8 also exhibits the BioWin predicted SRT values, and compared them with plant reported MCRT. The model predicted values somewhat agreed with the plant reported values, although in the first two week period the model under predicted the SRT.

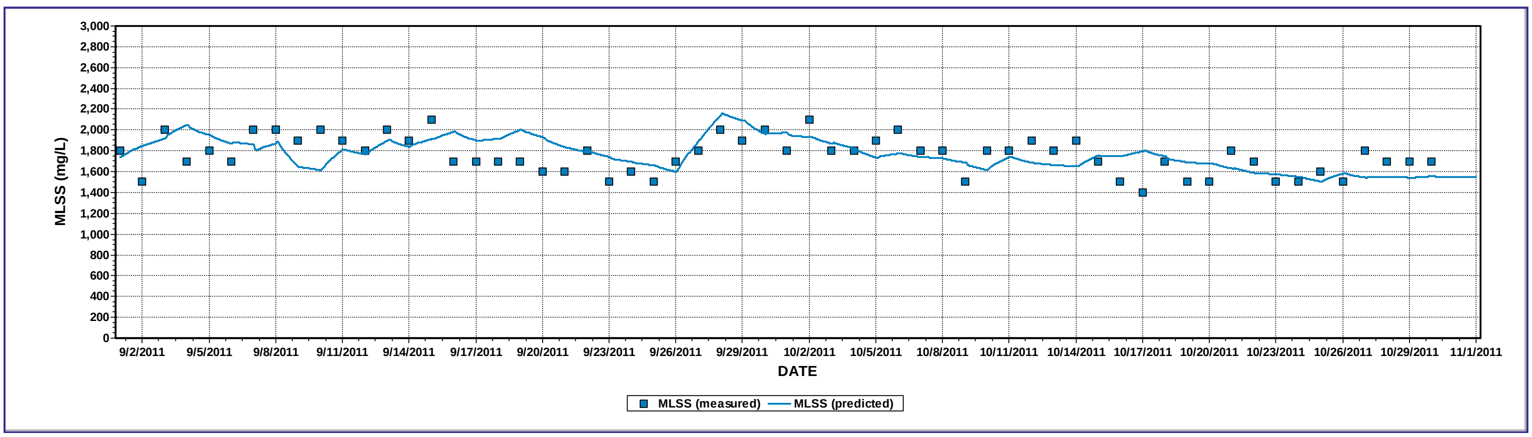

Figure 7 – Dynamic Simulation of MLSS Concentrations for 2-month Validation Period

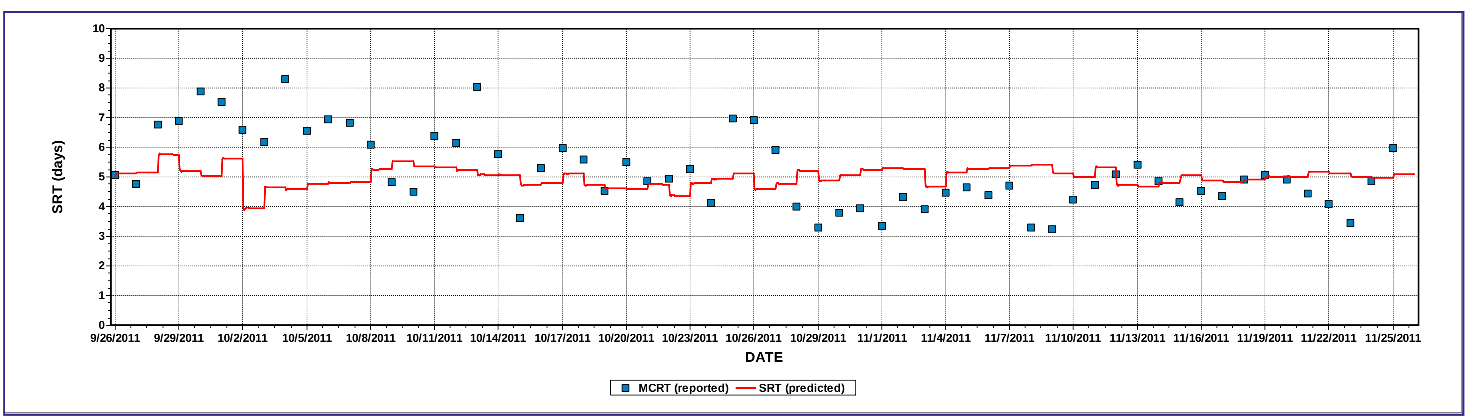

Figure 8 – Dynamic Simulation of SRT for 2-month Validation Period

Dynamic conditions were modeled for final effluent and comparisons of the model predictions with plant performance for the validation period are shown on Figures 9, 10, 11 and 12. Once modifications were made to the WAS flows, the dynamic simulation results for effluent TSS and COD (Figures 9 and 10) show reasonable agreement between predicted values and plant-measured values.

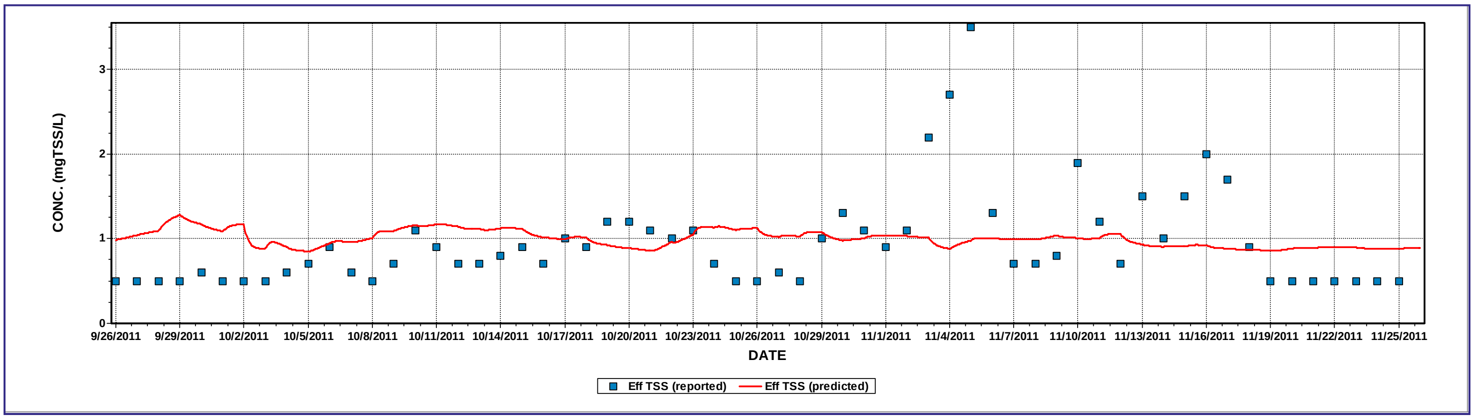

Figure 9 – Dynamic Simulation of Effluent TSS Concentrations for 2-month Validation Period

