Transmittal Cover Sheet

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To: Mr. Steven Marshall, P.E. Project Manager City of St. Petersburg One Fourth Street North St. Petersburg, FL 33701-3804	**Date:** 7/31/2014	**Transmittal No.:** 2
	Project No.: 145682	**Task No.:**
	Project Title: SWWRF Capacity and Liquid Treatment Project	
From: Todd Bosso, P.E.	**Contract No.:** 12-03-BC/W	

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No. of Copies	Revision Date or No.	Document or Drawing No.	Description
10	7/31/2014	1	Final Report – SWWRF Wet Weather and Liquid Process Capacity Assessment
1	7/31/2014	2	CD containing PDF of Final Report – SWWRF Wet Weather and Liquid Process Capacity Assessment

Remarks:

Enclosed is the final report for the SWWRF Wet Weather and Liquid Process Capacity Assessment project. We have corrected the hydraulic capacity as identified in Section 5. It was shown as 41.5 MGD in the Draft Report and has been updated to 40.5 MGD in the Final Report. This change was made because the limiting unit process at that flow was the filters. If you have any questions feel free to contact me at 813.371.9318. We appreciate the opportunity to work with you on this project and look forward to working with you again in the future.

cc: File	**Prepared by:** Todd Bosso, P.E. **Title:** Project Manager

City of St. Petersburg's
Southwest Water Reclamation Facility
Wet Weather and Liquid Process Capacity Assessment

Prepared for
City of St. Petersburg, Florida
July 31, 2014

Brown AND Caldwell
5405 Cypress Center Drive, Suite 250
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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 Water Reclamation Facility
BC	Brown and Caldwell
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
FOG	fats, oils and grease
ft	feet
g/L	grams per liter
GBT	gravity belt thickener
gpd	gallons per day
gph	gallons per hour
gpm	gallons per minute
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
lb/d	pound per day
MDF	maximum daily flow
MG	million of gallons
mg/L	milligrams per liter
MGD	million of gallons per day
mL/g	milliliter per gram
MLSS	mixed liquor suspended solids
MMF	maximum month flow
MOP	manuals of practice

n/a	not applicable
NEWRF	Northeast Water Reclamation Facility
NH_3-N	ammonia as nitrogen
NH_4	ammonium ion
NO_2-N	nitrite as nitrogen
NO_3-N	nitrate as nitrogen
NWWRF	Northwest Water Reclamation Facility
pCOD	particulate chemical oxygen demand
PDR	Preliminary Design Report
PHF	peak hour flow
PO4-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 phosphorus
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
Vo	initial reference settling velocity
VS	average volatile solids
Vs	interface settling velocity
VSS	volatile suspended solids
WAS	waste activated sludge
WRF	water reclamation facility

Section 1

Introduction

The Southwest Water Reclamation Facility (SWWRF) is one of the four water reclamation facilities owned and operated by the City of St. Petersburg (City). The facility 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. 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 preparing to decommission Albert Whitted Water Reclamation Facility (AWWRF) from service and to divert the associated wastewater generated in the southeast section of the City to the SWWRF. The plans include converting the AWWRF to a pump station and to divert all of its wastewater to the SWWRF for treatment and disposal. In addition, it is planned to build new sludge process units to handle the solids originated from both Northeast (NEWRF) and Northwest (NWWRF) Facilities at the SWWRF. As an effort to be proactive and to plan for future needs, the City desired to investigate the peak wet weather treatment capacity of the SWWRF and to determine possible hydraulic and process limitations that could restrict the facility for handling the additional flows and pollutant loadings generated from the AWWRF's service area and the waste activated sludge from NEWRF and NWWRF during abnormal events.

1.1 Scope

A scope of work was developed by Brown and Caldwell (BC) 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 level recommendations to eliminate hydraulic and treatment process bottlenecks at the SWWRF to handle the projected peak wet weather influent flow of 69 MGD. A combination of historical data analysis, hydraulic and process (BioWin™ and computational fluid dynamic, CFD) modeling were used to assess the treatment capacity of the processing units at the SWWRF.

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. Field and laboratory testing were conducted collaboratively with City staff to ensure that the process models could replicate existing and future operating conditions for the SWWRF. The following activities were conducted as part of the plan:

- Extensive analysis of historical operation data from 2007 through 2013.
- Special sampling campaign conducted during August-November 2011 to determine the influent wastewater characteristics.
- Field tests conducted in 2011 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 conducted in 2011 to determine existing process capacity.
- Development, calibration and validation of the secondary clarifier CFD models in 2011.
- Second special sampling campaign conducted in August 2013 to document the performance of the biological reactors at the SWWRF at the low dissolved oxygen (DO) operating conditions.
- Extensive field and batch test experiments conducted by EnviroSim Ltd. (a sub-consultant to BC) in March 2014 to understand the nitrogen and phosphorus removal pathways at the low DO conditions.

1.3 Objectives

The objectives of the treatment process assessment included the following components:

1. Establish the existing wet weather treatment capacity for the SWWRF to meet the existing effluent requirements.
2. Determine the capacity beyond current flow and pollutant loadings to accommodate future conditions.
3. List planning level improvements and/or operational strategies needed to increase the capacity of the SWWRF to handle the future projected wet weather flows.

Section 2

Processes

2.1 General Overview of Existing Facilities

The SWWRF is located at 3800 54th Avenue South. 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 preliminary treatment, secondary treatment, effluent filtration, and disinfection. Final effluent is distributed as reclaimed water to a public access urban reuse irrigation system and to Eckerd College for its use as cooling water. Deep-well injection is used as the backup effluent disposal method.

In general, the existing liquid treatment facilities at the SWWRF include influent screening; grit removal; activated sludge process and secondary clarification; deep bed filtration; and disinfection with sodium hypochlorite. The plant is permitted for an AADF of 20 MGD. The plant recently underwent a re-permitting process where aeration basin modifications were performed which allowed the plant to decommission the *old plant* while maintaining its 20-MGD AADF permitted capacity.

Residuals treatment currently includes gravity belt thickening, anaerobic digestion, and dewatering by belt filter presses. Residuals had historically been treated to Class B standards and land applied by a contract hauler.

Figure 2-1 provides an aerial view of the existing SWWRF with the major process units identified. Figure 2-2 provides a simplified process flow schematic for the existing treatment facility. Table 2-1 provides a summary of the design data for the major existing unit processes at the treatment facility. The existing facilities are grouped into the following categories and discussed in detail in the report titled *City of St. Petersburg's Southwest Water Reclamation Facility Treatment Process and Hydraulic Evaluation Capacity* prepared by BC and submitted on March 28, 2013.

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

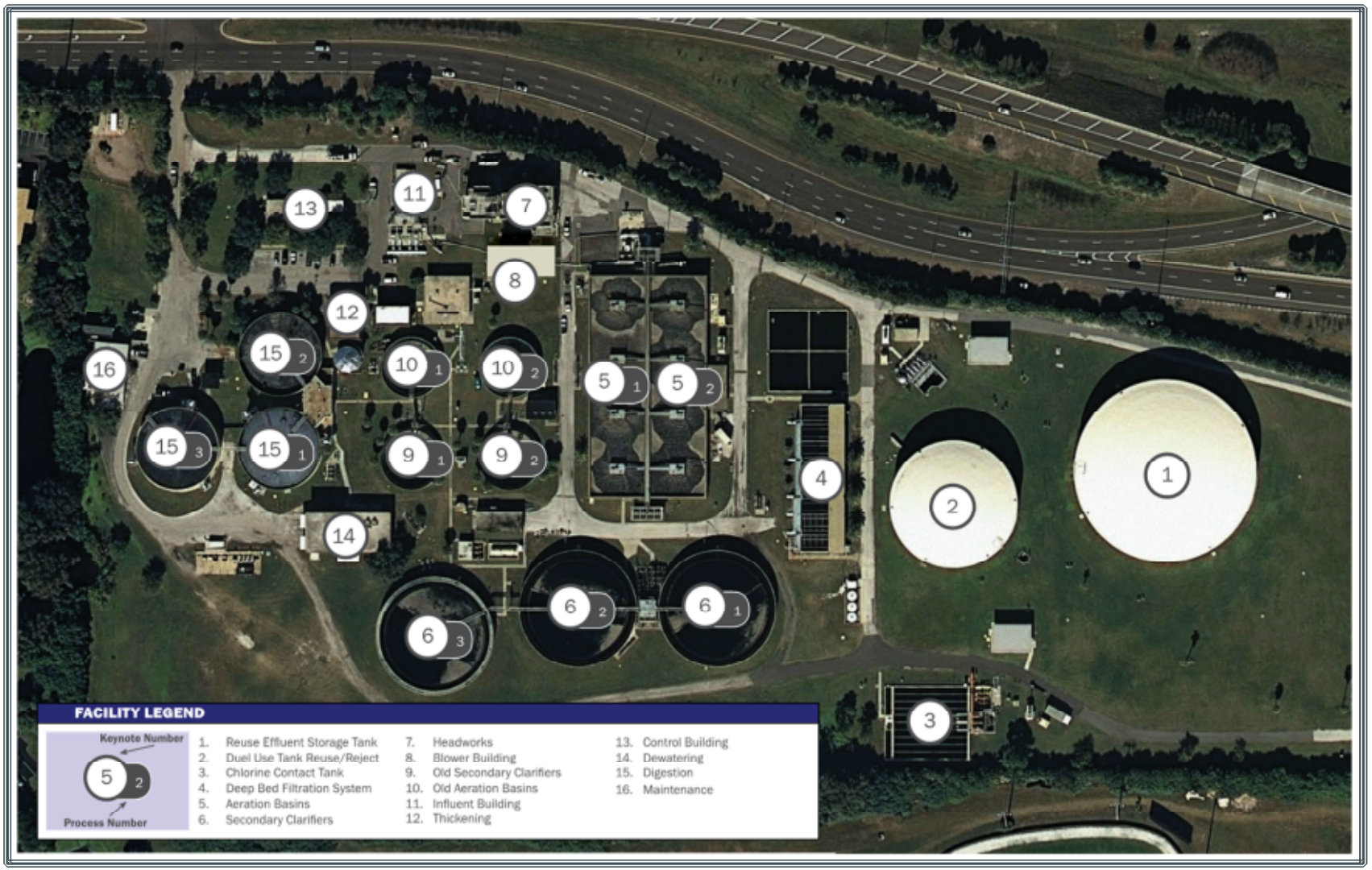

Figure 2-1. Aerial View of the Existing Facilities at the SWWRF

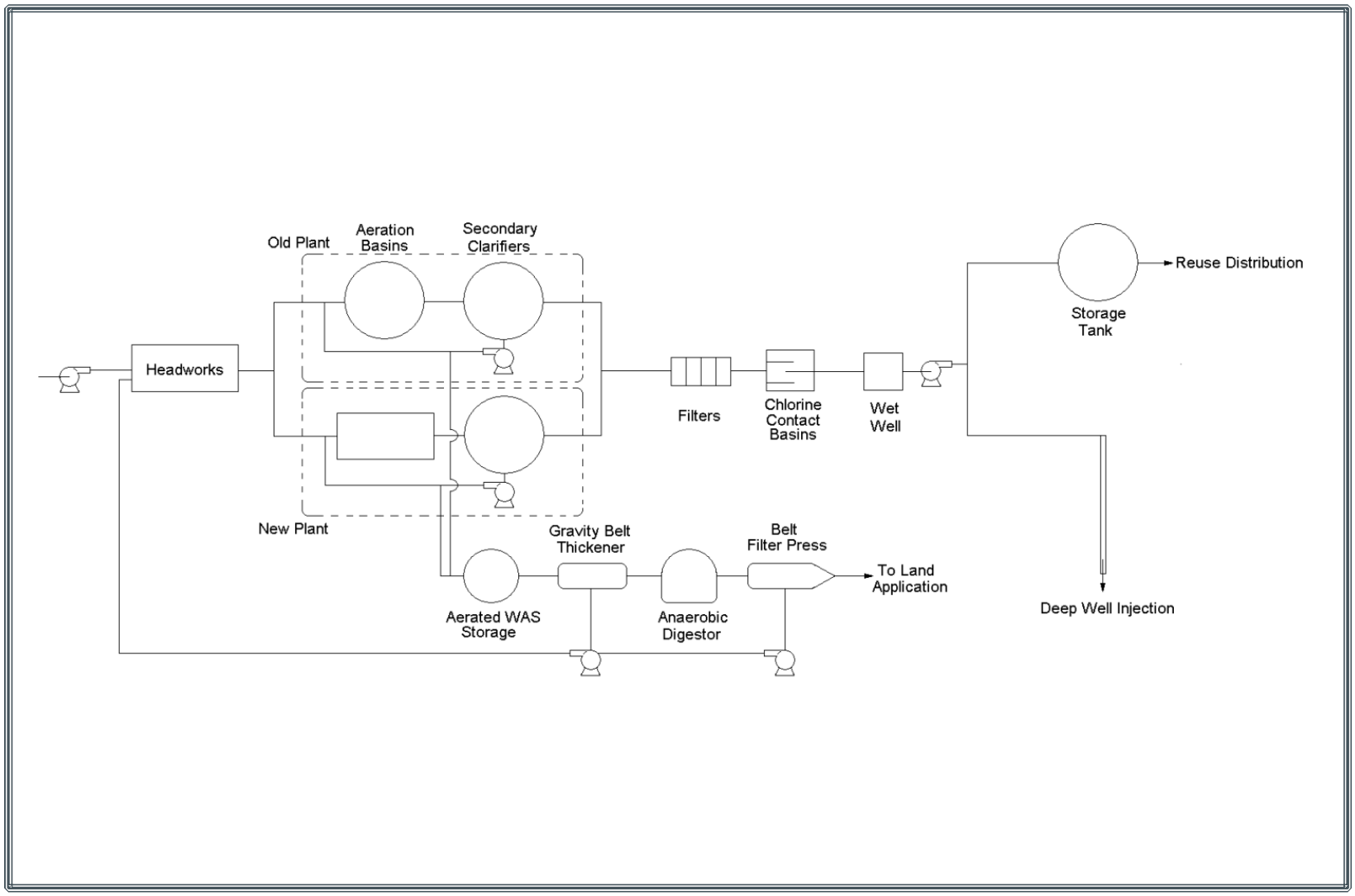

Figure 2-2. Simplified Existing Process Flow Diagram for the SWWRF

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	Number	4	
	Horsepower	HP	150	
Grit Removal	Low Energy Centrifugal	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 have different SWD.
	Total Clarifier Volume	MG	3.85	
	RAS Pump Capacity	GPM	3 pumps at 4,200	
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	--	2	
	Volume	MG	5, 10	
Effluent Pumping	Pumps	Number	5	
	Capacity	MGD	39.6	
Deep Wells	Capacity	MGD	27	

Table 2-1. Summary of Existing Unit Processes at the SWWRF				
	Parameter	Unit	Value	Notes
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 (currently not operating)
Anaerobic Digestion	Number of Tanks	--	3	Currently not operating
	Diameter	Ft	100	
	Side Water Depth	ft	22.5	At high level
	Volume	MG/each	1.3	
Belt Filter Press	Number of Units	--	2	
	Size	m	2	Belt width

2.2 General Overview of New Facilities

As part of the ongoing Biosolids Improvement's Program at the SWWRF new facilities associated will be constructed and these are provided in Figure 2-3. Additional information regarding the new facilities was presented in the draft report titled *Biosolids to Energy Preliminary Design Report (PDR)* prepared by BC and submitted on September 16, 2013. As presented in this PDR document, the Biosolids Improvements Program at the SWWRF consists of several discrete projects as identified below:

1. New splitter structure to divert influent flow to new primary clarifiers and to provide liquid stream process flexibility to utilize a new step feed and biological stabilization feature for the aeration basins,
2. Two new primary clarifiers and support facilities to capture additional sludge from the NWWRF and NEWRF for new anaerobic digestion processes,
3. Two new anaerobic digesters, a digested sludge batch tank, minor improvements to an existing digester, and support facilities to achieve Class AA biosolids,
4. New supplemental feedstock receiving facility for the digesters, envisioned to initially consist of limited amounts of fats, oils and grease (FOG) but with the capability of receiving other feedstock for potential inclusion (such as pulped food waste),
5. New digester gas handling facilities to provide momentary gas production peak storage and flare excess digester gas,
6. New digester gas upgrade systems to upgrade the quality of the digester gas to pipeline quality with a final compression system for transport via tube trailers to end-users, and
7. New odor control facilities for the primary clarifiers, and the sludge storage tank and thickening facilities (which includes the Thickening Project).
8. New engine facility to provide duty use of natural gas or upgraded digester gas to provide partial power needs for the plant (design by Black and Veatch),
9. New hot water boiler supply system for the anaerobic digesters, and
10. New electrical building to support new engine facility and other Biosolids Improvements Program efforts (design by Black and Veatch),
11. Upgrades to the thickening facilities (design by Carollo), and
12. New dewatering building to replace aged dewatering facilities (design by AECOM).