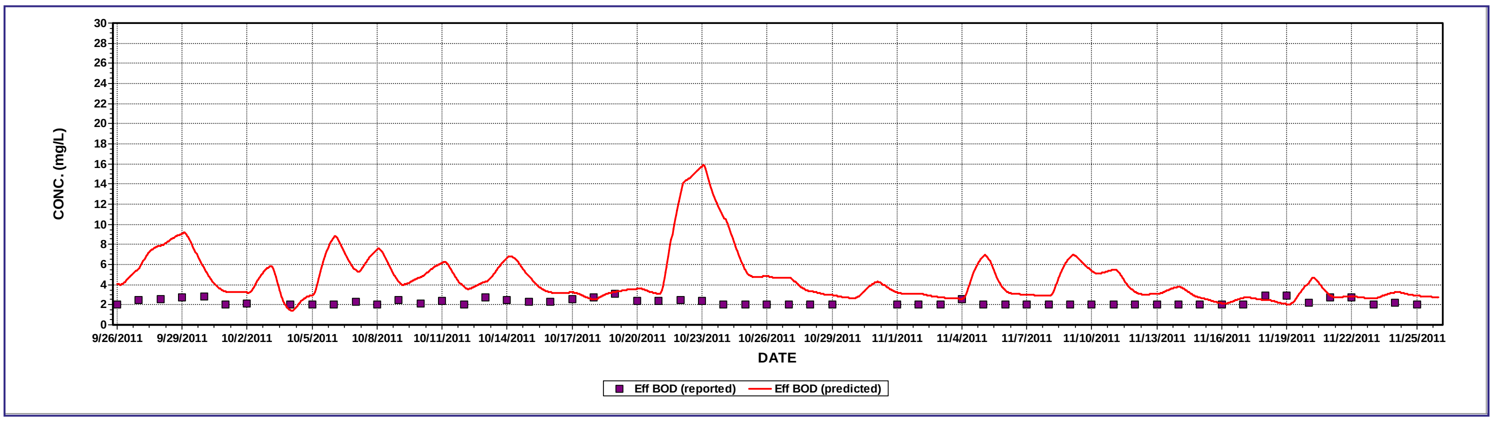

Figure 10 – Dynamic Simulation of Effluent BOD Concentrations for 2-month Validation Period

The model also closely predicts the effluent N species and effluent P, as shown in Figures 11 and 12, respectively. The calibrated and validated BioWin simulator for the SWWRF would be used in the plant capacity assessment.

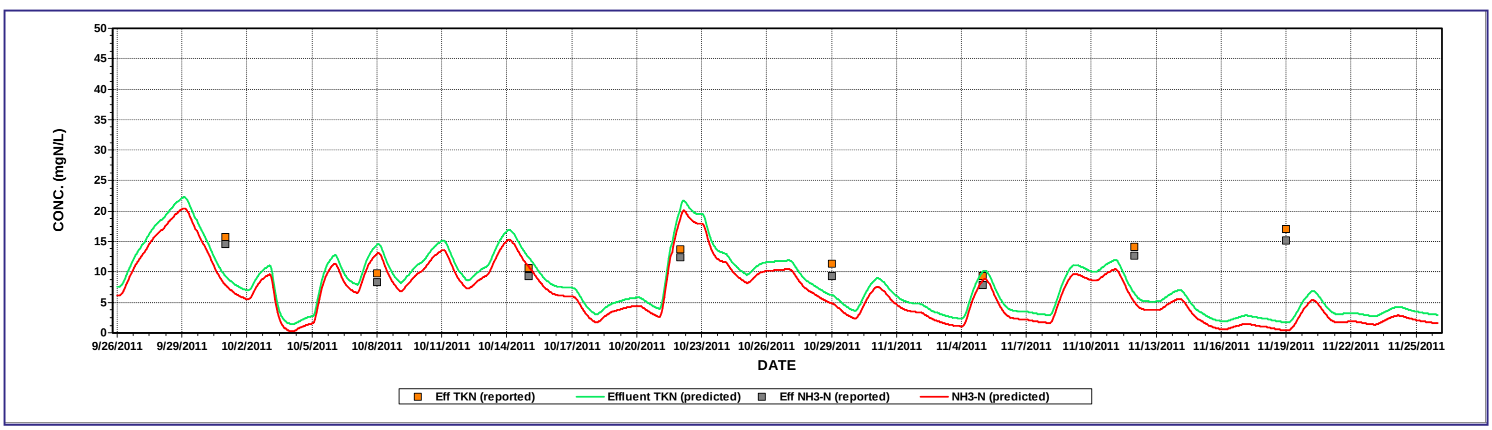

Figure 11 – Dynamic Simulation of Final Effluent TKN and NH_3-N Concentrations for 2-month Validation Period

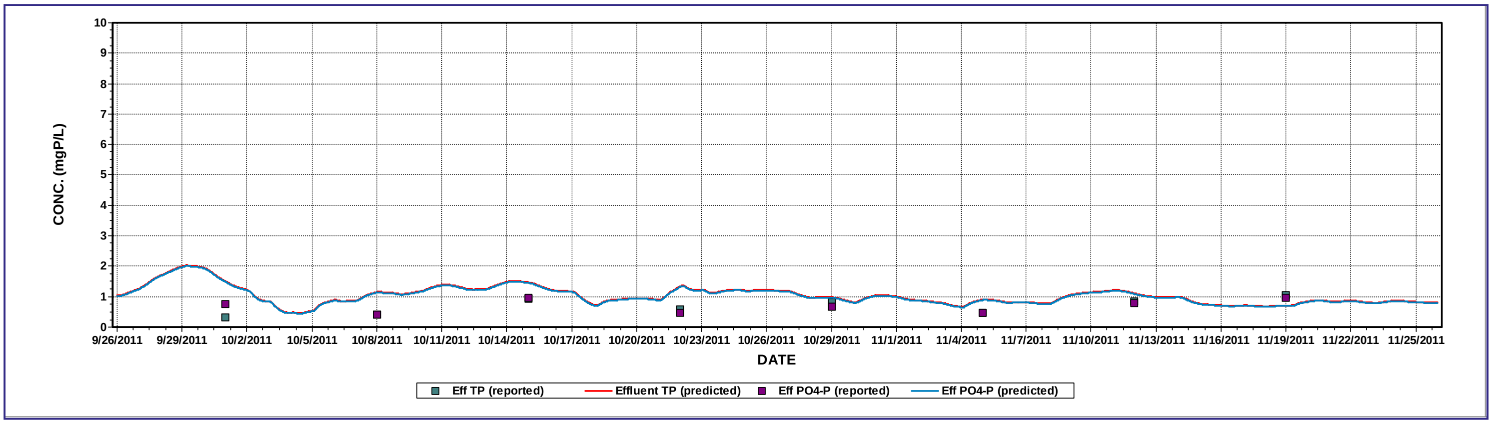

Figure 12 – Dynamic Simulation of Final Effluent TP and PO_4-P Concentrations for 2-month Validation Period

4. Conclusion

The calibration of the BioWin model for the SWWRF was performed based on historical and special sampling data. Based on data available for calibration, it is considered that the BioWin model predictions generally agreed well with the plant performance during the special sampling and the validation period.

In summary, the wastewater fractions calculated based on the special sampling data appear to adequately characterize the influent at the SWWRF.

In terms of effluent predictions, the model appears to adequately predict effluent TSS, BOD, ammonia and total phosphorus concentrations.

It is considered that the calibrated model can be used legitimately for the purpose the capacity assessment and analysis of future scenarios of the SWWRF.

Appendix F: Secondary Clarifier Hydrodynamic Model Calibration and Assessment

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Technical Memorandum

Project Title: City of St. Petersburg's Southwest Water Reclamation Facility Process and Hydraulic Evaluation

Project No: 142081

Technical Memorandum [No. 5]

Subject: Secondary Clarifier Modeling – Calibration and Capacity Assessment

Date: December 20, 2011

From: FILE

To: Gabriel Retana and Jose Jimenez

Limitations:

1. Introduction

This Technical Memo (TM) provides a summary of results of the calibration and capacity assessment of the secondary clarifiers at the SWWRF.

Table 1 presents the general characteristics of the secondary clarifiers at the SWWRF. The "new plant" treatment train includes three circular clarifiers. Clarifiers # 1 and # 2 were both constructed as part of the large expansion that was completed in 1978 and were retrofitted around 1996 when Clarifier # 3 was added. All three clarifiers are circular with diameters of approximately 135 feet. Clarifier #3 has the same surface area as the original two but it is deeper and has a different effluent launder system.

Table 1. Secondary Clarifier Characteristics

Parameter	Clarifiers 1 & 2	Clarifier 3
Clarifier Diameter, ft	135	135
Depth of Outer Wall, ft	12	15
Centerwell Diameter (Internal), ft	16	16
Centerwell Depth, ft	7	7
Effluent Launders	Inboard and Outboard	Outboard
Sludge Collection	Suction - Organ Pipe	Suction - Organ Pipe

2. Overview of the Clarifier Model

The 2DC model used for the secondary clarifier analysis was developed by a research team led by Professor J. Alex McCorquodale and coworkers at the University of New Orleans, Louisiana. The model accounts for axi-symmetric hydrodynamics (including swirl components), sludge settling, turbulence, sludge rheology, flocculation, clarifier geometry, and varying hydraulic loadings. Discrete particle settling, flocculation-induced settling, hindered settling, and compression settling also are described by the model. The model is the most advanced clarifier CFD model available for circular clarifiers in the world. Model inputs include: mixed liquor settling and flocculating characteristics, discrete settling fractions, secondary clarifier geometry, SOR, temperature, mixed liquor suspended solids (MLSS) concentration, and RAS flow rate. The mixed liquor settling and flocculating characteristics and the discrete settling fractions must be determined on site using field and laboratory methods. Using these inputs, the model predicts ESS and RAS suspended solids (RSS) concentrations. In addition, the model output can predict flow velocity vectors and solids concentrations throughout a two-dimensional, vertical slice of the clarifier. Sludge blanket depth (SBD) also can be determined from the solids concentration profile. For all 2DC modeling, we assumed that the mixed liquor splitter box operated as it was designed and flow splitting was equal between individual clarifiers.

3. Field and Laboratory Data Collection Methods

As part of the clarifier modeling and capacity assessment of the clarifiers at the SWWRF, a field program was designed to develop information useful for CFD model calibration and verification. In general, the protocols followed those in the "WERF/CRTC Protocols for Evaluating Secondary Clarifier Performance[i]". The field and laboratory data collection program was conducted on November 16 and 17, 2011.

3.1 Mixed Liquor Settling Characteristics

Batch settling tests were performed on various concentrations of mixed liquor and RAS to determine the settling characteristics of the mixed liquor. Figure 1 shows the experimental setup at the SWWRF as recommended in the WERF/CRTC Protocols. Figure 2 presents the summary results for the settling tests conducted at the SWWRF.

Figure 1. Experimental Steup to Determine the Mixed Liquor Settling Characteristics

The Vesilind equation was used to determine the sludge settling properties during the settling tests and is described by Equation 1.

$$V_S = V_o e^{-kX} \qquad (1)$$

where:

V_S = interface settling velocity, m/h

X = solids concentration, g TSS/L

Vo = sludge-specific settling parameter, m/h

k = sludge-specific settling parameter, L/g TSS

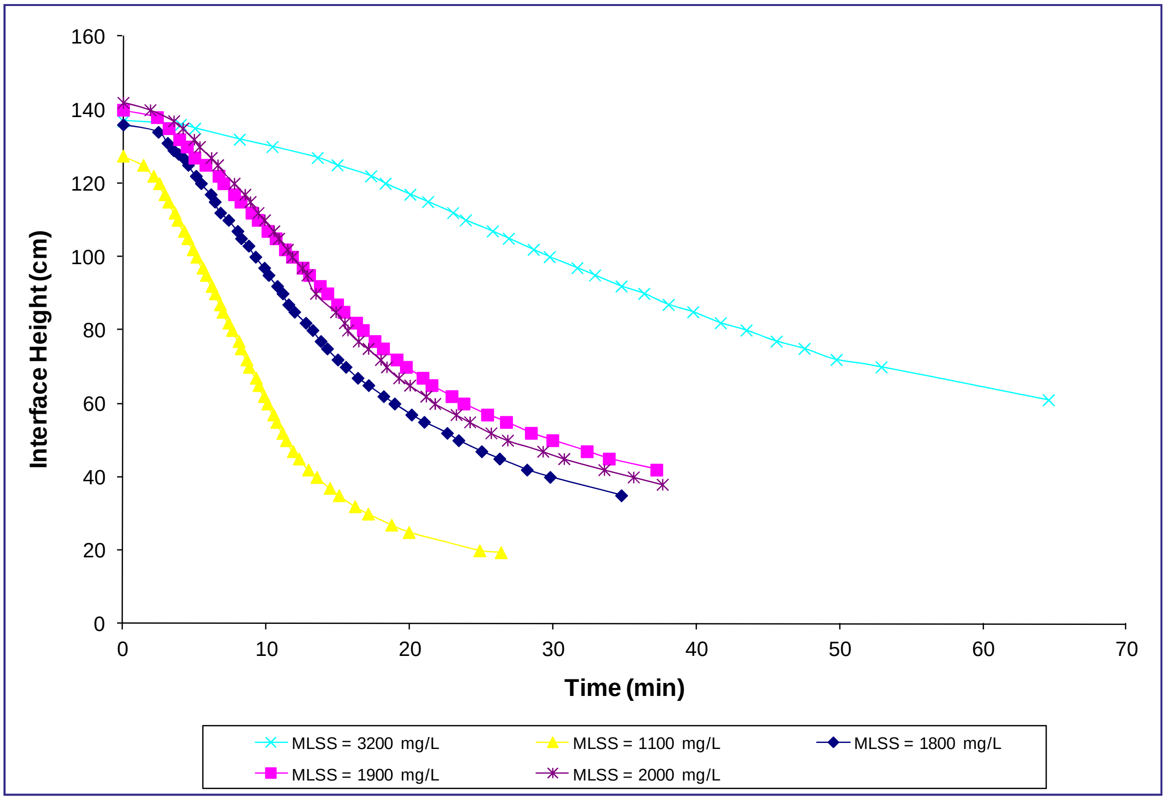

Figure 2. Settling Test Results at the SWWRF

3.2 Mixed Liquor Flocculation Characteristics

To determine the flocculation characteristics of the mixed liquor, jar test experiments were performed on site. A six-paddle stirrer (Phipps and Bird Stirrer) was used to flocculate the mixed liquor samples. Flocculation was induced mechanically by stirring the sample. Square jars (2 L) were used for the flocculation tests. The flocculation jars were filled with a 1.8-L mixed liquor sample with minimal delay. Each jar was randomly assigned a flocculation time. After the prescribed flocculation time had elapsed, the stirrer was removed carefully from the jar. After 30 min of settling, supernatant samples were withdrawn from the jars for analysis. Figure 3 shows the experimental setup for determining flocculation characteristics.