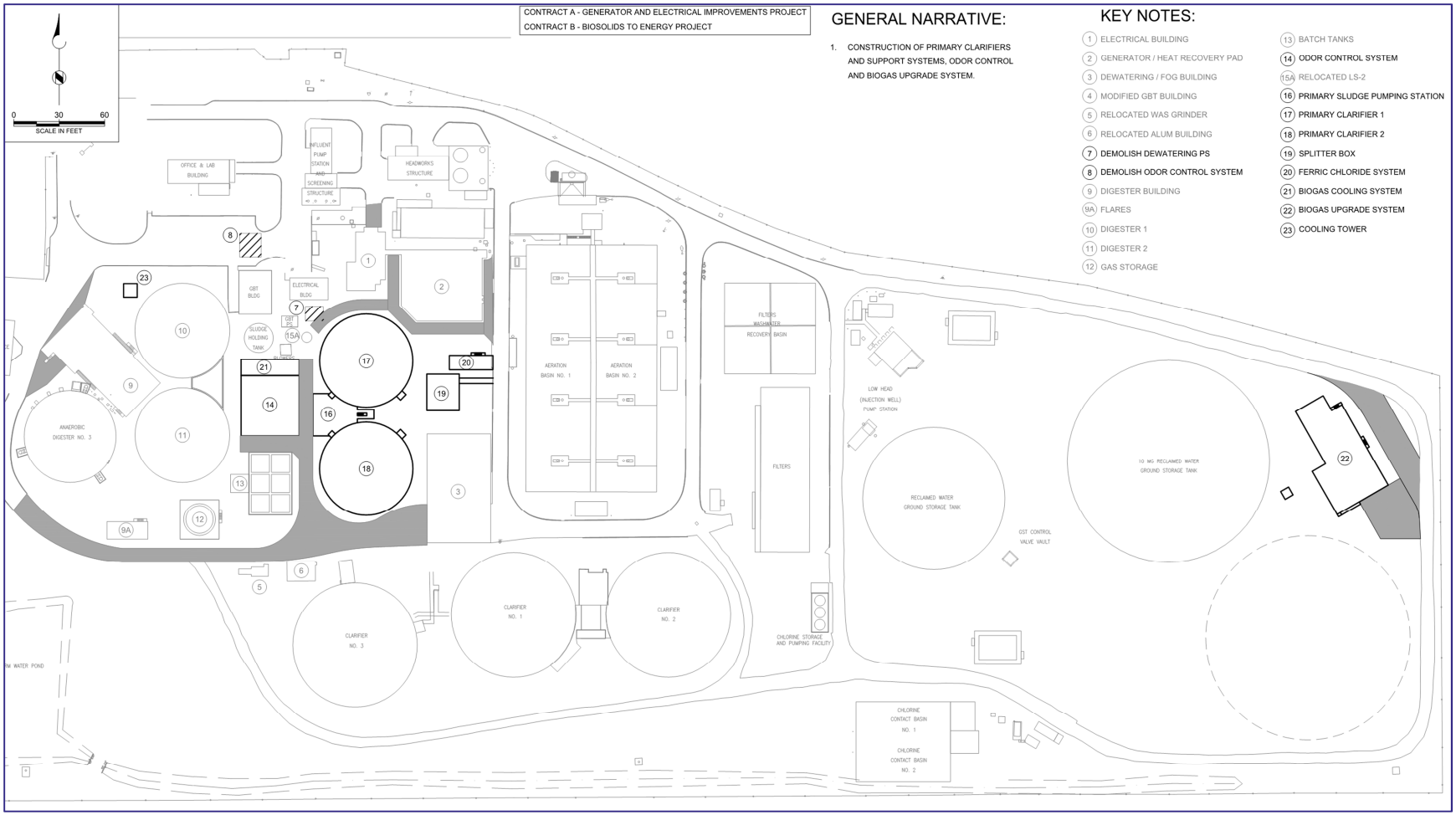

Figure 2-3. Proposed New Facilities

2.3 Permit Requirements

Operation of the SWWRF treatment and disposal facilities is subject to state and federal regulations stated in Florida Department of Environmental Protection (FDEP) Operating permit FLA 128848-016-DWI/MR.

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 (Maximum)
pH	SU	6.0 - 8.5
Total Nitrogen	mg/L	Max Report
Total Phosphorus	mg/L	Max Report
Total Residual Chlorine	mg/L	1 – Single Sample Minimum
Turbidity	NTU	Max Report

Reclaimed water which exceeds the reuse demand is disposed of through three existing Class I high-level disinfection injection wells located at the SWWRF site. The wells have a combined permitted disposal capacity of 27 MGD. 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

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Section 3

Influent Flow and Pollutant Loadings

The City of St. Petersburg has decided to consolidate two of their treatment facilities. As a result, the City is planning to discontinue the operation at the Albert Whitted WRF (AWWRF) and convert it into a master pump station which will pump its wastewater to the SWWRF. In addition, it is planned to build new sludge process units to handle the solids originated from both NEWRF and NWWRF at the SWWRF. A detailed analysis of flows and loads at the SWWRF and AWWRF were presented in the Capacity Assessment Report prepared by BC in March 2013. This section summarizes the combined anticipated flows and loadings to the facility during wet weather events when the AWWRF discontinues operation and the waste activated sludge (WAS) from NEWRF and NWWRF are divert to SWWRF.

This analysis includes review of recent historical hourly flow data (2012 & 2013) from the SWWRF and AWWRF. Special considerations were made to understand the future/projected wet weather flows when all the flow generated from the southeast and southwest service areas are consolidated, conveyed and treated at the SWWRF. The results of this analysis will be used as the basis for determining the future peak hydraulic and process capacities of the SWWRF after a number of upgrades recommended by Brown and Caldwell through the on-going Biosolids to Energy project are constructed and are in operation.

3.1 Historical Wet Weather Flows

Table 3-1 and 3-2 summarizes the historical influent flow data and resulting peaking factors for the SWWRF and AWWRF. Based on the analyses, the highest peaking factors were observed in wet weather events occurred in 2012 and 2013 (Table 3-2).

Table 3-1. Historical Flow Data (Combined Flow from SWWRF and AWWRF)

Year	2007	2008	2009	2010	2011	2012	2013
Annual Average (MGD)	14.25	15.22	15.54	16.10	16.52	16.43	17.80
Maximum Month (MGD)	18.22	18.63	21.67	21.35	22.96	23.70	27.89
Maximum Week (MGD)	21.72	20.24	23.93	25.84	27.00	32.91	37.20
Maximum Day (MGD)	27.37	22.42	29.12	30.98	32.93	47.78	46.82
Peak Hour (MGD)	n/a	n/a	n/a	n/a	n/a	54.25	60.72

Table 3-2. Flow Peaking Factors (Combined Flow from SWWRF and AWWRF)							
Year	2007	2008	2009	2010	2011	2012	2013
Maximum Month/Annual Average	1.28	1.22	1.39	1.33	1.39	1.44	1.57
Maximum Week/Annual Average	1.52	1.33	1.54	1.60	1.63	2.00	2.09
Maximum Day/Annual Average	1.92	1.47	1.87	1.92	1.99	2.91	2.63
Peak Hour/Annual Average	n/a	n/a	n/a	n/a	n/a	3.30	3.41

Historical hourly flow measurements for the AWWRF and the SWWRF were available for 2012 and 2013 and they are presented in Figure 3-1.

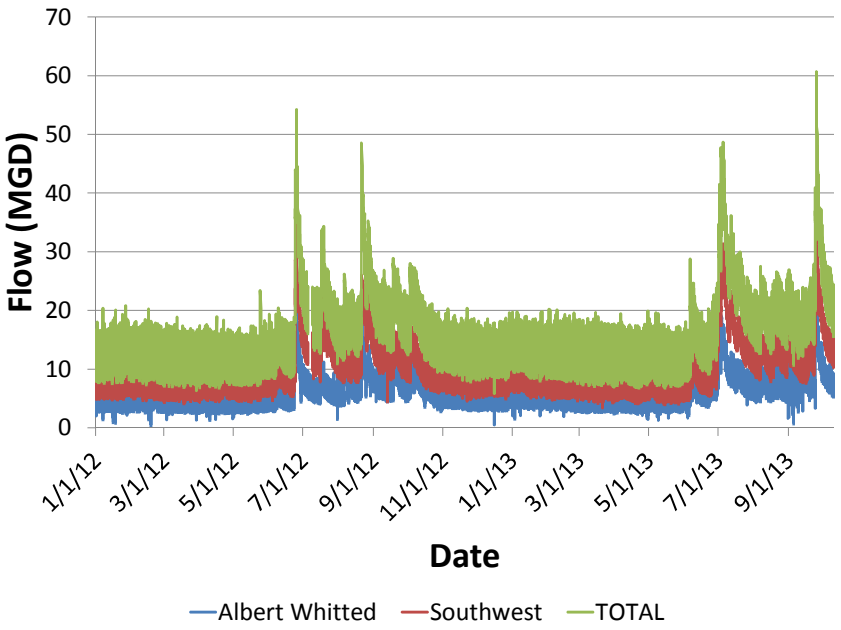

Figure 3-1. Historical Hourly Flow Measurements from January 2012 through November 2013

Based on the historical data, the highest combined annual AADF observed in 2013. The AADF has been 17.8 MGD with a peak hour flow of 60.72 MGD which occurred on September 25, 2013. Analysis of historical data indicated that, the hourly flows higher than 40 MGD happened less than 1% of the times and the duration of the main storm events generally did not extend more than a day. The maximum 24-hour rolling average flow was recorded as 50.5 MGD. Only 1% of the daily average flow was above 39 MGD. The analysis of historical data showed that the flows higher than 60 MGD occurred only for two consecutive hours in the 22-month worth of hourly data evaluated, which corresponds to 0.01% probability. The details of the wet weather flow analysis can be found in Appendix A.

Historical daily dry weather diurnal peaking factors have ranged from approximately 0.71 to 1.27; hence, under normal circumstances, the projected dry weather flow to the SWWRF at 20 MGD design flow conditions would range from 14 MGD to 25.5 MGD.

The historical flow data from the 2012-2013 period has shown a maximum wet weather flow contribution in excess of the dry weather flow of approximately 43.2 MGD. Assuming that the maximum dry weather flow and the storm flow occur at the same time, the total projected flow to the SWWRF would be 68.7 MGD (25.5 MGD + 43.2 MGD).

3.2 Historical Pollutant Loads

A detailed analysis of pollutant loads was conducted for Capacity Assessment Report dated in March, 2013. Figure 3-2 and Figure 3-3 show the historical relationship between influent flow and influent $CBOD_5$ and TSS concentrations. As these two figures depict, there is a correlation between high flow events and a drop in influent concentrations ($CBOD_5$ and TSS) at the SWWRF. Typically, as the flows increase during wet weather events, groundwater infiltration and storm flow contribute to the seasonal hydraulic load with no increase to the organic load to the plant. Based on historical data, it can be seen that the influent $CBOD_5$, and TSS concentrations drop below 80 mg/L during peak wet weather events due to the dilution effect of the dry weather influent with the groundwater infiltration and storm flow.

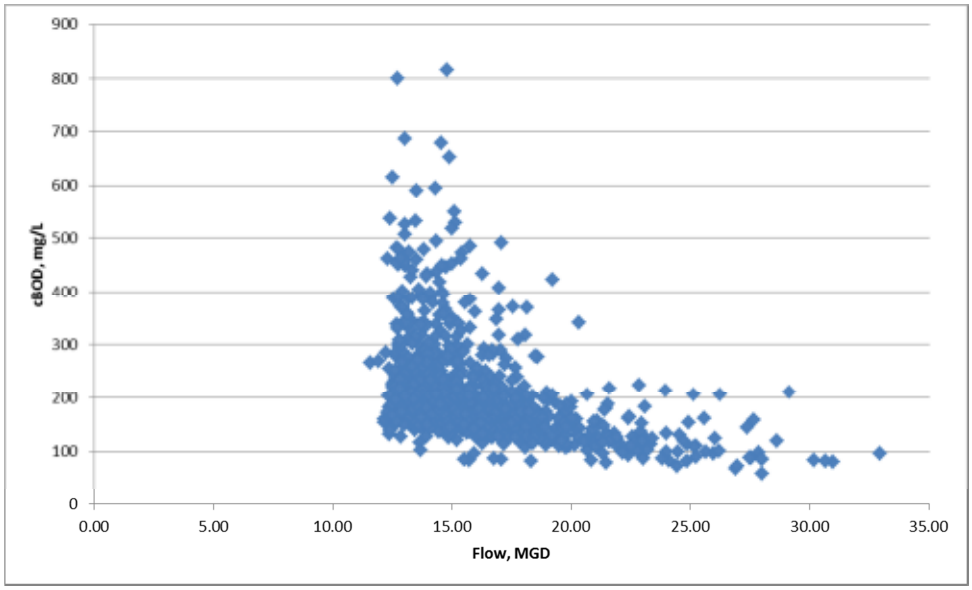

Figure 3-2. Historical Relationship between Influent Flow and $CBOD_5$

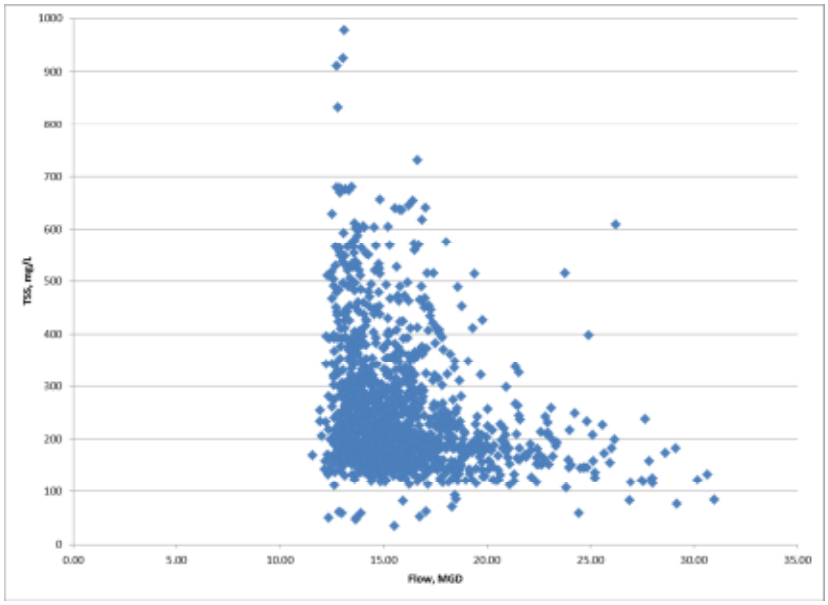

Figure 3-3. Historical relationship between influent flow and TSS

Therefore, it was assumed that the pollutant loadings would be similar to dry weather loadings during wet weather conditions.

3.3 Flow and Pollutant Loading Projections

For the purpose of wet weather analysis, emphasis was given, for the selection of the design conditions, to wet weather flows, as they have the strongest influence on wet weather plant capacity. Once peak conditions were selected, all values reported for these chosen period were used to define the pollutant loadings on the plant. Therefore, the defined peak conditions represent actual conditions experienced rather than inflated conditions based on selecting the highest load.

Table 3-3 presents a summary of the wet weather influent quality adopted for the purpose of this report based on the historical influent quality data and the capacity assessment results presented by BC in Capacity Assessment Report in March 2013.

Table 3-3. Wet Weather Basis of Design	
Parameter	Influent
Flow (MGD)	
Annual Average	20.0
Maximum Month	31.4
Maximum Day	52.6
Peak Hour	69.0
$CBOD_5$ 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

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Section 4

Results of Field Tests

The assessment of treatment process included extensive field tests not only to characterize the influent wastewater fractions but also to understand the performance of biological reactors at low DO operating conditions. These tests were found essential in order to tailor the process model to accurately simulate the expected conditions at the SWWRF. All field tests were conducted in collaboration with the City and the SWWRF staff. The following sections summarize the findings of the field tests.

4.1 Influent Characterization

Special sampling campaign conducted during August-November 2011 is used to determine the influent wastewater characteristics. Details of this special sampling campaign were provided in BC's Capacity Assessment Report in March 2013. Table 4-1 lists the influent wastewater fractions adopted for BioWin™ process model simulations.

Table 4-1. Influent Wastewater Fractions Adopted for BioWin Modeling

Name	Value
Fbs - Readily biodegradable (including Acetate)	0.143
Fac - Acetate	0.15
Fxsp - Non-colloidal slowly biodegradable	0.775
Fus - Unbiodegradable soluble	0.128
Fup - Unbiodegradable particulate	0.149
Fna - Ammonia	0.675
Fnox - Particulate organic nitrogen	0.50
Fnus - Soluble unbiodegradable TKN	0.02
FupN - N:COD ratio for unbiodegradable part. COD	0.035
Fpo4 - Phosphate	0.513
FupP - P:COD ratio for unbiodegradable part. COD	0.011

4.2 Stress Testing of Secondary Clarifiers

Evaluation and stress testing of secondary clarifiers was conducted in 2011 to determine the limiting conditions of existing secondary clarifiers and to collect adequate field data to calibrate a hydrodynamic model for the secondary clarifiers. Details of these tests were provided in BC's Capacity Assessment Report in March 2013.

4.3 Performance of Biological Reactors

A second special sampling campaign was conducted in August 2013 to document the performance of the biological reactors at the SWWRF at the low operating DO conditions. During the sampling event, one the aeration basin was taken off line and all of the flow was diverted to a single aeration basin in order to simulate the design conditions of the facility.

The results of the field testing indicated that due to the very low operating DO conditions in the biological reactors at the SWWRF, the influent total nitrogen was removed through simultaneous nitrification- denitrification process. At the operating SRT of 3 to 4 days, the biological reactors were monitored by measuring the influent and effluent nitrogen and phosphorus species over a period of 5 weeks. Figure 4-2 shows the effluent inorganic nitrogen species. During this period, the effluent NH_3-N, NO_2-N and NO_3-N averaged approximately 1.8, 0.41 and 0.23 mg N/L, respectively. The influent and effluent TP concentrations averaged 3.5 and 0.5 mg P/L, respectively, indicating good biological phosphorus removal even at very low DO conditions.