Figure 3. Jar Test Experiment Setup for Determining Flocculation Characteristics

The flocculation characteristics were determined by fitting Equation 2 to the experimental data. The flocculation characteristics of the mixed liquor used for the model were defined by K_A and K_B from Equation 2. Figure 4 presents the average results from the flocculation experiments.

$$n_t = \frac{K_B G}{K_A} + \left(n_o - \frac{K_B G}{K_A} \right) e^{-K_A X G t} \qquad (2)$$

Where:

n_t	=	number of particles at time t, g/L
n_o	=	initial number of particles, g/L
G	=	root-mean square velocity gradient, s^{-1}
X	=	mixed liquor concentration, g/L
K_A	=	floc aggregation rate coefficient, L/g
K_B	=	floc break-up rate coefficient, s
t	=	time, s

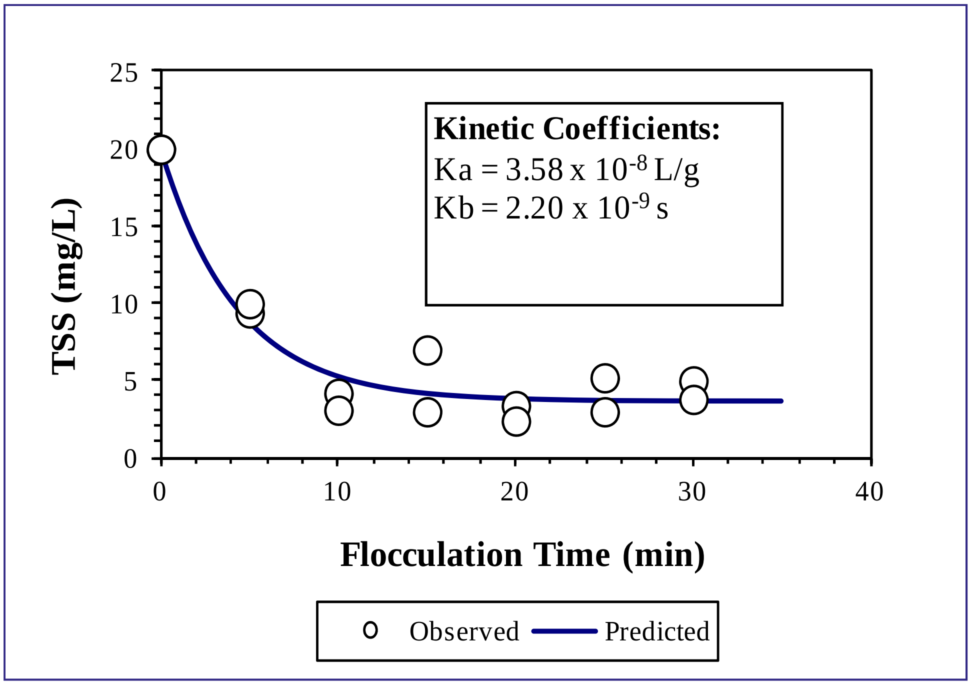

Figure 4. Flocculation Results and Kinetic Coefficients Determined During the Sampling Phase at the SWWRF

4. Hydrodynamic Model Calibration

A site visit was performed on November 16, 2011 to collect sludge settling and flocculation characteristics for model calibration. Table 2 summarizes the mixed liquor settling and flocculating characteristics determined during the sampling program. These values were used for model calibration.

Table 2. Summary of mixed liquor settling and flocculating characteristics determined from field and laboratory analyses for model calibration	
Parameter	Value
Hindered settling constants	Vo = 11.14 m/hr k = 0.77 L/g Vc = 3.90 m/hr kc = 0.27 L/g (SVI = 83 mL/g)
Floc aggregation rate coefficient	3.58×10^{-5} L/g
Floc breakup rate coefficient	2.20×10^{-9} s

As a part of the calibration, on November 17, 2011, a stress testing was performed to Clarifier # 1. Figure 5 depicts a cross-section of Clarifier # 1. This information was used to build the hydrodynamic clarifier model for the test clarifier. The stress testing for Clarifier # 1 was conducted by closing the gates located in secondary clarifier splitter box so that flow to the test clarifier was incrementally increased. The RAS flow for the test clarifier was held constant at approximately 6 MGD. The sludge blanket depth (SBD) of the test clarifier was monitored and ESS, RSS and MLSS samples were taken every 15 minutes. Figure 6 shows surface overflow rate (SOR) and solids loading rate (SLR) values for test clarifier during the stress testing. The average MLSS concentration during the stress testing was 1,550 mg/L.

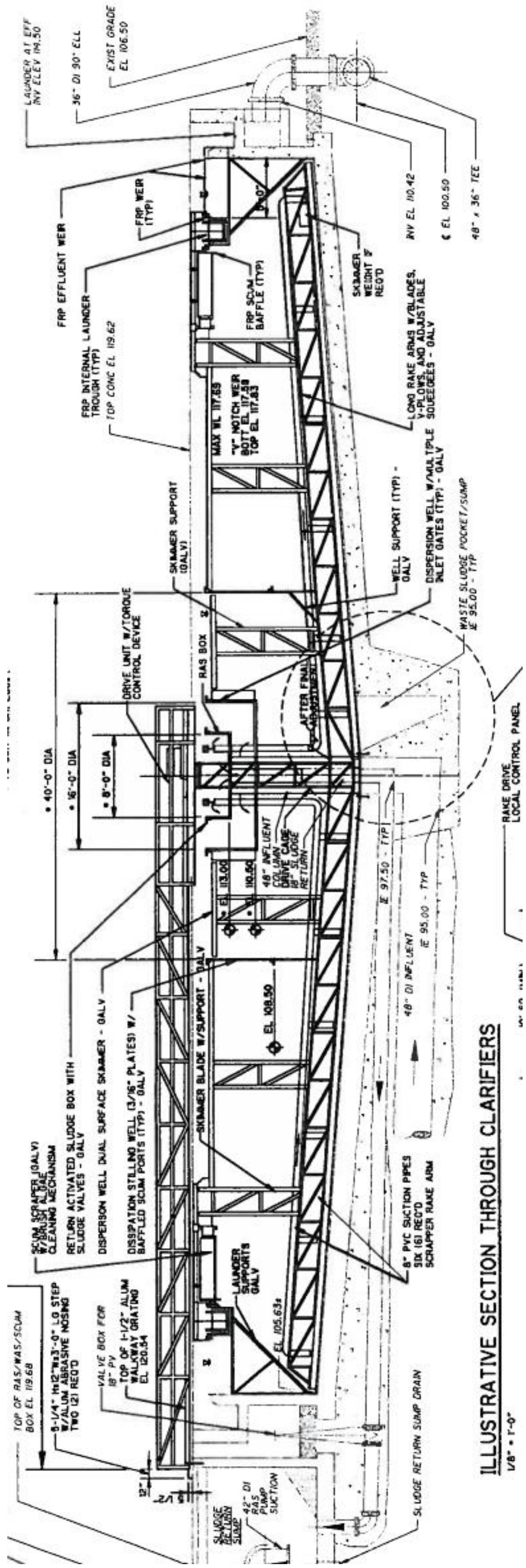

Figure 5. Cross Section of Clarifier # 1

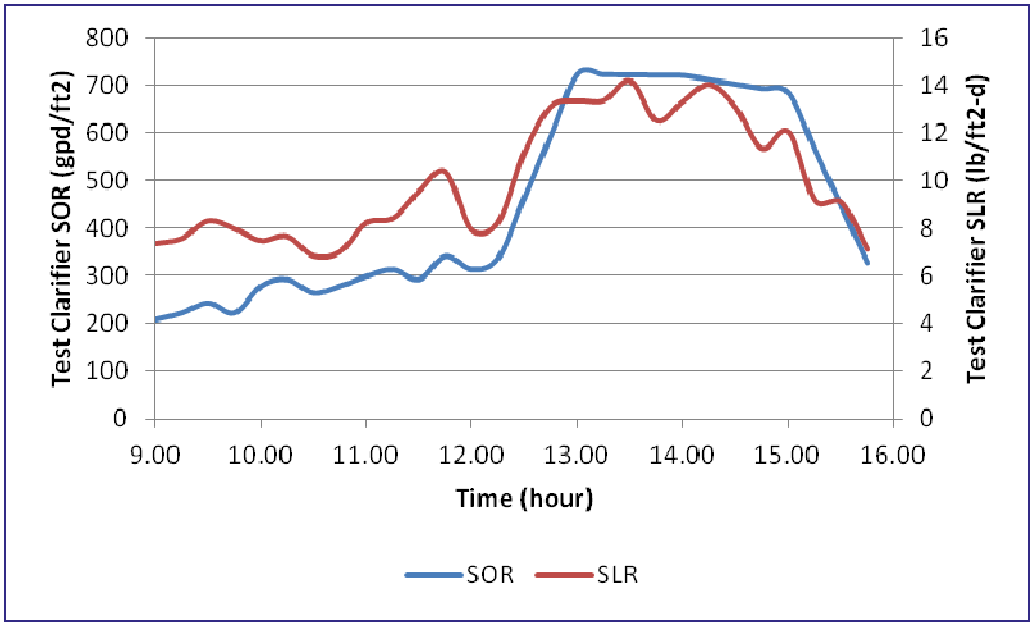

Figure 6. SOR and SLR Values During Clarifier # 1 Stress Tests on November 17, 2011

Figure 7 and Figure 8 show the results of the model-predicted and observed values for ESS and SBD, respectively. The model predicts slightly higher values of ESS than what was observed in the field. However, the difference between the model predicted and the observed values is considered insignificant. Figure 8 shows the SBD measurements and those predicted by the model. The trend in SBD matches very closely between the field measured data and the model predicted values. Figure 9 presents a screen-shot of the CFD model output for Clarifier # 1 during the calibration. Based on these results, the hydrodynamic model was considered calibrated and ready to be used to assess the capacity of the secondary clarifiers.