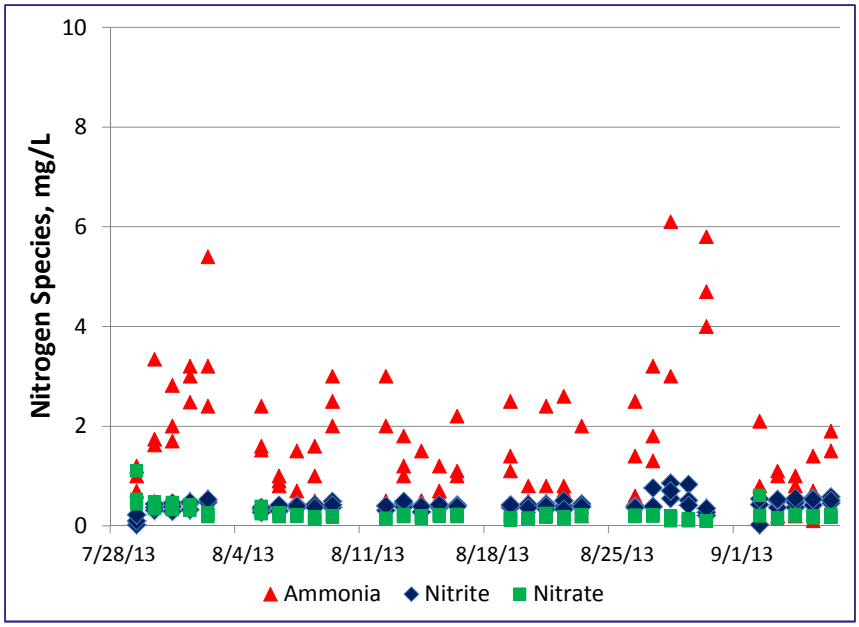

Figure 4-1. Effluent Inorganic Nitrogen Species during Stress Testing of the Reactors

During this sampling campaign, the operating DO in the reactors were low, with a gradient from approximately 0.4 mg/L at the front end of the aeration zone decreasing to approximately 0.1 mg/L at the back end. This operating mode achieved significant nitrogen removal with low NH_3-N, NO_2- and NO_3- in the effluent. The wastewater temperature ranged from 25 to 30°C. During the testing, approximately 2,500 SCFM were used for aeration at the SWWRF.

SVI values were also closely monitored throughout these tests since low DO conditions could impair the settling characteristics and increase the SVI values. The average SVI value was approximately 150 mL/g.

These results and operating conditions were adopted in evaluation of biological processes and tailoring of the simulation models.

4.4 Understanding the Nitrogen and Phosphorus Removal Pathways

Extensive field and batch test experiments were conducted by EnviroSim (a sub-consultant to BC) in March 2014 to understand the nitrogen and phosphorus removal pathways at low DO conditions. Appendix B provides the detailed results of this work. The objective of the field testing was to better understand the nitrogen and phosphorus removal pathways at low DO conditions. This was of particular interest to the team because of potential aeration energy savings with the current low DO operating mode. Furthermore, the team also focused on the biological phosphorus removal process, which is working extremely well despite the low DO in the aerated zones (Table 4-2).

EnviroSim conducted Specific Nitrification Rate Tests (SNR), Simultaneous Nitrification-Denitrification Rate Tests(SND), Denitrification Rate Tests(SDR), a test to detect the presence of Anammox bacteria, and Phosphorus Release-Uptake Tests. These tests were performed to complement the BioWin modeling effort undertaken by BC and to provide insights on the nitrogen and phosphorus removal pathways at the SWWRF.

Overall, the field testing results revealed that the SWWRF is performing exceptionally well using a simple process configuration. The key factors of this simple configuration include a large anaerobic zone (*i.e.* the unaerated zone is 25% of the total reactor volume) and accurate DO control at low DO levels (*i.e.* over a DO concentration range of 0.1 to 0.4 mg/L) to achieve a significant degree of nitrogen removal. Operating at such a low DO concentration substantially reduces the aeration costs at the plant. Although the treatment plant does not have to meet an effluent nitrogen and phosphorus limit; it is achieving remarkable nutrient removal with effluent TIN concentration of approximately 3.5 mg N/L and effluent orthophosphate concentration of around 0.1 mg P/L.

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Section 5

Hydraulic Profile

In order to conduct the hydraulic analysis of the SWWRF, the hydraulic model of the liquid treatment train, created previously using BC's PROFILE™ modeling software, a proprietary program developed by BC for calculating hydraulic and energy grade lines in water and wastewater treatment plants, was used. The hydraulic model included existing facility structures from the chlorine contact basin system to the headworks structure as well as improvements proposed through the Biosolids to Energy project. The model was constructed using available as-built drawings and other facility data provided by City.

The energy grade at any particular point is a measure of the total energy of the system at that point. The total energy is composed of the elevation head, the pressure head, and the velocity head. Subtracting the velocity head, (the kinetic energy of the system) from the energy grade, results in the hydraulic grade – a measure of the potential energy of the system at a specific point in the treatment plant. The energy grade is the fundamental reference for the profile calculations. All energy losses are applied to the energy grade rather than to the hydraulic grade. The velocity head is then subtracted from the energy grade to yield the hydraulic grade line through the treatment plant.

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 secondary clarifier # 3 which operates at a higher hydraulic level than secondary 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.

5.1 Biosolids to Energy Project Modifications

Section 6 discusses recommended process changes in order to handle future loadings to the plant. As a part of the Biosolids to Energy project, two primary clarifiers are proposed to be located hydraulically between the headworks and the aeration basin splitter box. The 30-inch flow meter that was shown to be a hydraulic restriction at the proposed peak flow rates will be taken out of the flow path and a new 48-inch flow meter will be located immediately upstream of the primary clarifier splitter box. This will provide sufficient hydraulic grade to support the new primary clarifiers within the treatment plant's hydraulic grade.

A new splitter box that equally proportions flow between the two proposed primary clarifiers using cutthroat flumes and weirs is also included in the design. An adjustable overflow/bypass is also proposed to divert some or all of the incoming flow from the headworks directly to the aeration basins thus bypassing the primary clarifiers. The cutthroat flumes have been sized to each pass future average and peak flow individually without entering submerged flow condition. Primary clarifier effluent will return to the opposite side of the primary clarifier splitter box where flow can be directed to three potential locations. The primary flow path is through another cutthroat flume feeding the head of the aeration basin splitter box. As flows increase, excess flow can be step-fed into to the second zone of the aeration basin using a modulating weir gate that adjusts according to the metered flow coming into the primary clarifier splitter box and the measured flow being sent to

the aeration basin splitter box. A third flow path will be for the highest flows which will enable flows to be sent to the third aeration zone for contact stabilization. A manual overflow weir gate will control the amount of flow sent to contact stabilization.

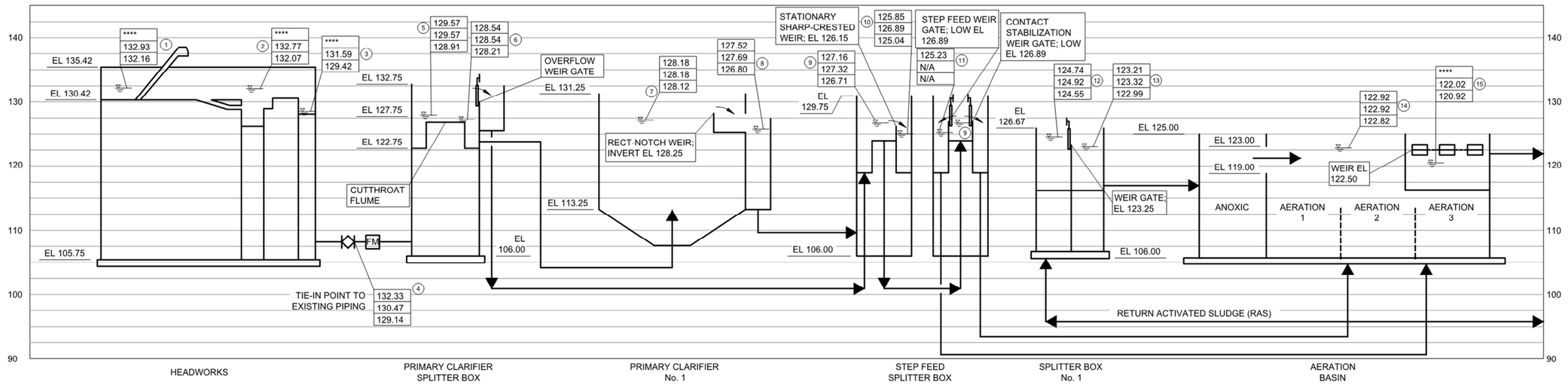
5.2 Hydraulic Capacity after Current Recommended Improvements

Various modeling scenarios were conducted to determine the hydraulic capacity of SWWRF after the modifications proposed in the Biosolids to Energy Preliminary Engineering Report. The hydraulic capacity of the unit process was defined as the flow at which the wastewater would over topple the top of wall for that tank. At 40 MGD flow, none of the unit processes were hydraulically challenged as shown in Figure 5-1; however, upon running the future peak wet weather flow of 69 MGD, the hydraulic elements failed to hold proper hydraulic gradients. The losses through the hydraulic elements were so great that the hydraulic grade line in all of the elements exceeded the top of the wall for the simulated process units. After various iterations and model runs, it was determined that the hydraulic capacity for the SWWRF was limited to 40.5 MGD. The limiting unit process at that flow was the filters.

The following hydraulic bottlenecks were identified under the 69 MGD future peak wet weather flow condition:

- Effluent Weir Channel at Chlorine Contact Basin.
- 54-inch pipe between Filters and Chlorine Contact Basin.
- 24-inch pipe from Filter #4 to Filter #3.
- 48-inch pipe between Secondary Clarifiers and Filters.
- 48-inch pipe between Aeration Basin to Secondary Clarifiers.

Additionally, the future peak wet weather flow of 69 MGD caused increased headloss through the Screens, Secondary Clarifiers Splitter Box, Secondary Clarifiers and Filtration process units.

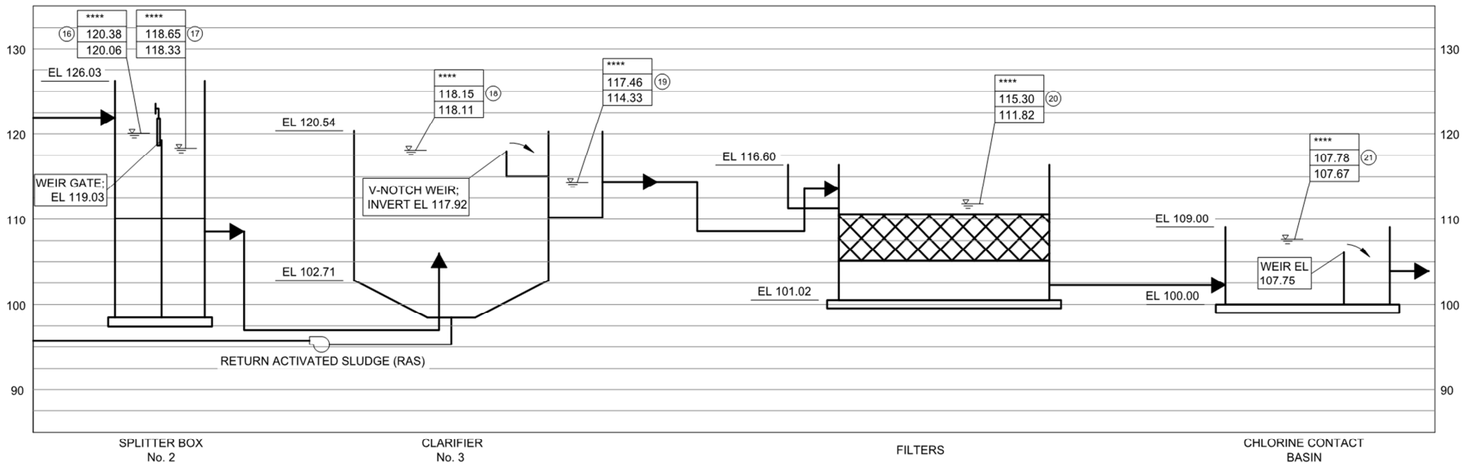


HYDRAULIC PROFILE NOTES:

1. ELEVATIONS OF EXISTING STRUCTURES BASED UPON SURVEY PERFORMED BY McKIM & CREED, OCTOBER 2013.
2. PROFILE WAS MODELED USING A RAS FLOWRATE OF 20 MGD.
3. PRIMARY CLARIFIERS HAVE A PEAK CAPACITY OF 40 MGD. INFLUENT FLOWS BEYOND THIS AMOUNT WILL BE BYPASSED AROUND THE PRIMARY CLARIFIERS.
4. AT FLOWS GREATER THAN 40 MGD, 29 MGD WILL BE SENT TO THE FRONT OF THE AERATION BASINS, WITH THE REMAINDER BEING SENT TO A COMBINATION OF THE STEP FEED AND CONTACT STABILIZATION FEED POINTS.
5. THE 69 MGD PROFILE IS BASED UPON A FIXED HYDRAULIC GRADE OF 122.92 IN THE AERATION BASINS AND IS MODELED UPSTREAM TO THE PROPOSED TIE-IN POINT OF THIS PROJECT. ADDITIONAL MODIFICATIONS ARE NEEDED TO PASS THESE FLOWS UPSTREAM AND DOWNSTREAM OF THE MODELED PORTION OF THE FACILITY.

LEGEND

69 MGD
40 MGD
20 MGD
SEE NOTES 3-5



LOCATION		MODELED CONDITION AND FLOW		
NUMBER	DESCRIPTION	20 MGD	40 MGD	69 MGD, SEE NOTE 5
1	HEADWORKS UPSTREAM OF SCREEN	132.16	132.93	****
2	HEADWORKS DOWNSTREAM OF SCREEN	132.07	132.77	****
3	HEADWORKS EFFLUENT BOX	129.42	131.59	****
4	DOWNSTREAM HEADWORKS TIE-IN	129.14	130.47	132.33
5	PRIMARY CLARIFIER SPLITTER BOX - UPSTREAM	128.91	129.57	129.57
6	PRIMARY CLARIFIER SPLITTER BOX - DOWNSTREAM	128.21	128.54	128.54
7	PRIMARY CLARIFIER NO. 1	128.12	128.18	128.18
8	PRIMARY CLARIFIER NO. 1 EFFLUENT BOX	126.80	127.69	127.52
9	AB/STEP FEED SPLITTER BOX - UPSTREAM	126.71	127.32	127.16
10	AB/STEP FEED SPLITTER BOX - DOWNSTREAM	125.04	126.89	125.85
11	STEP FEED DOWNSTREAM BOX	N/A	N/A	125.23
12	AERATION BASIN SPLITTER BOX - UPSTREAM	124.55	124.92	124.74
13	AERATION BASIN SPLITTER BOX - DOWNSTREAM	122.99	123.32	123.21
14	AERATION BASIN	122.82	122.92	122.92
15	AERATION BASIN EFFLUENT CHANNEL	120.92	122.02	****
16	CLARIFIER SPLITTER BOX - UPSTREAM	120.06	120.38	****
17	CLARIFIER SPLITTER BOX - DOWNSTREAM	118.33	118.65	****
18	CLARIFIER NO. 3	118.11	118.15	****
19	CLARIFIER NO. 3 EFFLUENT BOX	114.33	117.46	****
19	FILTERS	111.82	115.30	****
20	CCB	107.67	107.78	****

Figure 5-1. SWWRF Hydraulic Profile after Biosolids to Energy Project Improvements

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5.3 Proposed SWWRF Hydraulic Modifications

A planning level evaluation was conducted to determine the hydraulic modifications required at the SWWRF to pass the future peak wet weather flow of 69 MGD peak wet weather flow. The following improvement options are necessary to pass the future peak wet weather flow of 69 MGD.

5.3.1 Option 1 – Equalization of Flows with Existing Process Units

Based on historical data, the duration of the most intense portion of the recorded storm events have not exceeded 24 hours. In this option, the peak wet weather flow can be stored for a 24-hour period and then returned to the head of the facility at a controlled rate. Since the hydraulic capacity of the SWWRF is 40.5 MGD, flows higher than 40.5 MGD capacity will have to be equalized. For this option, an equalization tank with a volume of approximately 10 MG would be required to be able to reduce the wet weather flows from 69 MGD to 40.5 MGD entering the plant. This assumes that the influent flow at the SWWRF, after the storm event, returns to a rate less than 40.5 MGD and that there is not an extended duration of the associated storm event.

5.3.2 Option 2 – Equalization of Flows with Improved Secondary Clarifier Mechanism and Other Hydraulic Improvements

As part of this option, the following hydraulic elements were improved to treat a peak wet weather flow of 50 MGD. In order to accomplish this, the following improvements are necessary:

- Replace 48-inch DIP pipe from Secondary Clarifier Splitter Box to Secondary Clarifier #3 with a 54-inch pipe.
- Replace 36-inch DIP pipe from Secondary Clarifier #3 to Secondary Clarifier #2 with a 48-inch pipe.
- Replace 48-inch DIP pipe from Secondary Clarifier #2 to Secondary Clarifier #1 with a 54-inch pipe.
- Replace 48-inch DIP pipe from Secondary Clarifier #1 to Filters with a 54-inch pipe.
- Replace 24-inch DIP pipes from Filter #1-4 to Filter Effluent Pipe with 36-inch pipes.
- Replace 54-inch DIP pipe from Filters to Chlorine Contact Basin with a 60-inch pipe.
- Increase Effluent Weir Channel width from 2-ft to 3-ft at Chlorine Contact Basin.

This flow rate is still significantly less than the peak flow of 69 MGD. Hence, equalization of the flows beyond 50 MGD for a day is required. In this option, an equalization volume of approximately 4.5 MG is required. The same assumptions are made as in Option 1 with regards to the influent flow conditions after the storm event and the storm event duration.

5.3.3 Option 3 – No Equalization of Flows; Addition of New Hydraulic Elements and Unit Processes

This option modifies the existing configuration at the SWWRF by providing new hydraulic elements and unit processes for additional hydraulic capacity. The following hydraulic elements and unit processes were improved to treat a peak wet weather flow of 69 MGD:

- Replace 48-inch DIP pipe from Secondary Clarifier Splitter Box to Secondary Clarifier #3 with a 54-inch pipe.
- Replace 36-inch DIP pipe from Secondary Clarifier #3 to Secondary Clarifier #2 with a 48-inch pipe.