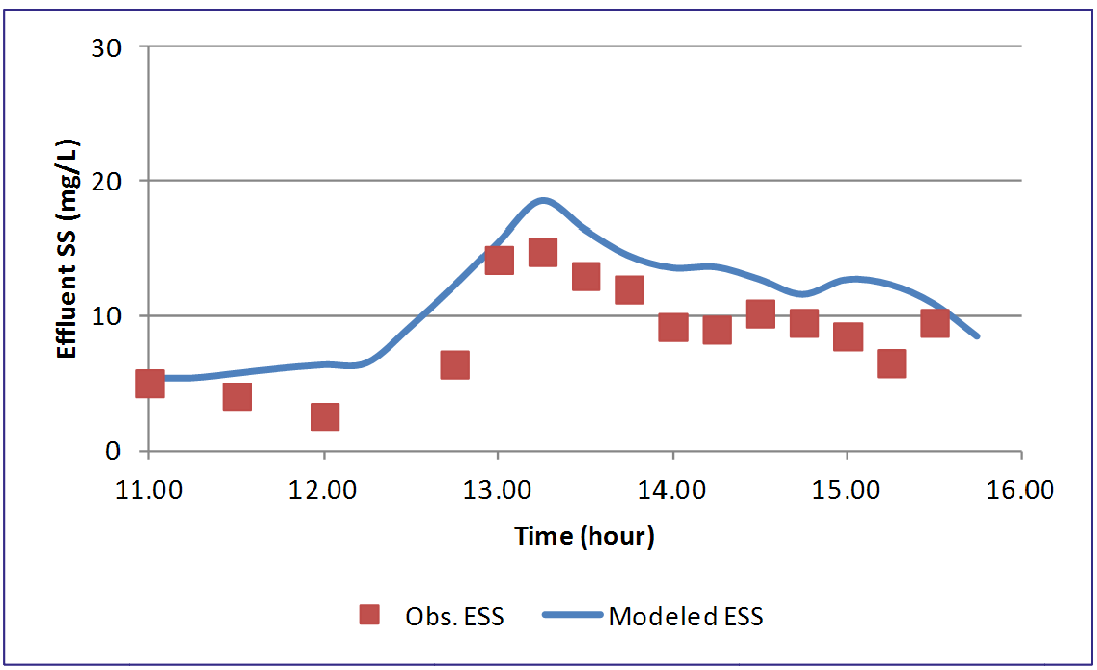

Figure 7. Field Measured and Model Predicted ESS for Model Calibration

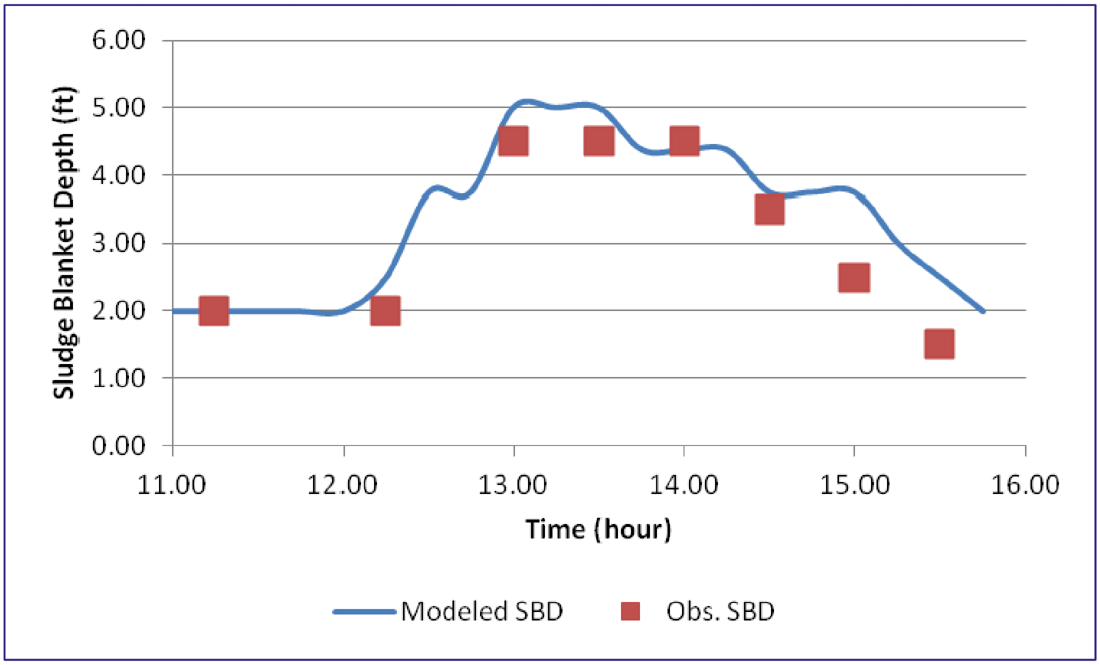

Figure 8. Field Measured and Model Predicted SBD for Model Calibration

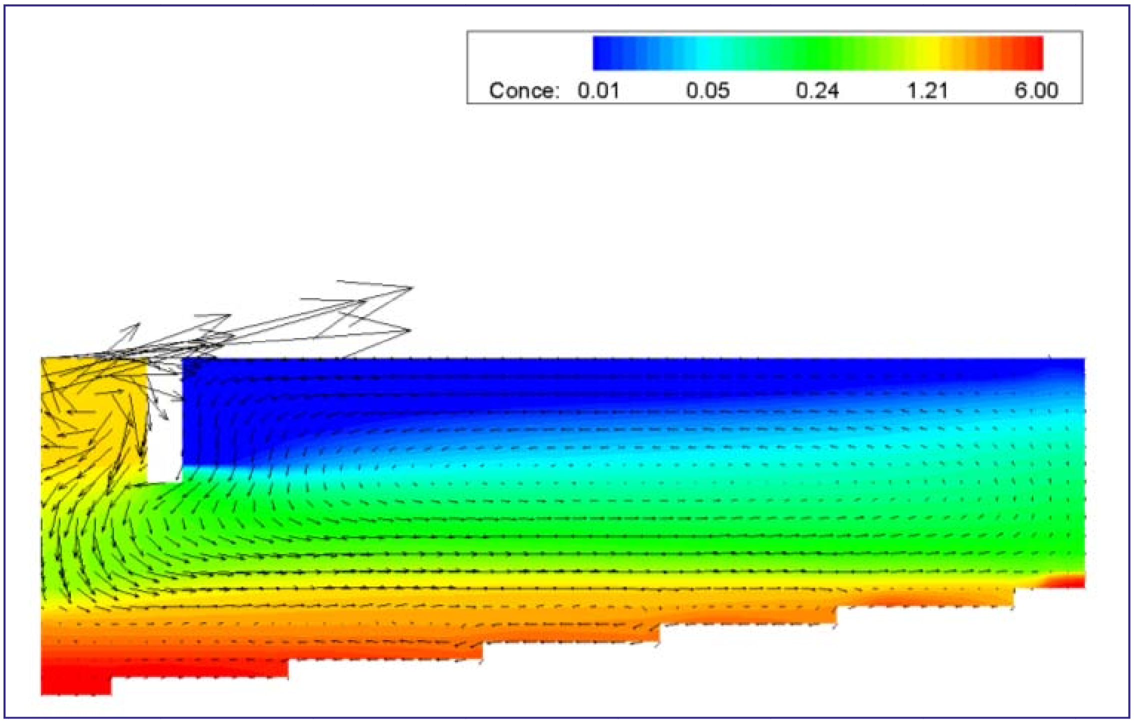

Figure 9. CFD Model Output for Clarifier # 1 for Model Calibration

5. Capacity Assessment

After model calibration, the calibrated model was used to determine the capacity of the existing clarifiers. A peak ESS concentration of 60 mg/L was used to define clarifier failure which corresponds to the grab sample requirement value in the existing permit.

One of the most important factors when assessing the capacity of secondary clarification system is the selection of the design mixed liquor settling characteristics. Often, historical operation data on SVI is available and can be used as reference or guideline in the selection of the design SVI value. There is an inherent uncertainty in sizing clarifiers based on historical SVI data, although Brown and Caldwell recommends using the 90-percentile value. Based on the January 2007- November 2011 data, average and the 90-percentile SVI values for the SWWRF are approximately 205 mL/g and 270 mL/g. These values are considered high and might be the result of low DO bulking conditions in the aeration basis. With the modifications of the existing aeration basins, including new anaerobic selectors and new aeration systems, it is expected that the quality of the mixed liquor be improved dramatically. An analysis of SVI data from facilities similar to the SWWRF but with adequate aeration control indicated that 90-percentile SVI values ranging from 120 to 150 mL/g can be achieved. Therefore, for the purpose of this analysis, this SVI range was adopted for the SWWRF. It should be noted that Brown and Caldwell recommends the following in order to minimize high SVI values:

- Effective automatic DO control to minimize the risk of low DO bulking.
- The ability to add polymer to enhance settling rates during high flow events.

Table 3 shows the assumptions used for the capacity assessment based on a SVI of 120 mL/g. The RAS flow was assumed to be 18 MGD for all four clarifiers, or 6 MGD per clarifier. The settling constants were determined based on the assumed 90th percentile SVI value of 120 mL/g. The SVI value was converted to Vo and k values using the empirical relationship developed by Wahlberg and Keinath (1995).

Table 3. Assumptions used for secondary clarifier capacity assessment

Parameter	Value
Settling Constants	**V_o = 8.84 m/h** **k = 0.42 L/g** **v_c = 3.09 m/h** **k_c= 0.15 L/g** **(SVI = 120 mL/g)**
Temperature	**25 degrees C**
RAS Flow	**6.0 mgd**
MLSS	**2000mg/L – 4000 mg/L**

Figure 10 shows an example of the model input used for each flow condition. This information corresponds to the maximum day loading condition to the secondary clarifiers at an annual average flow of 20 MGD and a peak hour flow of 40 MGD. For the purpose of this analysis, the model was base loaded for 4 hours; the clarifier starts empty and base loading fills the clarifier with sludge so that a representative base load condition exists in the clarifier prior to the storm event.

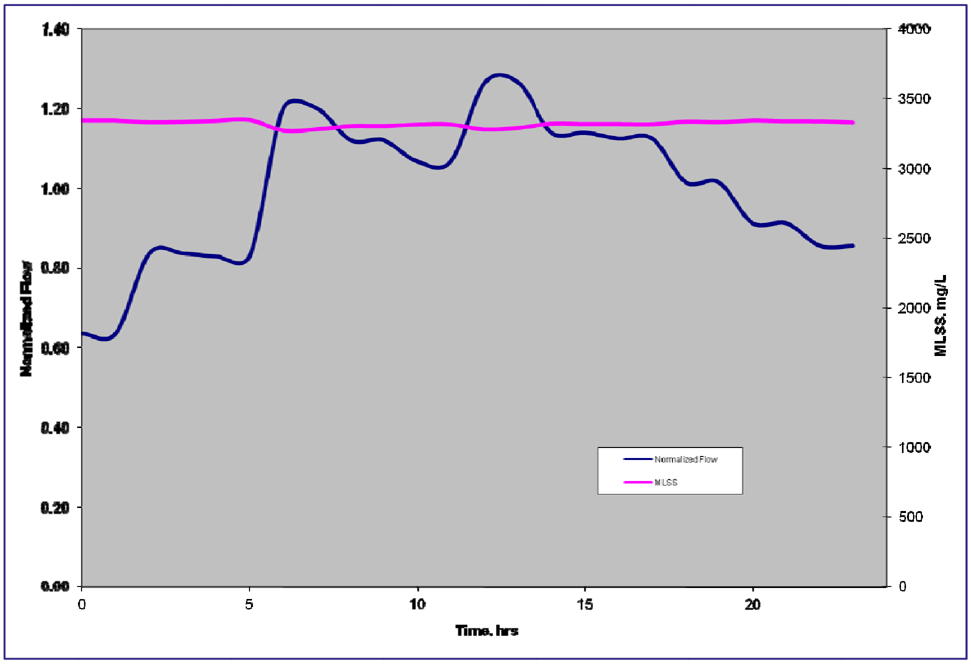

Figure 10. Example of Model Input Used for Capacity Assessment.

Table 4 summarizes the CFD modeling results for the secondary clarifier assessment of the secondary clarifiers at the SWWRF. As expected, the capacity of the secondary clarifiers is significantly affected by the selection of the SVI value; the higher the SVI, the lower the capacity of the clarifiers would be. Based on the information presented in Table 4, the capacity of the secondary clarification system at the SWWRF varies between 14.5 and 17 MGD as annual average daily flow or 29 to 34 MGD as peak hour flow. This table also presents, for reference purposes, the capacity of the secondary clarifier if no improvements in the SVI are achieved. It should be noted that the capacity at the existing SVI is less than the current flow because the mixed liquor concentration was kept at 3,600 mg/L which is approximately 63 percent higher than the current values. As the CFD modeling results indicate, the capacity of the secondary clarification system at the “new plant” does not have adequate capacity to handle the SWWRF’s rated capacity of 20 MGD as annual average flow. However, based on the original design of the “new plant”, this was designed to treat an average flow of 16 MGD, not 20 MGD.

Table 4. CFD Modeling Results for the Capacity Assessment of the Secondary Clarifiers

Parameter	SVI of 270 mL/g	SVI of 150 mL/g	SVI of 120 mL/g
Average MLSS (g/L)	3.60	3.60	3.60
RAS (MGD)	18	18	18
Effluent SS (mg/L)	50	49	36
Sludge Blanket Height (% of total depth)	72	62	65
SLR (lb/d-fs)	22.5	32.85	36.35
SOR (gpd/fs)	326	675	792
Capacity [Average/ Peak] (MGD)	7/ 14	14.5/ 29	17/ 34

Figure 11 shows the velocity vector fields and the concentration distributions of the simulation with an average SVI of 120 mL/g. As depicted in this figure, the sludge blanket (defined by the dark orange color) builds up above of the center well skirt creating an area prompt to short-circuiting of the flow. This allows for the development of a very limited clarification zone between the sludge blanket and the surface of the clarifier. Therefore, the higher velocities on the top of the sludge blanket are scouring the solids from the blanket to the effluent, limiting the capacity of the secondary clarifiers at the SWWRF. In addition, the size of the existing center well is too small providing limited time for flocculation and energy dissipation.

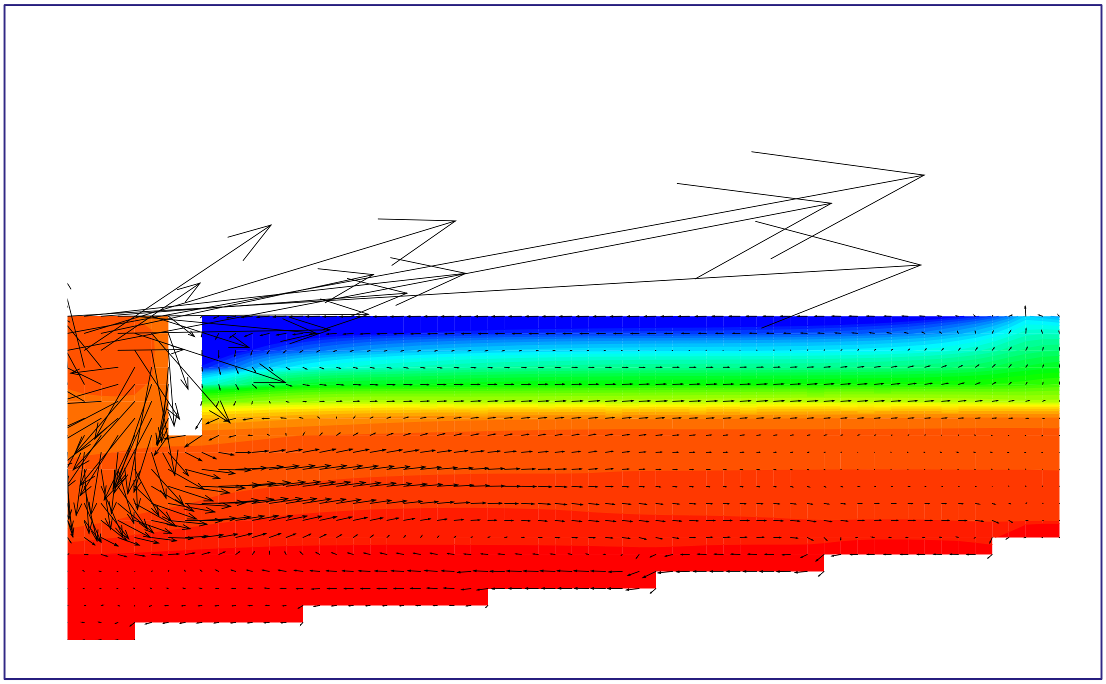

Figure 11. CFD model prediction for Secondary Clarifier # 1 at a design SVI of 120 mL/g - velocity vector field and solids suspended concentration at the maximum blanket level

As part of this assessment, physical modifications to the internal mechanisms of the secondary clarifiers were evaluated in order to increase capacity. Additional modeling was performed to determine if additional secondary clarifier capacity could be achieved by increasing the center wells of the secondary clarifiers. The flocculator center well is designed to dissipate energy of the incoming flow from the reactors as well as to provide contact and adequate detention time to promote flocculation of dispersed solids that may have broken up due to high degree of energy and flow conveyance between the reactors and the clarifiers. The existing center wells are sized smaller than modern design criteria for achieving optimal flocculation. The existing center wells are located at approximately 12 percent of the clarifier diameter and at 40 percent of total clarifier depth. Based on Brown and Caldwell's experience, the optimization alternative proposes a flocculator center well with a diameter of approximately 30 percent the clarifier diameter, and the skirt of the well to be extended to 50 percent of the total clarifier depth. For the purpose of this analysis, a design SVI of 120 mL/g was adopted. Table 5 summarizes the CFD modeling results for this alternative, which indicate that the changes to the existing center wells can provide significant capacity benefits at the SWWRF.

Table 5. CFD Modeling Results for the Capacity Assessment for Optimized Center Well

Parameter	Existing Center Well	Optimized Center Well
Center Well Diameter (%)	12	30
Center Well Depth (%)	40	50
Average MLSS (g/L)	3.60	3.60
RAS (MGD)	18	18
Effluent SS (mg/L)	36	45
Sludge Blanket Height (% of total depth)	65	60
SLR (lb/d-fs)	36.35	40.50
SOR (gpd/fs)	792	931
Capacity [Average/ Peak] (MGD)	17/ 34	20/ 40

Figure 12 shows the velocity vector fields and the concentration distributions of the clarifier simulation with the optimized flocculator center well at a design SVI of 120 mL/g.