- Replace 48-inch DIP pipe from Secondary Clarifier #2 to Secondary Clarifier #1 with a 54-inch pipe.
- Replace 48-inch DIP pipe from Secondary Clarifier #1 to Filters with a 54-inch pipe.
- Replace 24-inch DIP pipes from Filter #1-4 to Filter Effluent Pipe with 36-inch pipes.
- Replace 54-inch DIP pipe from Filters to Chlorine Contact Basin with a 60-inch pipe.
- Increase Effluent Weir Channel width from 2-ft to 3-ft at Chlorine Contact Basin.
- Construct and Install a new Headworks Structure with a 3rd Screen.
- Construct and Expand Secondary Clarifier Splitter Box.
- Construct and Install a new Secondary Clarifier #4.
- Construct and Install a new Filter #5.
- Construct and Install a new Chlorine Contact Basin #3.

It should be noted that this evaluation is based upon static hydraulic conditions. An evaluation to establish the final equalization volumes should be performed to understand the dynamic inflow differences. As identified by City staff, other offsite improvements such as diverting wastewater flows from various lift stations to other water reclamation facilities as well as equalization of wastewater flows at AWWRF were not evaluated as part of the scope of this project and hence were not considered.

Section 6

Process Capacity Assessment

This section describes the wet weather treatment process capacity assessment for the SWWRF and it builds on information already presented to the City in previous reports. For the purpose of this assessment, the process model (BioWin™) and the secondary clarifier CFD model built and calibrated in 2011 were used. The BioWin™ model was modified to include process units listed in the draft Biosolids PDR and to simulate the future conditions for the SWWRF. In addition, important kinetic parameters in the BioWin™ model were modified to better represent the low DO conditions. These parameters were obtained through field and batch test experiments data collected during the field and laboratory testing conducted in March 2014.

6.1 Treatment Assessment

During the Southwest WRF Process Evaluation and Hydraulic Capacity Report dated March 2013, the secondary clarifiers were identified as the process bottleneck for the SWWRF with a capacity of 40 MGD. Therefore, recommendations were made to modify the configuration of the biological reactor from an AO process to a step-feed/biological contact process during wet weather events. Figure 6-1 shows a process schematic of the proposed modifications for the biological reactors.

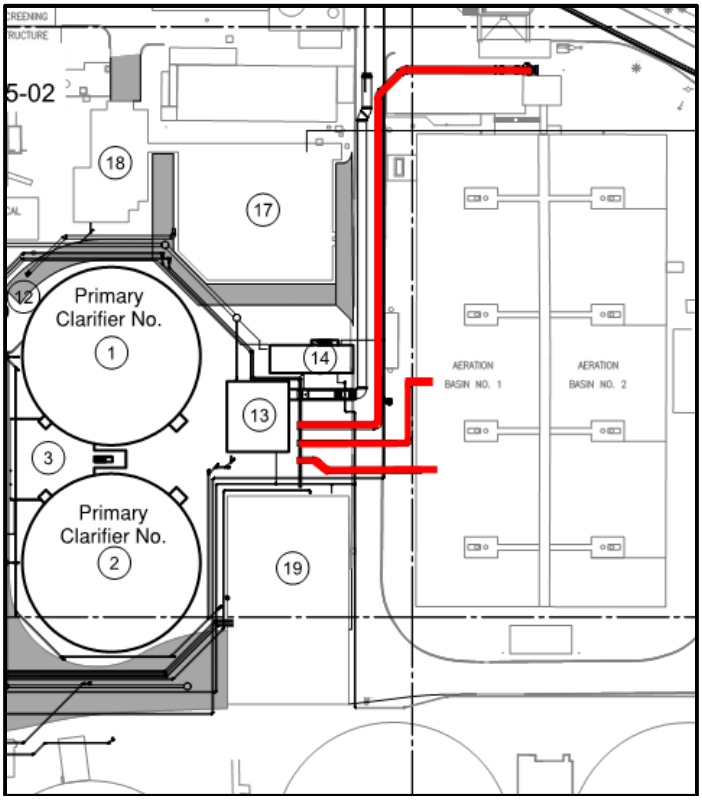

Figure 6-1. Step-Feed/Biological Contact Configuration for Wet Weather Control

This modified configuration was adopted to handle wet weather flow conditions since it provided a reduction to the loadings to the secondary clarifiers. Based on the modeling results, the following flow split is recommended during wet weather conditions:

- Influent flows of less than 30 MGD are fed to the front end of the biological reactor.
- During flow conditions between 30 to 40 MGD, a step feed pipe would be activated to divert the excess flow to the middle of the biological reactor.
- During flows in excess of 40 MGD, a second step feed line is activated to divert flow to the back end of the reactor.

Table 6-1 presents a summary of the predicted mixed liquor concentration profile through the biological reactor and compares this with a configuration without the recommended step-feed/biological contact treatment configuration. For this, total and aerobic SRT of 6 and 4 days were assumed based on information presented the Capacity Report dated on March 2013.

Table 6-1. Estimated Mixed Liquor Concentration Profile in Reactors during Wet Weather Conditions

Reactor Zone	No Step Feed	Recommended Step Feed
Anaerobic Zone	4,000 mg/L	4,000 mg/L
Aerobic Zone No:1		4,000 mg/L
Aerobic Zone No:2		3,000 mg/L
Aerobic Zone No:3		2,200 mg/L

Table 6-2 presents a summary of the aeration demands estimated during design conditions. For the purpose of this analysis, the low DO operation was kept since it provides effluent quality benefits plus significant energy savings for the City. Appendix C shows detail calculations for the aeration system at design conditions. Overall, the aeration system has adequate capacity to handle the SWWRF design conditions if the plant can maintain it current low DO operation and the removal of nitrogen through nitritation-denitritation as it was observed during the field and batch testing. Hence, no additional capacity is required. Due to the narrow DO range currently being operated at the SWWRF to maintain the low DO nitrogen removal conditions, BC recommends implementing an online automatic NH_3-N based aeration control system to provide process reliability and flexibility during future operation at design conditions.

Table 6-2. Estimated Aeration Demands

Condition	Total Demand, SCFM
Annual Average	7,000
Peak Hour	13,000
Minimum Day	3,500

6.1.1 Secondary Clarifiers

Table 6-3 summarizes the characteristics of existing secondary clarifiers. The secondary clarification system has a maximum RAS capacity of 18 MGD.

Table 6-3. 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 Treatment Process and Hydraulic Evaluation Report dated March 2013, identified the secondary clarifiers as the main process bottleneck at the SWWRF with an existing capacity of approximately 34 MGD peak wet weather flow. The CFD model identified the internal mechanisms and sludge withdrawal mechanisms as the main components limiting the clarification capacity at the facility. It should be noted that the CFD modeling results were based on a design sludge volume index (SVI) of approximately 120 mL/g which is considerably lower than the historical SVI values at the SWWRF. BC recommended the adoption of positive means to reduce the SVI at the SWWRF, especially during wet weather conditions. The report identified recommendations to the internal mechanisms of the existing secondary clarifiers to increase their capacity and efficiency, including increasing the dimensions of the center well to approximately 40 feet in diameter and 7 feet deep and to replace the existing sludge withdraw mechanisms by TowBro-type suction mechanisms. In addition, the report included recommendations for the addition of peripheral baffles to improve effluent quality.

A series of scenarios were simulated with the CFD model built during the capacity assessment in 2011. The model was modified and adopted to simulate the wet weather conditions described herein. Table 6-4 presents a summary of the capacity assessment results for the secondary clarifiers at the SWWRF.

Table 6-4. Secondary Clarifier Capacity Assessment Results			
Parameter	Existing Condition	Optimized Internals[4]	Optimized Internal plus a New Secondary Clarifier[5]
Capacity (MGD)	40	50	69
No. of Units	3	3	4
RAS per Unit (MGD)	6.0	6.0	6.0
SVI (mL/g)1	120	120	120
Effluent TSS (mg/L)2	35.0	30.0	25.0
Sludge Blanket Depth (% of total depth)3	60	48	35

[1] Design SVI of 120 mL/g. It is required to adopt settling control strategies to ensure low SVI during wet weather flow conditions.

[2] CFD model predictions for maximum day and peak hour effluent suspended solids concentrations.

[3] CFD model predictions for maximum sludge blanket depth.

[4] Optimized internals include replacement of existing center well for a new flocculator center well (40-ft diameter, 7-ft deep), replacement of organ pipe collectors for Tow-Bro mechanisms, additional of peripheral baffle.

[5] New secondary clarifier (135 ft diameter and 16 ft side water depth) with flocculator center well and Tow-Bro collectors and peripheral baffles.

6.1.2 Filtration and Disinfection

Filtration is provided by four multi-media filters with a total surface area of 5,624 square feet. Based on BC's experience and information published in the Manual of Practice (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. When all units are online, capacity of the filtration system is approximately 30 MGD AADF and 65 MGD PHF which is less than the defined wet weather peak flow of 69 MGD. However, if the storm event does not persist and only continues for an hour, the unit might recover from this single event. If one filter is out of service, the capacity of the filtration system is approximately 24 MGD AADF and 48 MGD PHF. Addition of another filter unit is recommended.

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. Under these conditions, the capacity of the existing disinfection process at the SWWRF is approximately 23 MGD AADF and 46 MGD PHF. At 69 MGD peak wet weather flow conditions, the detention time in the CCT would be approximately 20 minutes, therefore the chlorine feed system should be operated at 6 mg/L residual in the contact basin effluent.

6.2 Options to Increase Wet Weather Capacity

This section provides a brief description of possible options at the SWWRF to increase the capacity of the secondary clarifiers and to handle wet weather flow conditions. All options considered aimed at reducing the loadings to the existing secondary clarifiers, which is the capacity-limiting process unit at the SWWRF.

The following presents a summary list of the treatment options considered during this project:

- Wet weather flow equalization.
- Addition of a new secondary clarifier.

6.2.1 Option 1 – Equalization of Flows with Existing Secondary Clarifiers

Based on historical data, the duration of the main storm events have been less than a day. If the peak wet weather flow can be stored for a day, then it can be pumped into the facility at the same equalized rate. Since the capacity of the secondary clarifiers is 40 MGD, flows higher than 40 MGD capacity could be equalized. For this option, approximately 10 MG storage volume would be required to be able to treat 69 MGD peak hour flow with three existing secondary clarifiers.

6.2.2 Option 2 – Equalization of Flows with Improved Secondary Clarifier Mechanism

As part of this assessment, physical modifications to the internal mechanisms of the secondary clarifiers were evaluated in order to increase capacity. Additional modeling indicated that the capacity of existing secondary clarifiers could be increased up to 50 MGD at SVI of 120 mL/g. This flow rate is still significantly less than the peak flow of 69 MGD. Hence, equalization of the flows beyond 50 MGD for a day would be another option. In this option, it is required to add an equalization volume of approximately 4.5 MG.

6.2.3 Option 3 - Addition of a New Secondary Clarifier

This option modifies the existing configuration at the SWWRF by providing additional secondary clarification capacity. Based on modeling results, addition of one 135-foot diameter clarifier is required to increase the capacity of the SWWRF to 69 MGD. This option assumes that the internal mechanisms of the secondary clarifiers would be improved and maximum SVI would be controlled at 120 mL/g.

Summary List of Planning Level Improvements

Table 6-5 provides a summary of options to handle wet weather flows at the SWWRF.

Table 6-5. Summary of Options to Handle Wet Weather Conditions at the SWWRF

Parameter	Option 1	Option 2	Option 3
Projected Wet Weather Flow (MGD)	69	69	69
Maximum Treatment Capacity (MGD)	40	50	69
Equalization Requirements	Yes	Yes	No
Equalization Volume Required (MG)	10	4.5	Not Required
Hydraulic Modification Requirements	No	Yes	Yes
List of Hydraulic Improvements	--	• Replace 48-inch pipe from Secondary Clarifier Splitter Box to Clarifier #3 with a 54-inch pipe • Replace 36-inch pipe from Clarifier #3 to Clarifier #2 with a 48-inch pipe • Replace 48-inch pipe from Clarifier #2 to Clarifier #1 with a 54-inch pipe • Replace 48-inch pipe from Clarifier #1 to Filters with a 54-inch pipe • Replace 24-inch pipes from Filters #1-4 to Filter Effluent Pipe with 36-inch pipes • Replace 54-inch pipe from Filters to Chlorine Contact Basin with a 60-inch pipe • Increase Effluent Weir Channel width from 2-ft to 3-ft at Chlorine Contact Basin	• All improvements for Option 2, and • Construct and Install a new Headworks Structure with a 3rd Screen • Construct and Expand Secondary Clarifier Splitter Box • Add a new Secondary Clarifier #4 • Add a new Filter #5 • Add a new Chlorine Contact Basin #3
Biological Reactor Configuration	Step Feed/Biological Contact	Step Feed/Biological Contact	Step Feed/Biological Contact
Future Aeration Capacity Needs	No	No	No

Table 6-5. Summary of Options to Handle Wet Weather Conditions at the SWWRF			
Parameter	Option 1	Option 2	Option 3
Secondary Clarifiers			
Number of Units	3	3	4
Center Well Dimensions	16-ft dia., 7-ft deep	40-ft dia., 7-ft deep	40-ft dia., 7-ft deep
Peripheral Baffle	No	Recommended	Recommended
Sludge Withdrawal Mechanism	Organ Pipe	Tow-Bro	Tow-Bro
Total RAS Capacity (MGD)	18	18	24
SVI Control Requirements	Yes	Yes	Yes

Section 7

Limitations

This document was prepared solely for the City of St. Petersburg in accordance with professional standards at the time the services were performed and in accordance with the contract between City of St. Petersburg and Brown and Caldwell dated November 20, 2012. This document is governed by the specific scope of work authorized by City of St. Petersburg; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. We have relied on information or instructions provided by City of St. Petersburg and other parties and, unless otherwise expressly indicated, have made no independent investigation as to the validity, completeness, or accuracy of such information.

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Appendix A: Wet Weather Flow Analysis

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DRAFT Technical Memorandum

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Prepared for: City of St. Petersburg

Project Title: Wet Weather Capacity Assessment

Project No.: 145682

DRAFT Technical Memorandum

Subject: Wet Weather Flow Projections

Date: April, 18, 2014

To: Steve Marshall, Project Manager

From: Jose Jimenez, Ph.D., P.E.

Copy to: Todd Bosso, P.E.

Prepared by: Derya Dursun, Ph.D., P.E.
Process Engineer

Reviewed by: Jose Jimenez, Ph.D., P.E.
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Section 1: Introduction

This Technical Memorandum summarizes the results of the analysis performed to estimate the wet weather flow projections for the City of St. Petersburg's Southwest Water Reclamation Facility (SWWRF). Preliminary Design Report (PDR) prepared for the City of St. Petersburg in September 2013 provides the details of the flow projection efforts that were performed for Biosolids to Energy Project. However, in these analyses, the peak flow conditions were capped at 40 MGD due to hydraulic limitations of the facility. Following to this work, very high flow conditions were recorded at the Southwest (SWWRF) and Albert Whitted (AWWRF) Water Reclamation Facilities. Hence, the City would like to update the capacity assessment of the SWWRF to identify potential modifications or improvements within the SWWRF necessary to meet future peak flows created by wet weather conditions.

This analysis includes review of most recent historical flow data (2012 & 2013) from the Southwest (SWWRF) and Albert Whitted (AWWRF) Water Reclamation Facilities. Special considerations were made to understand the future/projected wet weather flows when all the flow generated from the southeast and southwest service areas are consolidated, conveyed and treated at the SWWRF. The results of this analysis will be used as the basis for determining the future peak hydraulic and process capacities of the SWWRF after a number of upgrades recommended by Brown and Caldwell through the on-going Biosolids to Energy project are constructed and in operation.

Section 2: Historical Flow Data Overview

Historical wastewater flow data from 2007 through 2012 for the SWWRF and AWWRF were analyzed in the content of Biosolids to Energy Project. Recent historical flow data from January 2012 through December 2013 from the AWWRF and SWWRF were combined with previous data set and analyzed to determine the current and future peak influent flows to the SWWRF. Based on the analyses, the highest peaking factors were observed in 2012 and 2013. Therefore, the preceding sections discuss the conditions observed in 2012 and 2013 and the results are summarized below.

2.1 Historical Hourly Flow Measurements

Figure 1 presents historical hourly flow measurements for the AWWRF and the SWWRF as well as the calculated combined flow values from both facilities. Results are summarized in Table 1. Based on the data used for this analysis, the combined annual average daily flow (AADF) for 2012 and 2013 is approximately 17 million gallons per day (MGD) with a peak hour flow of 60.72 MGD which occurred September 25, 2013. The historical maximum monthly average flow (MMF) is approximately 28 MGD which occurred in July 2013. The 2012-2013 data shows that during annual average conditions approximately 60 percent of the combined flow is produced in the southwest service area whereas 70-75 percent of the combined peak hour flow is produced in the southwest service area.

An analysis of the 2012-2013 flow data, as provided by the City, was performed to understand the dry-weather flow and wet weather flow components of the historical flow data. The results are summarized in Figures 2 and 3. Figure 2 presents the combined average dry-weather diurnal flows for weekday and weekend days for the 2012-2013 period. The average dry-weather diurnal flow patterns for the combined flow are considered typical. This data shows that higher flows are often observed during weekend days than weekdays. Based on this data, the combined dry-weather flow peaking factors are approximately 1.22 and 1.27 for weekdays and weekend days, respectively.

Figure 3 presents the daily wet-weather flows only (*total flow minus dry-weather flow*) during the 2012-2013 period. This data shows that the wet weather period in the combined service area extends through the months of June to September, which is typical for Florida. Figure 3 depicts that the maximum combined wet weather flows generated were approximately 43.2 and 41.5 MGD for 2012 and 2013, respectively.

Table 2 presents the flow peaking factors based on the historical data presented in Figure 1. Data from 2013 offers higher combined flow peaking factors than the 2012 data and therefore, they were assumed for this analysis. The maximum peak hour flow peaking factor for this period is 3.41.

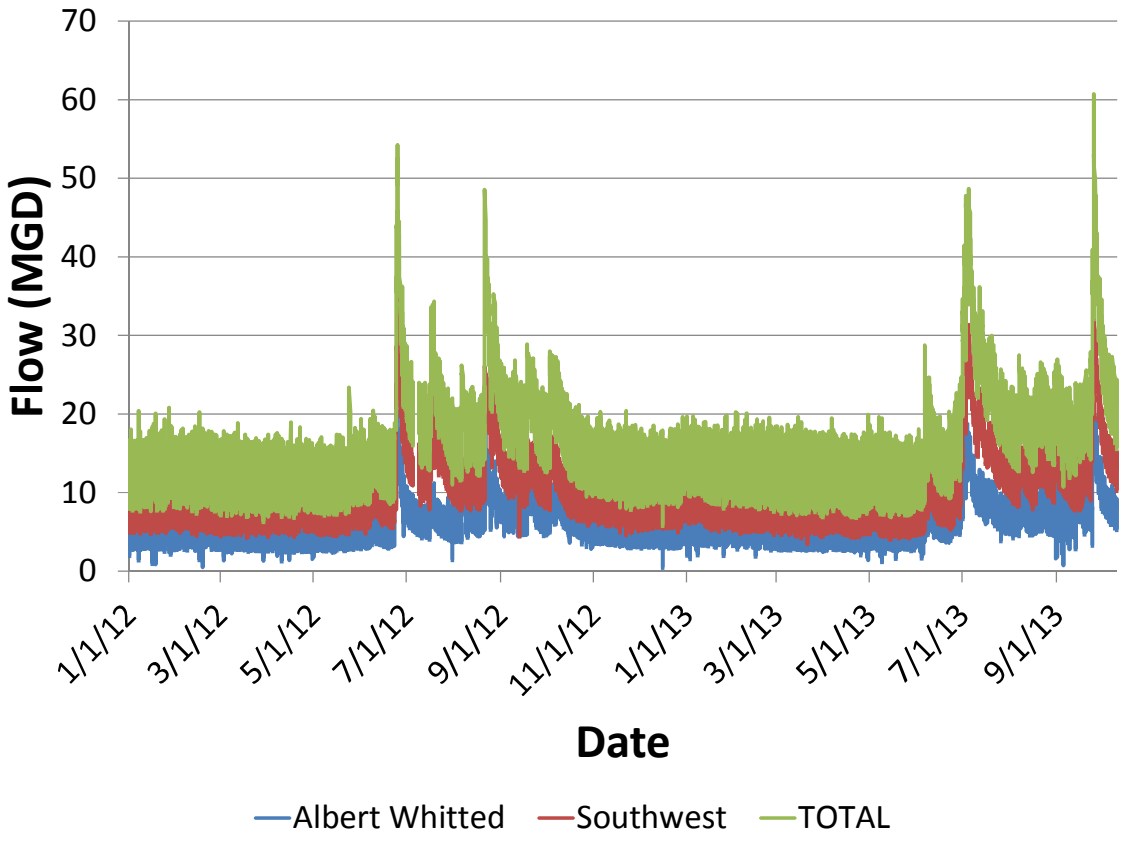

Figure 1. Historical hourly flow measurements from January 2012 through November 2013

Table 1. Analysis of Historical Flow Data

Condition	AWWRF (MGD)		SWWRF (MGD)		Combined Flows (MGD)	
	2012	2013	2012	2013	2012	2013
Annual Average	6.30	6.78	10.26	11.02	16.43	17.80
Maximum Month	8.76	10.13	16.18	17.83	23.70	27.89
Maximum Day	17.98	18.80	29.80	28.02	47.78	46.82
Peak Hour	22.49	25.84	35.10	34.88	54.25	60.72

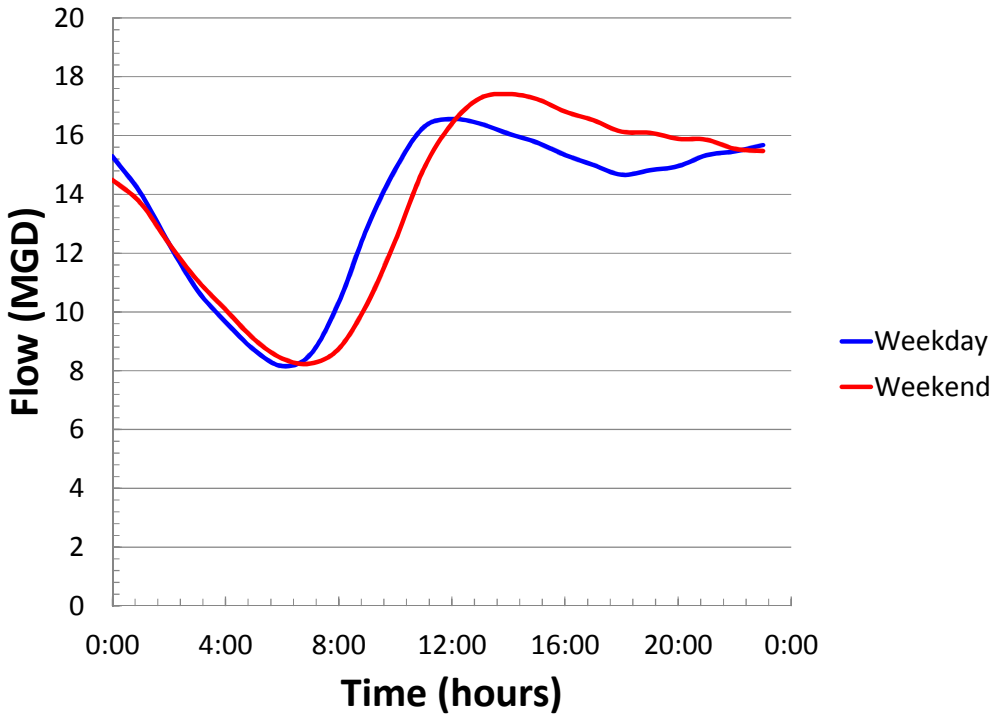

Figure 2. Historical average combined dry-weather diurnal flows for weekdays and weekend days

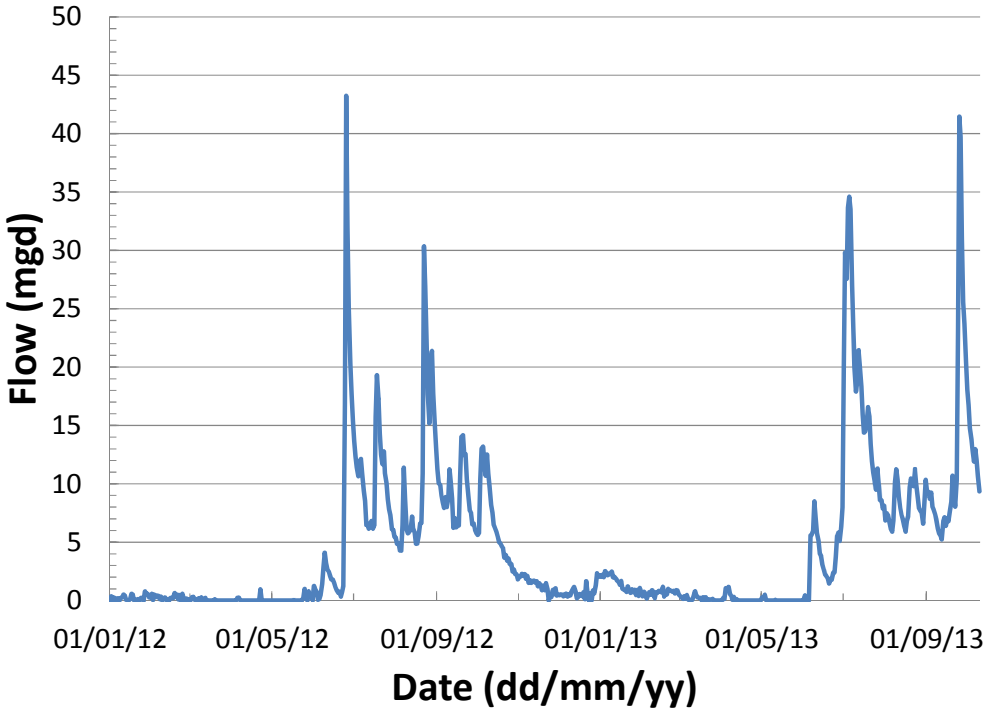

Figure 3. Historical combined wet weather flow from January 2012 through November 2013

Table 2. Summary of Flow Peaking Factors						
Condition	AWWRF		SWWRF		Combined Flows	
	2012	2013	2012	2013	2012	2013
MMF/AADF	1.39	1.49	1.58	1.62	1.44	1.57
MDF/AADF	2.85	2.77	2.91	2.54	2.91	2.63
PHF/AADF	3.57	3.81	3.42	3.16	3.30	3.41

Section 3: Flow Projections

Historical hourly flow data from 2012 through 2013 was reviewed to establish the projected combined flows to the SWWRF. The same approach used in the PDR was adopted for wastewater flow projections. BC recommends 0.4% increase in average yearly wastewater flow to be used for the design year (2035). Table 3 summarizes wastewater flow projections.

Table 3. WRF Wastewater Flow Projections			
Condition	Wastewater Flows MGD		
	SWWRF	AWWRF	Combined
2012	10.26	6.30	16.43
2013	11.23	6.78	17.80
2015	11.32	6.83	17.94
2020	11.55	6.97	18.30
2025	11.78	7.11	18.67
2030	12.02	7.26	19.05
2035	12.26	7.40	19.43
Recommended Design Flow			20.00

The current combined AADF generated in the southeast and southwest service areas averages 17 MGD and the SWWRF is currently permitted to treat up to 20 MGD as AADF. The projected wastewater flow for 2035 is 19.43 MGD. It is recommended to use design flow rate of 20 MGD. Historical daily dry weather diurnal peaking factors have ranged from approximately 0.71 to 1.27; hence, under normal circumstances, the projected dry weather flow to the SWWRF would range from 14 MGD to 25.5 MGD.

The historical flow data from 2012-2013 period has shown a maximum wet weather flow contribution in excess of the dry weather flow of approximately 43.2 MGD. Similar wet weather contribution was observed in 2012 and 2013; hence, it was used for this analysis. Assuming that the maximum dry weather flow and the storm flow occur at the same time, the total projected flow to the SWWRF would be 68.7 MGD (25.5 MGD + 43.2 MGD). This analysis assumes that the collection systems in the southeast and southwest service areas are capable of conveying such to the SWWRF.

As a check, another second method was also used to estimate the projected flows to the SWWRF. This method was based simply on using direct flow peaking factors estimated from the historical data. Table 4 presents a summary of the selected peaking factors based on the historical data. Table 4 also presents the

projected flow values for the SWWRF. Based on this approach, the maximum combined peak hour flow to the facility would be 68.2 MGD which is consistent with the results obtained using the first approach (which yielded a peak of 68.7 mgd).

Table 4. Flow Ratios and Projected Flow Rates

Description	Ratio	Flow Rate (MGD)
AADF, MGD	n/a	20.0
MMF/AADF	1.57	31.4
PDF/AADF	2.63	52.6
PHF/AADF	3.41	68.2
Minimum Daily Flow: AADF	0.71	14.2

Based on the above analysis, BC recommends that the City use 69 MGD as future peak hourly flow to determine the hydraulic and process capacities at the SWWRF.

Appendix B: EnviroSim St. Petersburg SWWRF Process Investigation

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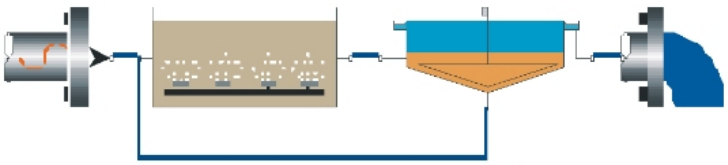
ST. PETERSBURG SOUTHWEST WRF PROCESS INVESTIGATION

TECHNICAL MEMORANDUM

May 2014



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Date: May 14, 2014

Subject: St. Petersburg Southwest Water Reclamation Facility Process Investigation

1.0 INTRODUCTION

This report describes the testing and analysis that was carried out at the City of St. Petersburg Southwest Water Reclamation Facility (WRF) to assess the performance of nitrification, denitrification and biological phosphorus removal. Of particular interest is whether shortcut nitritation-denitritation (nitrite-shunt) is contributing to nitrogen removal in the plant. Furthermore, focus is placed on the biological phosphorus removal process, which is working well despite the low dissolved oxygen in the aerated zones.

The traditional biological nitrogen removal process employed in wastewater treatment plants (WWTPs) involves the autotrophic oxidation of ammonium (NH_4^+) to nitrate (NO_3^-) (nitrification) and subsequent heterotrophic reduction to nitrogen gas (N_2) with an organic carbon source (denitrification). Nitrite (NO_2^-) is an intermediate in both nitrification and denitrification. Therefore, if nitrification is stopped at NO_2^- (nitritation - NH_4^+ conversion to NO_2^-), then denitritation from NO_2^- to N_2 can be achieved. The coupled process is known as nitritation-denitritation or nitrite-shunt. Nitrite-shunt may yield a reduction in aeration of up to 25% (if an external carbon source was required for denitrification) and a reduction in COD requirements of up to 40% when compared to conventional nitrification-denitrification. Implementing nitrite-shunt requires repression of the nitrite oxidizing bacteria (NOB; those that oxidize NO_2^- to NO_3^-) while retaining activity of the NH_3 oxidizing bacteria (AOB; those that oxidize NH_4^+ to NO_2^-).

The means for operating in the nitrite-shunt mode often is presented as a process control approach based on manipulating kinetics. It is suggested that the activated sludge reactors are operated at a relatively low SRT and under conditions such that NOBs are washed out, but not the AOBs. The SRT is targeted such that [μ = growth rate; b = decay rate; f_A = aerated mass fraction]:

$$\frac{1}{f_A \cdot \mu_{AOB} - b_{AOB}} < SRT < \frac{1}{f_A \cdot \mu_{NOB} - b_{NOB}}$$

Many factors impact the kinetics, often in an interactive manner; these include DO concentration, temperature, free ammonia (FA) and free nitrous acid (FNA) concentrations, pH, and so on. In sidestream applications (e.g. centrate treatment with high ammonia) several of these factors can be harnessed to achieve washout of NOBs with sustained growth of AOBs. That satisfies the *nitritation* part of nitrite-shunt. [In suspended growth sidestream systems carbon addition is required for denitritation and this usually happens in a separate reactor or during an unaerated phase of operation]. In mainstream treatment not all of the factors are available for manipulation, and process control to achieve nitrite-shunt has both a kinetic and a stoichiometric component. For example, operating all (or perhaps part) of the aerated stage at low DO conditions may cause ordinary heterotrophic organisms (OHOs) to utilize NO_2^- generated by AOBs as an electron acceptor for growth on influent substrate; *i.e.* denitritation. In this situation the stoichiometric yield of NOBs is reduced because not all of the NO_2^- is converted to NO_3^-. This operating

mode promotes simultaneous nitritation-denitritation, which works to repress NOB generation and may lead to complete NOB washout. An objective in this study was to assess the degree of NOB washout at the Southwest WRF.

As an aside, usually the quoted advantages of achieving NOB washout and nitrite-shunt are two-fold: (a) reduced aeration demand (because NO_2^- is not oxidized to NO_3^-), and (b) reduced carbon requirements (because we remove the step of OHOs reducing NO_3^- to NO_2^-). In sidestream systems both those benefits should be attained because there is external carbon addition. However, in mainstream treatment it is a moot point whether nitrite-shunt is a substantial benefit; this depends largely on influent COD:N ratio. Consider the case where the COD:N is high, and there is sufficient COD available for converting either NO_3^- or NO_2^- to nitrogen gas. The oxygen requirement for NOBs to convert NO_2^- to NO_3^- is 1.14 mgO/mgN. However, the oxygen credit for denitrifying NO_3^- to NO_2^- is 1.14 mgO/mgN which exactly offsets the demand. The dual benefits of nitrite-shunt only become important if there is a COD limitation. This is less of a factor at the Southwest WRF currently because raw influent wastewater is treated rather than primary effluent; COD:N decreases across primary settlers. Nevertheless, operating at low DO typically implies reduced aeration energy costs.

The Southwest Water Reclamation Facility is owned and operated by the City of St. Petersburg, Florida. The facility is permitted to treat an annual average daily wastewater flow of 75,000 m^3/day from the southwest section of the City. Current flow is approximately 30,000 m^3/day (8 mgd). Final effluent is distributed in the City's reclaimed water system or disposed through deep injection wells located on the plant property. Figure 1 presents a schematic flow diagram for the facility that includes screening, grit removal, activated sludge process, secondary clarification, deep-bed filtration, and disinfection. Residuals treatment includes gravity belt thickening, anaerobic digestion, and dewatering by belt filter presses.

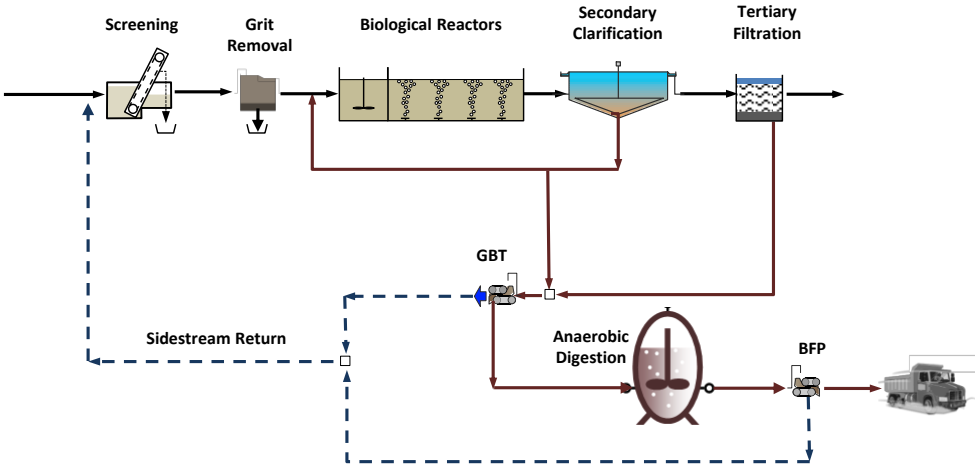

FIGURE 1: PROCESS FLOW DIAGRAM FOR THE SOUTHWEST WRF.