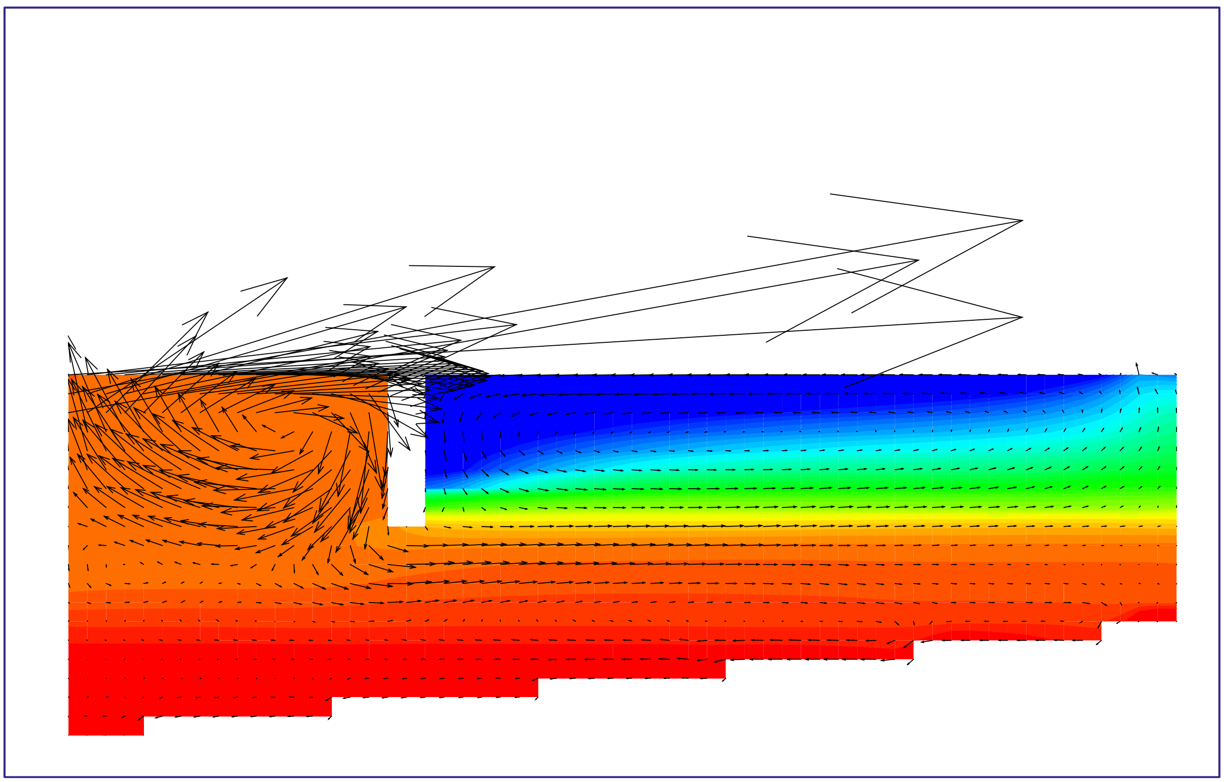

Figure 12. CFD model prediction for Secondary Clarifier # 1 with optimized center well and a design SVI of 120 mL/g - velocity vector field and solids suspended concentration at the maximum blanket level

The hydrodynamic model was then used to estimate the capacity gains of the secondary clarification system due to the reduction in mixed liquor levels due to process modifications such as step-feed or the addition of primary clarifiers. For the purpose of this analysis and similar to the analysis conducted previously, two SVI values were used, 120 mL/g and 150 mL/g. For the purpose of this analysis, no improvements to the secondary clarifier internal structures were considered. Table 6 summarizes the CFD modeling results. Based on the information presented in this table, the capacity of the secondary clarification system varies between 18 and 20 MGD if the mixed liquor can be significantly reduce. As the CFD modeling results indicate, the capacity of the secondary clarification system at the “new plant” can be increased to 20 MGD if the step-feed configuration is implemented.

Table 7-5. CFD Modeling Results for the Capacity Assessment for Alternative 1

Parameter	SVI of 150 mL/g	SVI of 120 mL/g
Average MLSS (g/L)	2.2	2.2
RAS (MGD)	18	18
Effluent SS (mg/L)	50	22
Sludge Blanket Height (% of total depth)	45	30
SLR (lb/d-fs)	31.46	33.80
SOR (gpd/fs)	838	931
Capacity [Average/ Peak] (MGD)	18/ 36	20/ 40

Figure 13 shows the velocity vector fields and the concentration distributions of the simulation with an average SVI of 120 mL/g for the alternative with lower mixed liquor concentrations. From this figure, one can observe as the sludge blanket (defined by the dark orange color) stays below the center well skirt providing adequate clarification area during peak flow conditions. Short-circuiting is still observed below the center well’s skirt resulting in high blankets and high effluent suspended solids.

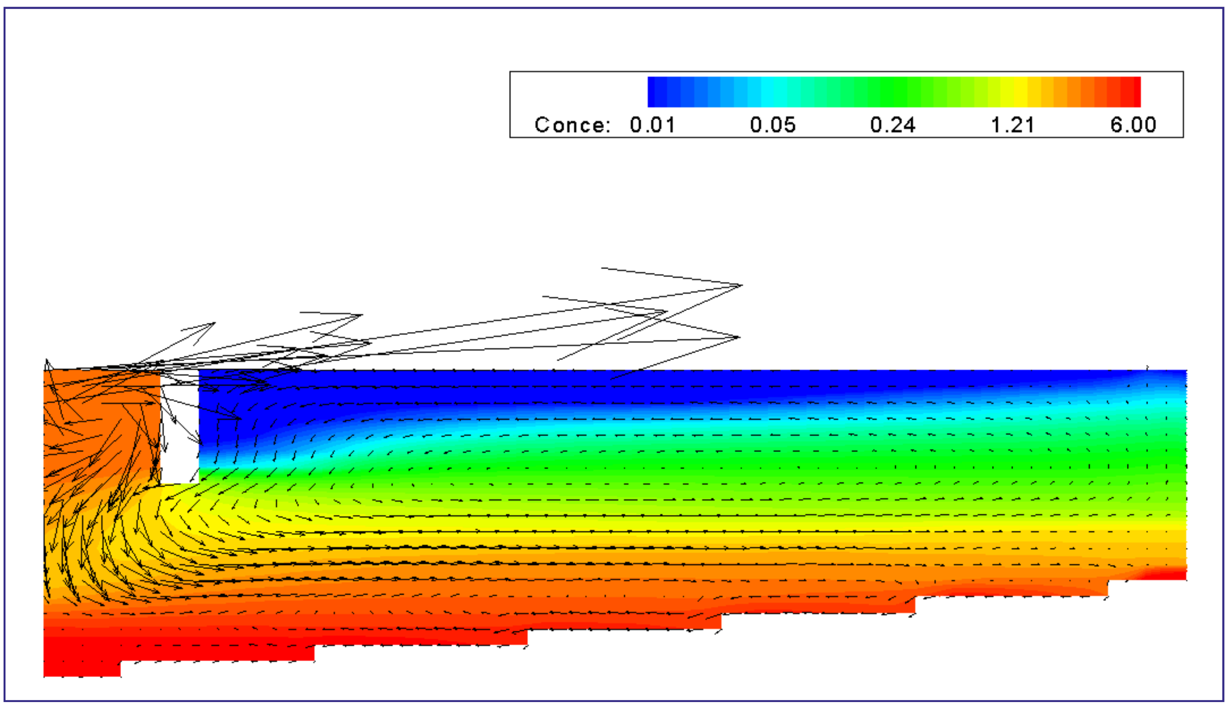

Figure 13. CFD model prediction for Alternative 1 at a design SVI of 120 mL/g - velocity vector field and solids suspended concentration at the maximum blanket level

6. Conclusions and Recommendations

6.1 Conclusions

- The calibration and capacity modeling helped to determine deficiencies in the existing secondary clarification system that limits capacity.
- The capacity estimates are all based on the Clarifier # 1.
- Using the 90th percentile SVI value and peak flow conditions the capacity of the existing clarifiers was determined to be 14.5 and 17 MGD for 150 and 120 mL/g, respectively. These values are based on an average MLSS of 3,600 mg/L predicted by the BioWin process model.
- Lower mixed liquor concentrations would also realize in higher capacity values.
- The capacity estimates assume that there is adequate flow splitting between the secondary clarifi-ers.
- Clarifier optimization showed that additional clarifier capacity can be realized by increasing the size of the flocculator center wells in all the clarifiers.
- The maximum capacity of the clarifiers, after the recommended modifications are in place, is 40 MGD total.

[i] Wahlberg, E.J. (2004) WERF/CRTC Protocols for Evaluating Secondary Clarifier Performance. Water Environment Research Foundation, Alexandria, VA.