The biological process at the Southwest WRF is an activated sludge system configured as an A/O process. The system currently operates at total and aerobic solids retention times (SRT) of approximately 5 days and 3.5 days, respectively. No supplemental carbon or chemicals are added for nutrient removal or pH control. The biological process is operated with a simple control strategy whereby the average dissolved oxygen (DO) in the reactors is low, with a gradient from approximately 0.4 mg/L at the front end of the aeration zone decreasing to approximately 0.1 mg/L at the back end. This operating mode achieves significant nitrogen removal with low NH_4^+, NO_2^- and NO_3^- in the effluent. These effluent parameters are monitored regularly at the treatment plant and used as indicators to change SRT and DO setpoints. Table 1 presents a summary of the control strategy implemented at the Southwest WRF. The wastewater temperature ranges from 25 to 30°C.

TABLE 1: SUMMARY OF PROCESS CONTROL STRATEGY AT THE SOUTHWEST PLANT

Control Parameter	Condition	Action
NH_4^+ Control	NH_4^+ lower than 1 mg N/L	Reduce SRT to limit NH_4^+ removal and keep the average DO to a minimum value of 0.1 mg/L.
	NH_4^+ higher than 3 mg N/L	Increase SRT to improve NH_4^+ removal and keep DO to a minimum value of 0.1 mg/L. If SRT approaches 5 days, increase the DO to a maximum value of 0.3 mg/L until NH_4^+ is reduced
NO_3^- Control	NO_3^- higher than 1 mg N/L	Decrease the DO to a minimum value of 0.1 mg/L and monitor NO_2^- accumulation (profile) in the aeration basin
	NO_3^- lower than 1 mgN/L	No action required
NO_2^- Control	Monitor effluent NO_2^- as surrogate measurement of shunt performance	

Brown and Caldwell carried out a detailed sampling campaign at the Southwest WRF during 2013. The average measured influent COD-to-nitrogen ratio (COD:N ratio) was 7:1 and the average measured influent COD-to-NH_4^+ ratio was 10:1. Figure 2 shows the effluent inorganic nitrogen concentrations during the first phase of the sampling campaign. The effluent NH_4^+, NO_2^- and NO_3^- averaged approximately 1.8, 0.41 and 0.23 mg N/L, respectively. Figure 3 shows the influent and effluent total phosphorus (TP) concentrations with averages of 3.5 and 0.5 mg P/L, respectively, indicating good biological phosphorus removal even at very low DO conditions. Based on the monthly operating reports for the first quarter of 2014, the effluent NH_4^+, NO_2^-, NO_3^- and PO_4^{3-} averaged 3 mgN/L, 0.4 mgN/L, 0.2 mgN/L and 0.1 mgP/L, respectively.

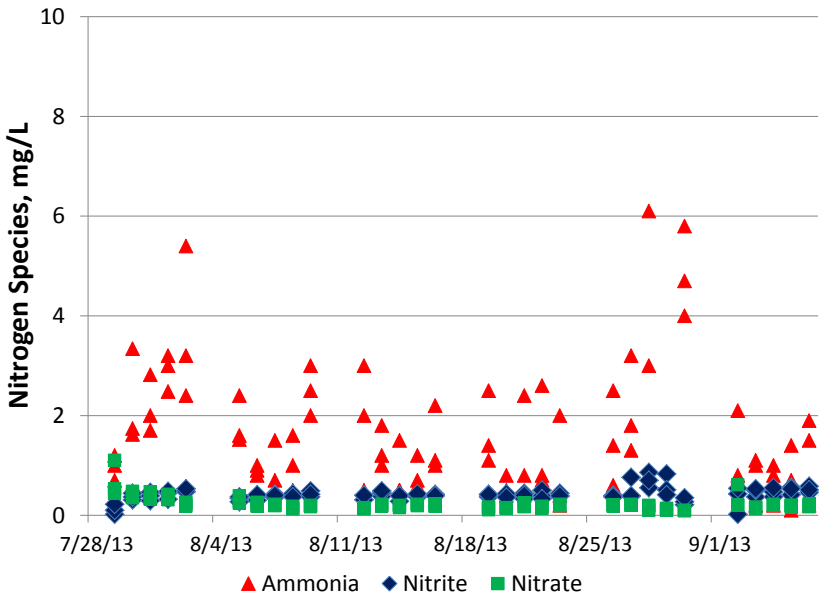

FIGURE 2: FINAL EFFLUENT AMMONIA, NITRATE AND NITRITE CONCENTRATIONS.

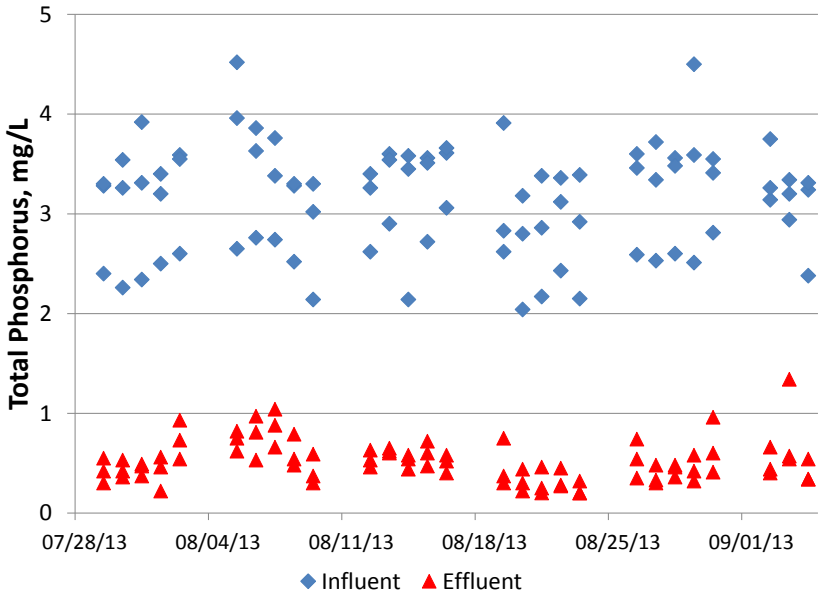

FIGURE 3: INFLUENT AND FINAL EFFLUENT TOTAL PHOSPHORUS CONCENTRATIONS.

2.0 BACKGROUND ON TESTING

This section provides further information on the objectives and methodology for the tests that were conducted at the St. Petersburg Southwest WRF. These included Specific Nitrification Rate Tests, Simultaneous Nitrification-Denitrification Rate Tests, Denitrification Rate Tests, a test to detect the presence of Anammox bacteria, and Phosphorus Release-Uptake Tests.

2.1 SPECIFIC NITRIFICATION RATE TEST

In a Specific Nitrification Rate (SNR) test, a volume of mixed liquor or RAS is collected from a nitrifying plant and mixed with a diluent (*e.g.* treatment plant influent wastewater, primary effluent, *etc.*). Ammonia typically is added to the batch test at the start to set a target initial concentration in the range of 25 to 35 mgN/L. The batch test is then fully-aerated at a DO concentration of at least 3 mg/L and the production of nitrite and nitrate (NO_X) is monitored over time (*e.g.* 4-5 hours). Because the relative change in nitrifier population is small over the duration of the test, typically there is a linear response in the nitrogen species. Figure 4 shows an example of the response of ammonia-, nitrite-, and nitrate-nitrogen in a fully-aerated SNR test. Total inorganic nitrogen (TIN) is also plotted. Linear regression analysis is used to estimate the NH_3 removal rate (ARR), observed nitrite accumulation rate (NO_2AR), nitrate production rate (NO_3PR) and NO_X production rate (NO_XPR). Dividing the ARR and NO_XPR by the batch volatile suspended solids concentration yields the specific NH_3 removal rate (SNH_3RR) and the specific NO_X production rate (SNO_xPR), as shown in Table 2. It should be noted that the SNH_3RR and SNO_xPR are different. Hence the TIN concentration changes slightly throughout the test. This is expected, since nitrification is not the only process impacting the NH_3 concentration in these tests. For example, NH_3 also is produced in the test via the ammonification of organic nitrogen from the influent wastewater or that released through heterotrophic bacteria decay; it is consumed as a cellular synthesis requirement during heterotrophic bacteria growth, *etc.* Because of the multiple processes impacting NH_3 concentration, it is important that ***only*** NO_X production be used to assess nitrification kinetics.

Although these tests do not yield an estimate of the maximum specific growth rate required for process modelling input, they can provide very useful quantitative information (comments on problems with estimating nitrifier maximum specific growth rates from SNR test data are provided in WERF, 2003).

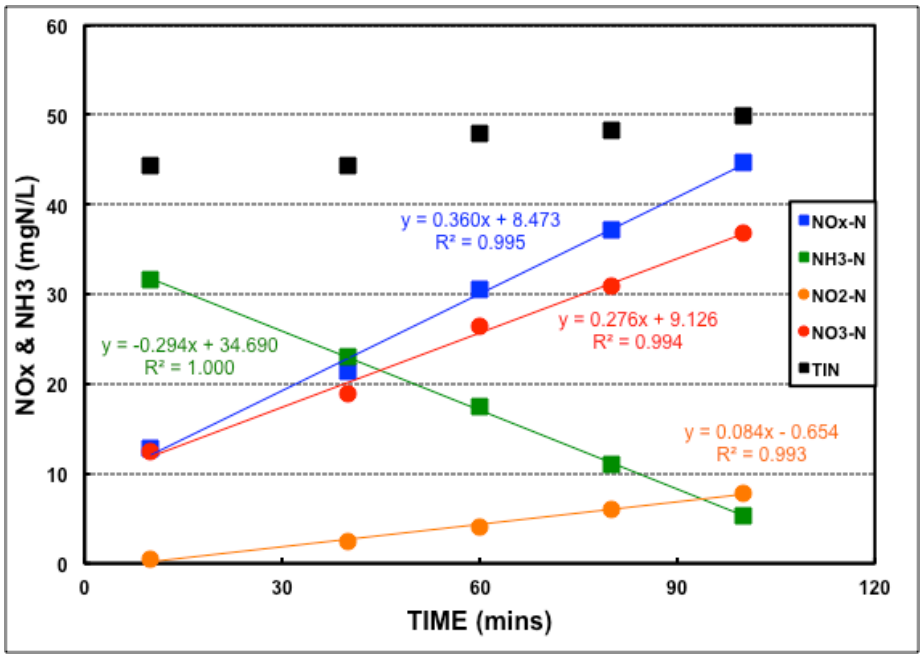

FIGURE 4: AMMONIA REMOVAL AND OXIDIZED NITROGEN PRODUCTION VS. TIME IN AN SNR TEST.

TABLE 2: SUMMARY OF RESULTS FOR EXAMPLE SNR TEST

VSS (mg/L)	3116
Average Test Temperature (°C)	21.9
NH_3RR (mgN/L/min)	0.294
SNH_3RR (mgN/gVSS/hr)	5.65
SNH_3RR corrected to 20°C (mgN/gVSS/hr)	4.96
NO_XPR (mgN/L/min)	0.360
SNO_XPR (mgN/gVSS/hr)	6.92
SNO_xPR corrected to 20°C (mgN/gVSS/hr)	6.07
NO_3PR (mgN/L/min)	0.276
NO_2AR (mgN/L/min)	0.084

The specific rates were corrected to 20°C using the following equation where θ is the Arrhenius value. An Arrhenius value of 1.072 was used for the SNH_3RR, SNO_3PR and SNO_XPR.

$$SNPR_{20} = SNPR_T\theta^{(20-T)} \quad [1]$$

In a *fully nitrifying* system, if all of the nitrite generated from AOB oxidation is in turn converted to nitrate

by NOB (i.e. no nitrite-shunt) then the ratio of NOB/AOB should equal the ratio of the respective yield coefficients. [An underlying assumption is that the decay rates of AOB and NOB are similar]. For example, if Y_{NOB} = 0.09 and Y_{AOB} = 0.15, then NOB/AOB = 0.6. In this way, the SNR test (performed at high test DO) may be used to assess whether nitrite-shunt is happening in the plant. The plant mixed liquor NOB/AOB ratio is estimated from test results and compared to the Y_{NOB}/Y_{AOB} ratio applied in analysis of the test data [exact knowledge of the yield coefficients is not necessary]. For these Y values if the NOB/AOB estimate from test results is less than 0.6 then some degree of NOB repression likely is occurring.

In the fully-aerated SNR test, the nitrite and nitrate accumulation rates ($\Delta NO_2/\Delta t$ and $\Delta NO_3/\Delta t$) are determined from the linear increase in nitrite and nitrate concentrations over the initial period before NH_3 is fully utilized. The ***production*** rates for nitrite and nitrate are:

$$NO_2R = \left(\frac{\mu_{AOB}}{Y_{AOB}}\right)\frac{S_{NH}}{K_{NH}+S_{NH}}\cdot X_{AOB}\cdot\frac{S_O}{K_{DO,AOB}+S_O}$$

$$NO_3R = \left(\frac{\mu_{NOB}}{Y_{NOB}}\right)\frac{S_{NO2}}{K_{NO2}+S_{NO2}}\cdot X_{NOB}\cdot\frac{S_O}{K_{DO,NOB}+S_O}$$

In the batch test nitrite is generated from NH_3 and converted to nitrate simultaneously, so the overall NO_2 production rate (NO_2PR) equals the observed nitrite accumulation (NO_2AR) rate plus the nitrate production rate (NO_3PR):

$$NO_3PR = \Delta NO_3/\Delta t$$

$$NO_2PR = \frac{\Delta NO_2}{\Delta t}+\frac{NO_3}{\Delta t}=\frac{\Delta NO_X}{\Delta t}=NO_XPR$$

The ratio NO_3PR/NO_XPR [i.e. $(\Delta NO_3/\Delta t)$ / $(\Delta NO_X/\Delta t)$] is linked directly to the ratio NOB/AOB (but not equal to NOB/AOB because NO_3PR/NO_XPR incorporates the maximum growth rates of AOBs and NOBs). For example, in a fully nitrifying plant the ratio typically will be close to 0.8. However, if a lower value is measured in a fully-aerated SNR test on plant mixed liquor, then it is likely that the NOB population is suppressed and nitrite shunt is occurring in the plant.

In the fully-aerated example SNR test in Figure 4, the NO_3PR is 0.276 mgN/L/min and the NO_XPR is 0.360 mgN/L/min, hence the NO_3PR/NO_XPR is 0.77. This indicates that NOB are not repressed and that nitrite-shunt is not occurring at the plant.

Another utility of the SNR test is that, by varying a single factor (*e.g.* DO concentration) between two tests, the two results can be compared to see if that factor impacts nitrification rate. For example, to investigate the impact of low DO conditions on the nitrification rate, a pair of SNR tests would be conducted where one test is fully aerated and the other test is operated at a low DO concentration. Figure 5 below shows an example of such an approach where mixed liquor from a fully-aerated activated sludge reactor was combined with raw influent wastewater in parallel SNR tests operated at respective DO concentrations of 5 mg/L (top chart) and 0.2 mg/L (bottom chart).

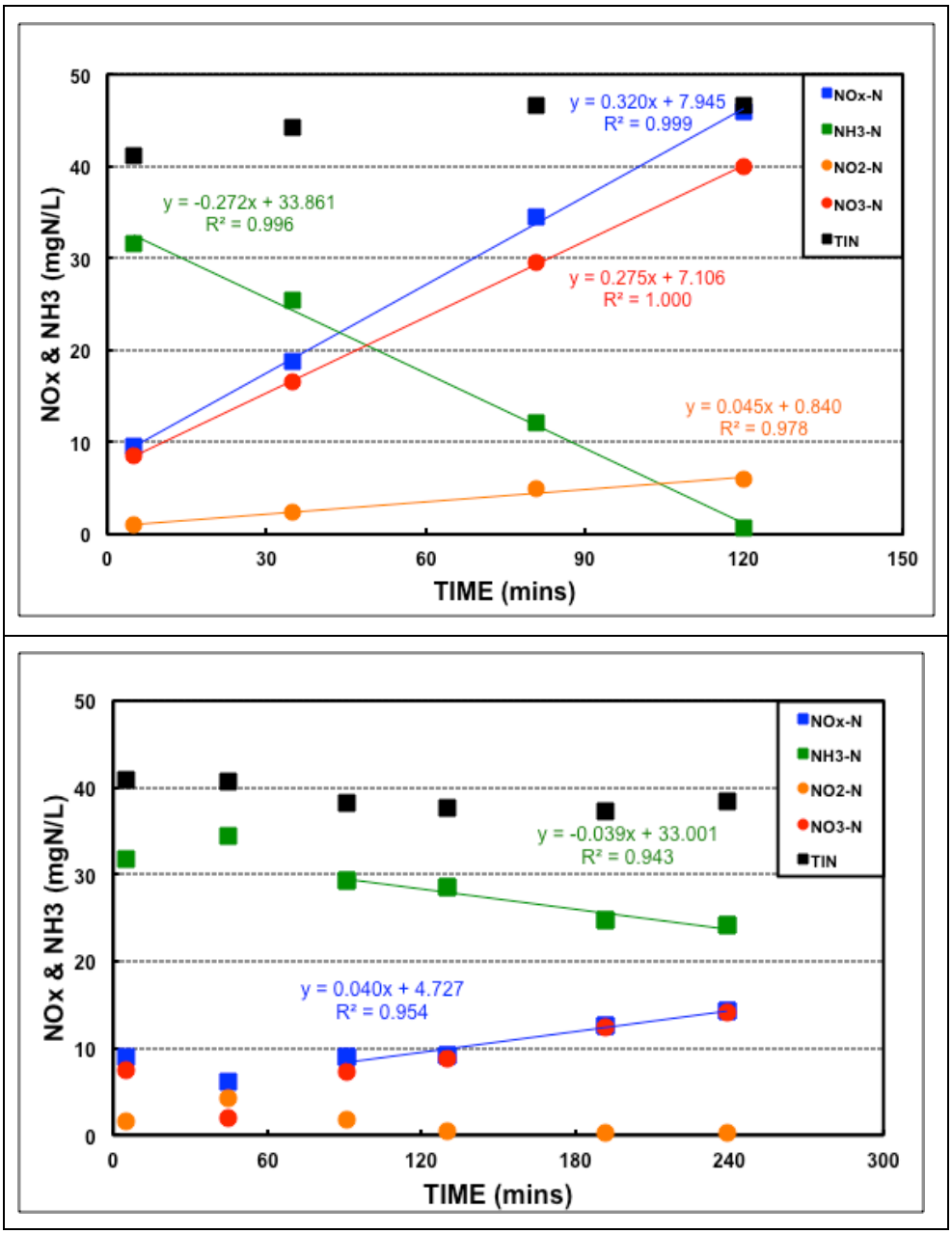

FIGURE 5: PARALLEL SNR TESTS OPERATED AT DO CONCENTRATIONS OF 5 MG/L (TOP CHART) AND 0.2 MG/L (BOTTOM CHART).

In the fully-aerated SNR test, the NO_3PR is 0.275 mgN/L/min and the NO_XPR is 0.320 mgN/L/min, hence the NO_3PR/NO_XPR is 0.86. This indicates that NOB are not repressed in the plant from which the mixed liquor was obtained. The SNO_XPR (at 20°C) for the fully aerated test was 5.78 mgN/gVSS/h whereas the SNO_XPR for the low DO test was 0.76 mgN/gVSS/h. This clearly demonstrates that nitrification would be substantially limited if the plant decreased the DO concentration in the aerated reactors to 0.2 mg/L. It should be noted that the TIN concentration changes slightly throughout both tests, as is expected because nitrification is not the only process impacting the NH_3 concentration in these tests. When viewing the impact of low DO on nitrification rates in these tests it should be recognized that simultaneous denitrification may be occurring and this can confuse the interpretation of data.

2.2 SIMULTANEOUS NITRIFICATION-DENITRIFICATION TEST

The simultaneous nitrification-denitrification (SND) test is operated identically to the SNR test except that the DO concentration is maintained at a low level and a spike of NO_2 or NO_3 is added at the start of the test so that denitritation or denitrification is not limited, in addition to NH_3.

In an SND test, a volume of mixed liquor or RAS is collected from a plant and mixed with treatment plant influent wastewater. Ammonia is added to the batch test at the start to set a target initial concentration in the range of 25 to 35 mgN/L. Nitrite or NO_3 is also added at the start typically at a target concentration of 30 mgN/L. The DO concentration in the batch test is then controlled at a desired low concentration (*e.g.* 0.3 mg/L) and the change of NH_3, NO_3, NO_2, NO_X and TIN is monitored over time (*e.g.* 6-8 hours). [In our case DO was controlled in a narrow band between lower and upper setpoints, switching aeration on/off. For example, in a test with a desired DO of 0.3 mg/L, setpoints of 0.25 and 0.35 mg/L were applied]. Because the relative change in nitrifier and OHO population is small over the duration of the test, it is expected that there will be a linear response in the nitrogen species. As with the SNR test, linear regression analysis is used to estimate the production or removal rates for NH_3, NO_3, NO_2, NO_X and TIN. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates.

In these tests at low DO levels NO_2 and NO_3 can be both generated and consumed. This complicates the analysis of results. The principal indicator of simultaneous nitrification-denitrification is a decrease in the TIN during the test.

2.3 SPECIFIC DENITRIFICATION RATE TEST

In a Specific Denitrification Rate (SDR) Test, a volume of mixed liquor or RAS is combined with an organic carbon source (*e.g.* treatment plant influent wastewater, acetate, *etc.*). Nitrite and/or NO_3 are added at the start of the batch test as electron acceptors. The batch test is then mixed and is not aerated. The mixing speed is adjusted so that the liquid is adequately mixed while avoiding the creation of a vortex that could entrain air into the liquid. The liquid surface is covered with ping-pong balls to further limit the surface transfer of oxygen. The removal of NO_2 and/or NO_3 is monitored over time (*e.g.* 6-8 hours). Because the relative change in OHO population is small over the duration of the test, typically there is a linear response in the nitrogen species. Figure 6 shows an example of the response with sequential spikes of nitrite- and nitrate-nitrogen in an SDR test with acetate as the electron donor. At the start of the test, NO_2 and NO_3 were dosed to achieve initial starting concentrations of 25 and 5 mgN/L. The NH_3 concentration (not plotted) remained around 1 mgN/L throughout the test. Linear regression analysis is used to estimate the NO_3 removal rate (NO_3RR) and nitrite removal rate (NO_2RR). Dividing the NO_3RR and NO_2RR by the batch volatile suspended solids concentration yields the specific NO_3RR (SNO_3RR) and the specific NO_2RR (SNO_2RR), as shown in Table 3.

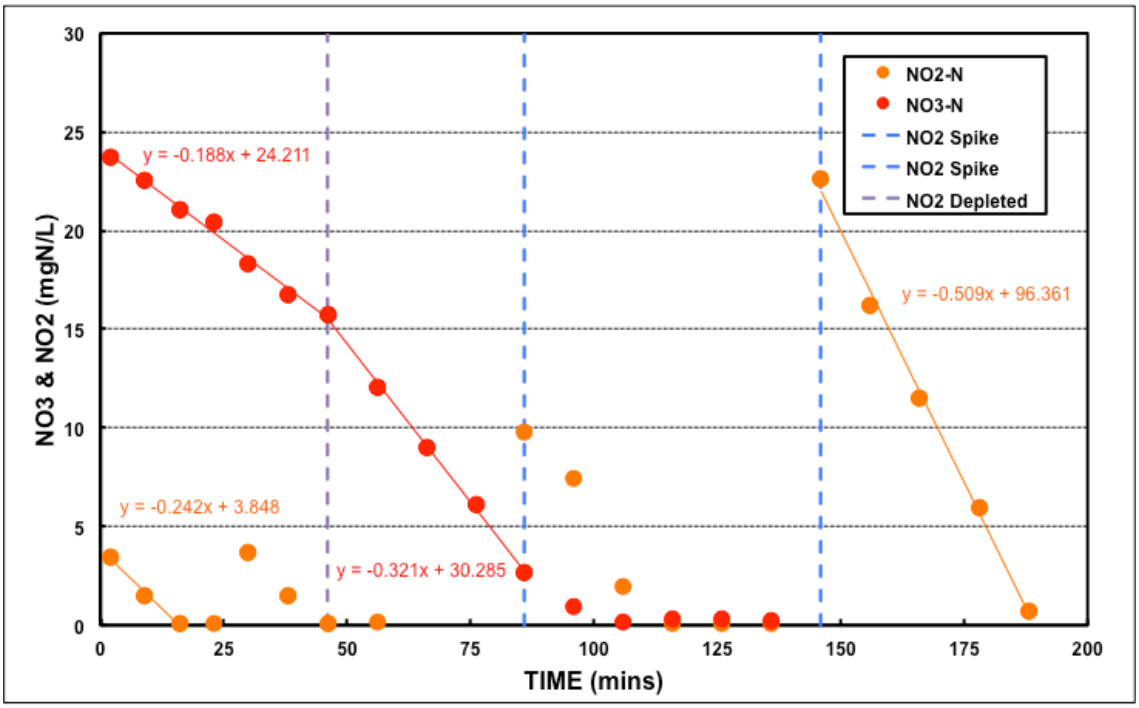

FIGURE 6: NITRATE AND NITRITE REMOVAL VS. TIME IN AN SDR TEST WITH ACETATE AS THE ELECTRON DONOR.

TABLE 3: SUMMARY OF RESULTS FOR EXAMPLE SDR TEST

VSS (mg/L)	2510
Average Test Temperature (°C)	22.4
NO_3RR in Presence of NO_2 (mgN/L/min)	0.188
SNO_3RR in Presence of NO_2 corrected to 20°C (mgN/gVSS/hr)	4.20
NO_3RR in Absence of NO_2 (mgN/L/min)	0.321
SNO_3RR in Absence of NO_2 corrected to 20°C (mgN/gVSS/hr)	7.16
NO_2RR in Presence of NO_3 (mgN/L/min)	0.242
SNO_2RR in Presence of NO_3 corrected to 20°C (mgN/gVSS/hr)	5.40
NO_2RR in Absence of NO_3 (mgN/L/min)	0.509
SNO_2RR in Absence of NO_3 corrected to 20°C (mgN/gVSS/hr)	11.36

The specific rates were corrected to 20°C using Equation [1]. An Arrhenius value of 1.029 was used for the SNO_3RR and SNO_2RR.

As shown in Figure 6, NO_2 and NO_3 were removed simultaneously during the first 46 minutes of the test. Once NO_2 was depleted, NO_3 was removed at a faster rate than the NO_3RR in the presence of NO_2. Once NO_3 was depleted, NO_2 was dosed at 146 minutes. The NO_2RR in the absence of NO_3 was faster than the NO_2RR when NO_3 was present. It is interesting to note that the NO_3RR in the absence of NO_2 was 63% of the NO_2RR in the absence of NO_3. This is expected because the oxygen states of NO_3 and NO_2 are +5 and +3, respectively, and with reduction to nitrogen gas (oxidation state 0) the ratio of

electrons transferred will be 5:3.. Hence, per unit COD oxidized, NO_3 is reduced to N_2 gas at a rate that is 3/5 (*i.e.* 60%) the rate that NO_2 is reduced to N_2 gas.

2.4 TEST TO DETECT PRESENCE OF ANAMMOX BACTERIA

A single batch test was carried out to determine whether ANaerobic AMMonia OXidizing (Anammox) bacteria are present in the activated sludge at the Southwest WRF. Anammox bacteria convert NH_4 and NO_2 to N_2 (and a small fraction of NO_3) under anaerobic conditions according to the following empirical equation:

$$NH_4^+ + NO_2^- \rightarrow 0.86N_2 + 0.14NO_3^-$$

In this process NH_4^+ is used as the electron donor and NO_2^- as the electron acceptor. The energy from this process is used to synthesize organic material from inorganic carbon (*i.e.* fixing CO_2). Organic carbon is not utilized so the process neither increases biosolids production nor generates CO_2.

In the test, a volume of mixed liquor or RAS is added to a batch reactor. Ammonia and NO_2 are dosed to initial target concentrations of 30 and 20 mgN/L, respectively. The batch test is then mixed and left unaerated. The mixing speed is adjusted so that the liquid is adequately mixed while avoiding the creation of a vortex that could entrain air into the liquid. The liquid surface is covered with ping-pong balls to further limit the surface transfer of oxygen. If anammox bacteria are present, it is expected that NH_4 and NO_2 will be removed at equal rates. Because the relative change in bacterial population is small over the duration of the test, there will be a linear response in the nitrogen species.

2.5 PHOSPHORUS RELEASE AND UPTAKE TEST

A phosphorus release and uptake test is useful as a check to confirm that enhanced biological phosphorus removal (EBPR) is occurring at a WWTP. This test is a useful indicator because phosphorus release and uptake will ***only*** be measured if EBPR is occurring in the system. Knowing whether the current plant configuration / operation is achieving the intended biological phosphorus removal will be of use in decisions impacting nitrification performance. For example, factors such as total overall SRT and the relative bioreactor divisions into aerobic, anaerobic, and anoxic zones impact both EBPR and nitrification performance.

Briefly, the procedure for conducting the phosphorus release and uptake tests is as follows:

- A sample of mixed liquor or RAS is collected from the plant.
- The VSS and TSS concentrations of the mixed liquor sample are measured.
- A volume of the mixed liquor or RAS sample is placed in a beaker on a mixing stand. The mixing speed is adjusted so that the liquid is adequately mixed while avoiding the creation of a vortex that could entrain air into the liquid. The liquid surface is covered with ping-pong balls to further limit the surface transfer of oxygen.
- A solution of sodium acetate is added to the beaker with the intention of achieving an initial soluble COD in the batch test in the range of 100 to 200 mg/L.
- Samples are collected from the batch at regular intervals for analysis of soluble orthophosphate and soluble COD. The soluble COD is obtained by filtering the sample through a 1.5 µm glass fibre filter.
- Once the release of phosphate with time has reached a plateau, an electron acceptor (*i.e.* NO_X or oxygen) is introduced into the reactor. The phosphate uptake rate is monitored over time.

A typical response for a phosphorus release test (not including uptake) for a mixed liquor sample withdrawn from a treatment plant achieving EBPR is shown in Figure 7 below. Soluble phosphorus is released by phosphorus accumulating organisms as they take up soluble COD and store it for oxidation later in the EBPR process. In this particular test the initial P release rate over the first 20 or 30 minutes was reduced because there was some NO_3 in the mixed liquor sample, and anoxic P uptake reduced the net release rate until all the NO_3 was removed.

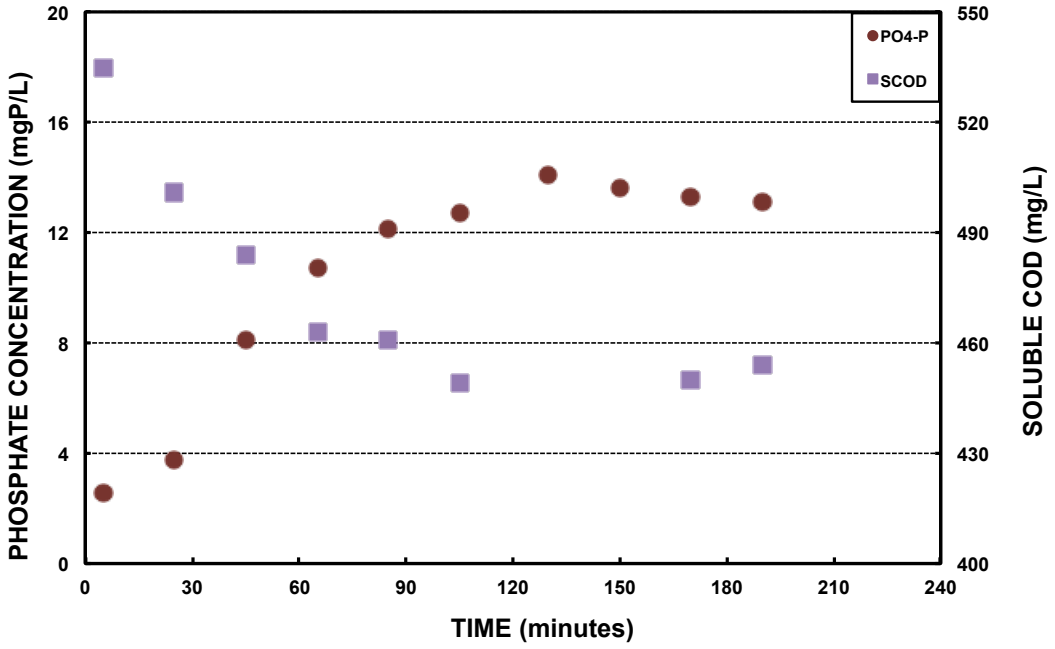

FIGURE 7: TYPICAL RESULTS FROM A PHOSPHORUS RELEASE TEST.

3.0 RESULTS AND DISCUSSION

Each of the SNR, SND, SDR, Anammox and P-Release tests were operated in reactors similar to the setup shown in Figure 8. The 10 L reactors were operated in parallel, allowing two tests to be run simultaneously.

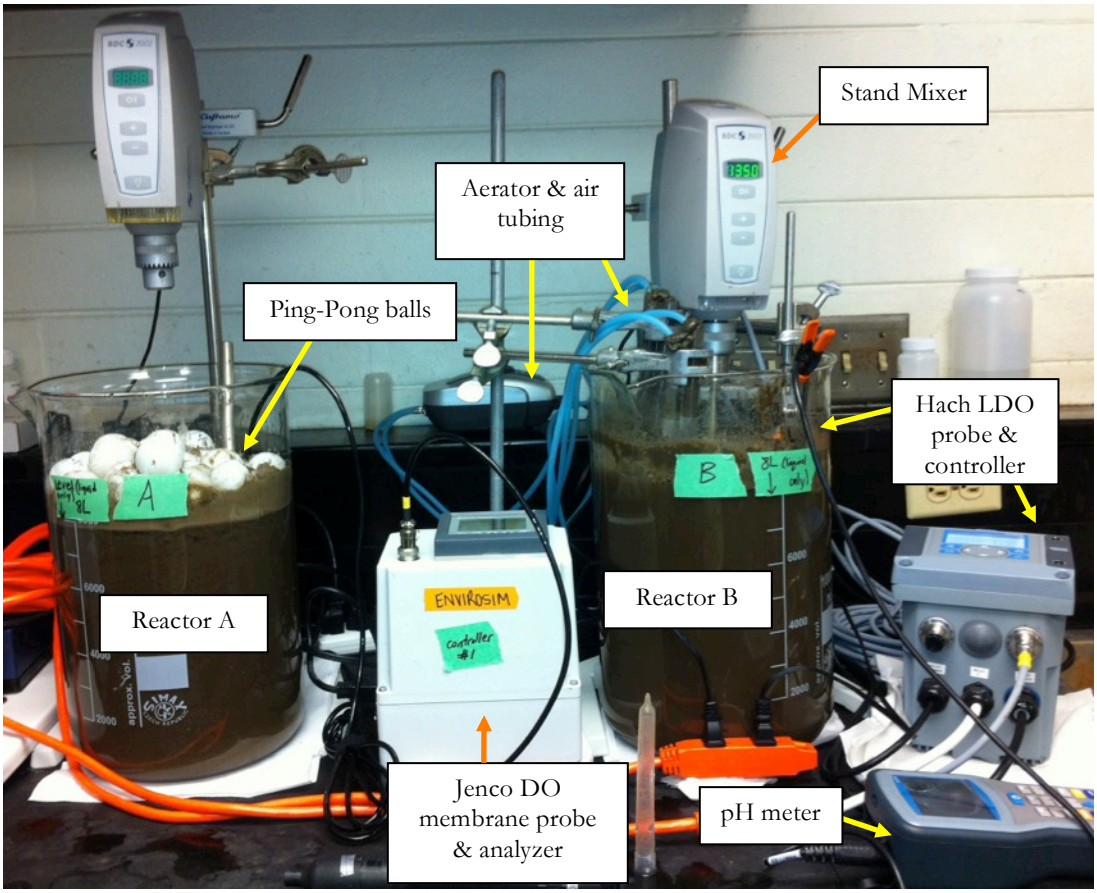

FIGURE 8: TYPICAL SETUP OF DAILY BATCH TESTS OPERATED BY ESA AT THE SOUTHWEST WRF.

As previously mentioned, the St. Petersburg Southwest WRF typically operates with aeration basins at a DO level of 0.1 to 0.4 mg/L. To mimic these conditions in the batch tests, a Hach LDO probe was used in combination with a Hach sc200 DO controller to maintain the DO concentration in the reactor at a certain low level. For example, to achieve a DO setpoint of 0.2 mg/L, the controller was programmed to turn on the aerators when the DO concentration dropped below 0.15 mg/L and turn off the aerators when the DO concentration rose above 0.25 mg/L. Before starting the tests, the LDO probe was immersed in a saturated solution of sodium sulphite, *i.e.* in the absence of DO. After 1 hour in the solution, the LDO probe reported a consistent concentration of 0.09 mg/L. Thus it is believed that this reading of 0.09 mg/L on the sc200 controller interface in fact corresponds to "zero" DO. In this report, the DO reading on the controller interface will be used. However, it is important to bear in mind that the *actual* DO, especially at low DO concentrations, likely is 0.09 mg/L less than the reported reading. Thus the DO setpoints of 0.15 and 0.25 mg/L correspond to actual DO concentrations of 0.06 and 0.16 mg/L.

A Jenco membrane DO probe (ID-900-3-DO) connected to a Jenco analyzer (Model 6309PDT) was used to measure the DO concentration in the reactor run alongside the reactor with the Hach LDO probe and controller. A "zero" DO reading on this probe corresponds to an actual DO concentration of 0 mg/L.

3.1 SPECIFIC NITRIFICATION RATE TESTS

Four SNR tests were conducted; two tests under fully aerated conditions and two tests at low DO setpoints of 0.4 and 0.2 mg/L, respectively. The fully aerated SNRs were used to determine the NO_3PR/NO_2PR ratio and hence the NOB/AOB ratio to determine whether the NOB population is

suppressed and nitrite-shunt is occurring in the plant. The SNRs operated at low DO levels were used to determine the impact of low DO on the nitrification rate.

Each of the four SNR tests was seeded with a grab sample of 4 L of RAS from the Southwest WRF. Grab samples of raw influent from lines 741 A and 741 B were combined in a volumetric ratio of 3:1 in a bucket, to match the flow proportion received at the plant. In each SNR test, 4 L of this combined influent was added. Ammonium chloride was added such that the initial NH_3 concentration was approximately 25 mgN/L. A small amount of supplemental phosphorus (in the form of potassium dihydrogen phosphate) was added to ensure nutrient limitations would not impact the test. Additional alkalinity in the form of sodium bicarbonate was added as needed to ensure stable pH throughout the test. Each test was aerated using aquarium air pumps and air stones, and a stand mixer provided mixing. The results from each test are summarized in the following sections.

3.1.1 SNR #1 (FULLY AERATED)

The first SNR test was fully aerated to achieve a DO concentration of at least 5 mg/L. The DO concentration was measured at the sample times using the Jenco membrane DO probe and analyzer. At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for ammonia-, nitrite-, and nitrite-nitrogen.

Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses for NH_3 removal and oxidized nitrogen production, as shown in Figure 9.

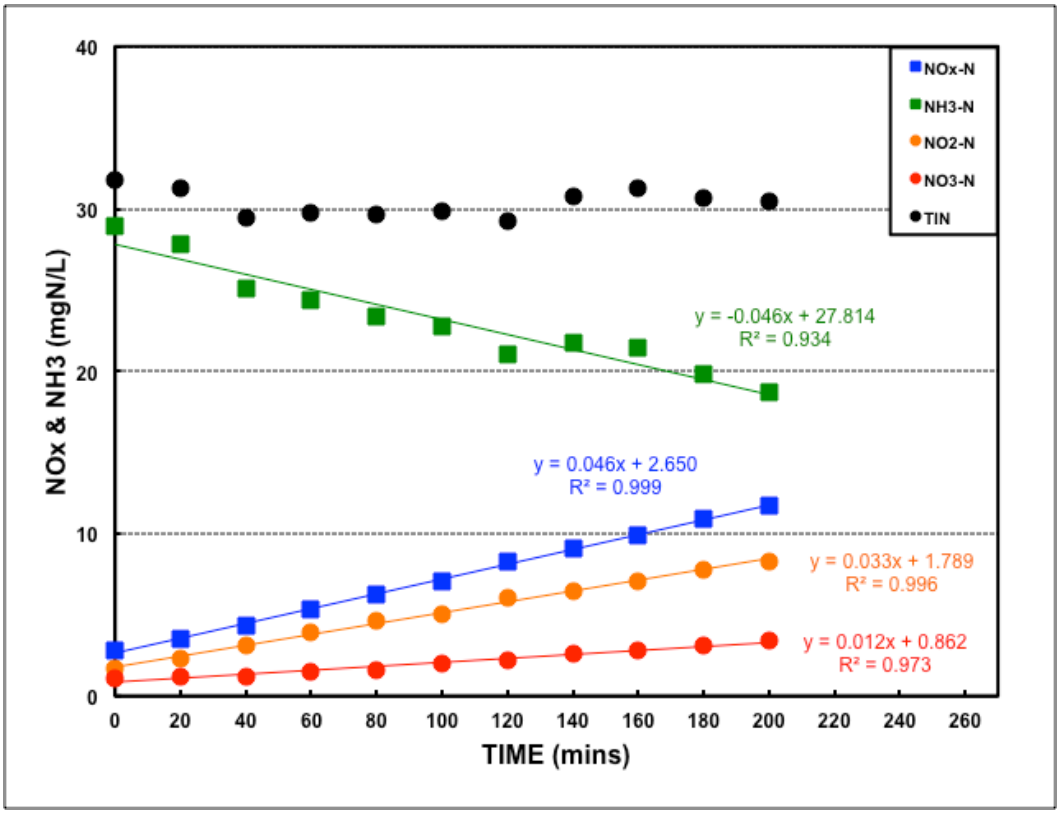

FIGURE 9: AMMONIA REMOVAL AND OXIDIZED NITROGEN PRODUCTION VERSUS TIME FOR SNR TEST #1 (DO CONCENTRATION 5 MG/L).

Linear regression analysis is used to estimate the removal rate of NH_3 and production rates of NO_2, NO_3 and NO_X. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates, as shown in Table 4.

TABLE 4: SUMMARY OF RESULTS FOR SNR TEST #1

VSS (mg/L)	2443
Average Test Temperature (°C)	22.5
NH_3RR (mgN/L/min)	0.046
SNH_3RR (mgN/gVSS/hr)	1.14
SNH_3RR corrected to 20°C (mgN/gVSS/hr)	0.96
NO_XPR (mgN/L/min)	0.046
SNO_XPR (mgN/gVSS/hr)	1.12
SNO_xPR corrected to 20°C (mgN/gVSS/hr)	0.94
NO_3PR (mgN/L/min)	0.012
SNO_3PR (mgN/gVSS/hr)	0.30
SNO_3PR corrected to 20°C (mgN/gVSS/hr)	0.25
NO_2AR (mgN/L/min)	0.033
SNO_2AR (mgN/gVSS/hr)	0.81

The specific rates were corrected to 20°C using Equation [1]. An Arrhenius value of 1.072 was used for the SNH_3RR, SNO_3PR and SNO_XPR.

As previously mentioned, the ratio NO_3PR/NO_XPR is linked directly to the ratio NOB/AOB (but not equal to NOB/AOB). For example, in a fully nitrifying plant with balanced AOB and NOB populations the ratio should be close to 0.8. In SNR Test #1, the SNO_3PR is 0.25 mgN/L/min and the SNO_XPR is 0.94 mgN/L/min, hence the NO_3PR/NO_XPR is 0.27. This indicates that NOB are repressed significantly and that nitrite-shunt is occurring at the plant. The TIN concentration remained relatively constant throughout the test.

3.1.2 SNR #2 (FULLY AERATED)

The second SNR test was operated identically to the first SNR test in order verify the results. Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses for NH_3 removal and oxidized nitrogen production, as shown in Figure 10.

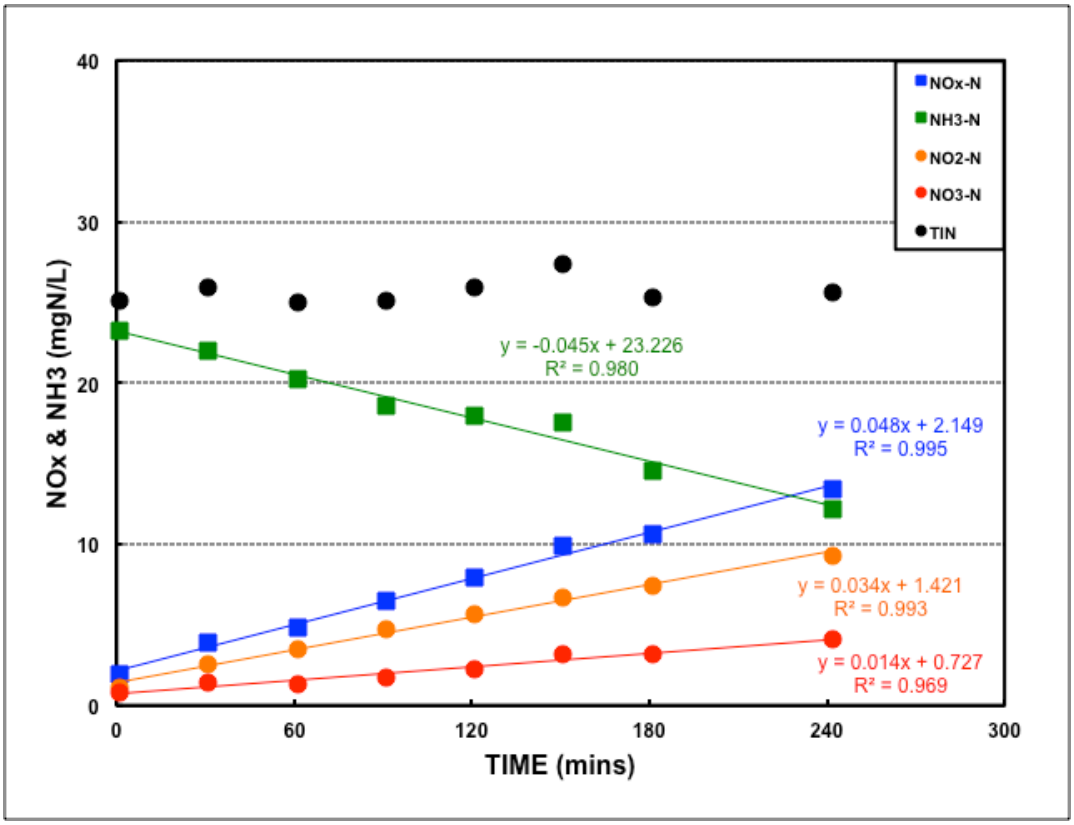

FIGURE 10: AMMONIA REMOVAL AND OXIDIZED NITROGEN PRODUCTION VERSUS TIME FOR SNR TEST #2 (DO CONCENTRATION 5 MG/L).

Linear regression analysis is used to estimate the removal rate of NH_3 and production rates of NO_2, NO_3 and NO_X. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates, as shown in Table 5.

TABLE 5: SUMMARY OF RESULTS FOR SNR TEST #2

VSS (mg/L)	2280
Average Test Temperature (°C)	22.7
NH_3RR (mgN/L/min)	0.045
SNH_3RR (mgN/gVSS/hr)	1.18
SNH_3RR corrected to 20°C (mgN/gVSS/hr)	0.98
NO_XPR (mgN/L/min)	0.048
SNO_XPR (mgN/gVSS/hr)	1.26
SNO_xPR corrected to 20°C (mgN/gVSS/hr)	1.04
NO_3PR (mgN/L/min)	0.014
SNO_3PR (mgN/gVSS/hr)	0.37
SNO_3PR corrected to 20°C (mgN/gVSS/hr)	0.30
NO_2AR (mgN/L/min)	0.034
SNO_2AR (mgN/gVSS/hr)	0.89

In SNR Test #2, the NO_3PR/NO_XPR ratio was 0.29, which is statistically equivalent to the NO_3PR/NO_XPR ratio of 0.27 obtained in SNR Test #1. This provides further evidence that NOB are repressed and that nitrite-shunt is occurring at the plant.

3.1.3 SNR #3 (DO CONCENTRATION 0.4 MG/L)

The third SNR test was operated at a DO setpoint of 0.4 mg/L, which is close to the upper limit of the DO range at which the aeration basins at the Southwest WRF are operated. Plotting the ammonia-, nitrate- and nitrite-nitrogen data *versus* time results in linear responses for NH_3 removal and oxidized nitrogen production, as shown in Figure 11.

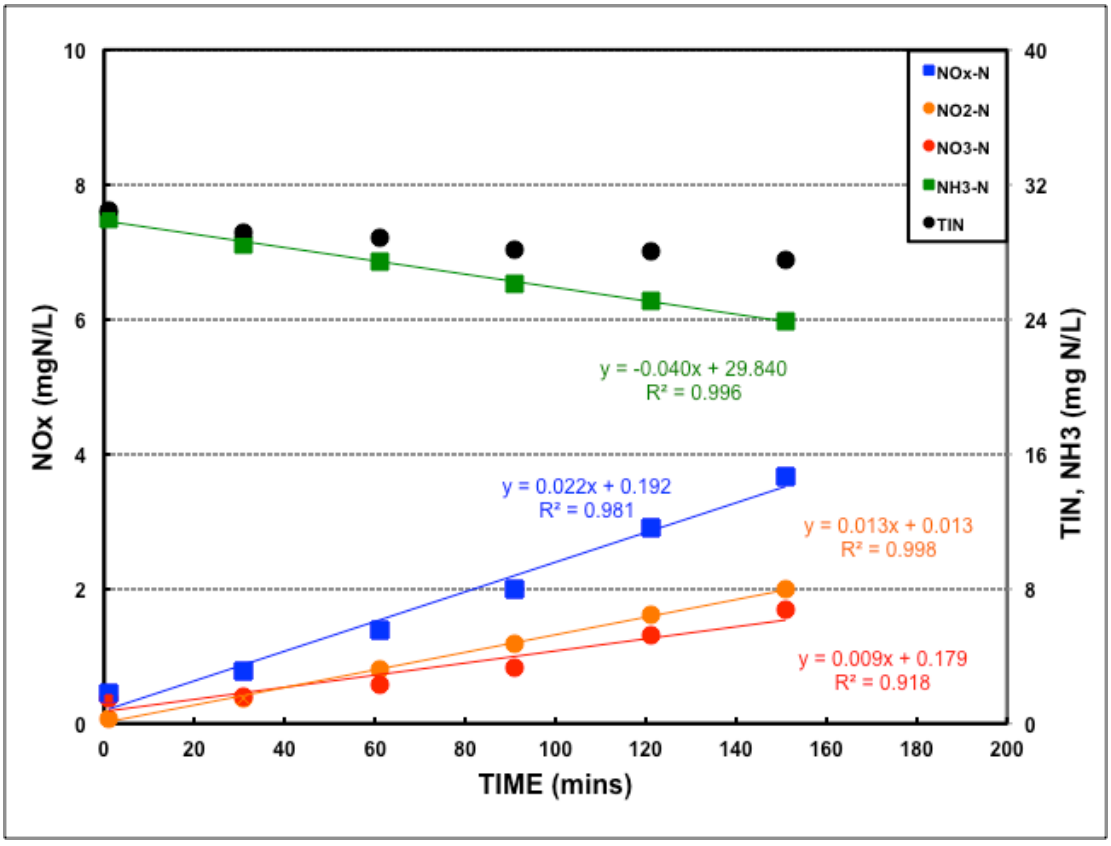

FIGURE 11: AMMONIA REMOVAL AND OXIDIZED NITROGEN PRODUCTION VERSUS TIME FOR SNR TEST #3 (DO CONCENTRATION 0.4 MG/L).

The results of SNR Test #3 are shown in Table 6.

TABLE 6: SUMMARY OF RESULTS FOR SNR TEST #3

VSS (mg/L)	2217
Average Test Temperature (°C)	22.4
NH_3RR (mgN/L/min)	0.040
SNH_3RR (mgN/gVSS/hr)	1.07
SNH_3RR corrected to 20°C (mgN/gVSS/hr)	0.91
NO_XPR (mgN/L/min)	0.022
SNO_XPR (mgN/gVSS/hr)	0.60
SNO_XPR corrected to 20°C (mgN/gVSS/hr)	0.51
NO_3PR (mgN/L/min)	0.009
SNO_3PR (mgN/gVSS/hr)	0.24
SNO_3PR corrected to 20°C (mgN/gVSS/hr)	0.21
NO_2AR (mgN/L/min)	0.013
SNO_2AR (mgN/gVSS/hr)	0.35

As shown in Figure 11, the TIN concentration decreased throughout the test indicating a degree of simultaneous denitritation and/or denitrification. The NO_3PR/NO_XPR ratio was not evaluated for SNR Test #3 since this test was operated at a low DO concentration.

The DO controller was programmed to maintain the DO concentration in the batch test between 0.35 and 0.45 mg/L. As shown in Figure 12, the measured DO concentration actually exceeded this range. However the overall average measured DO concentration during the test was 0.37 mg/L.

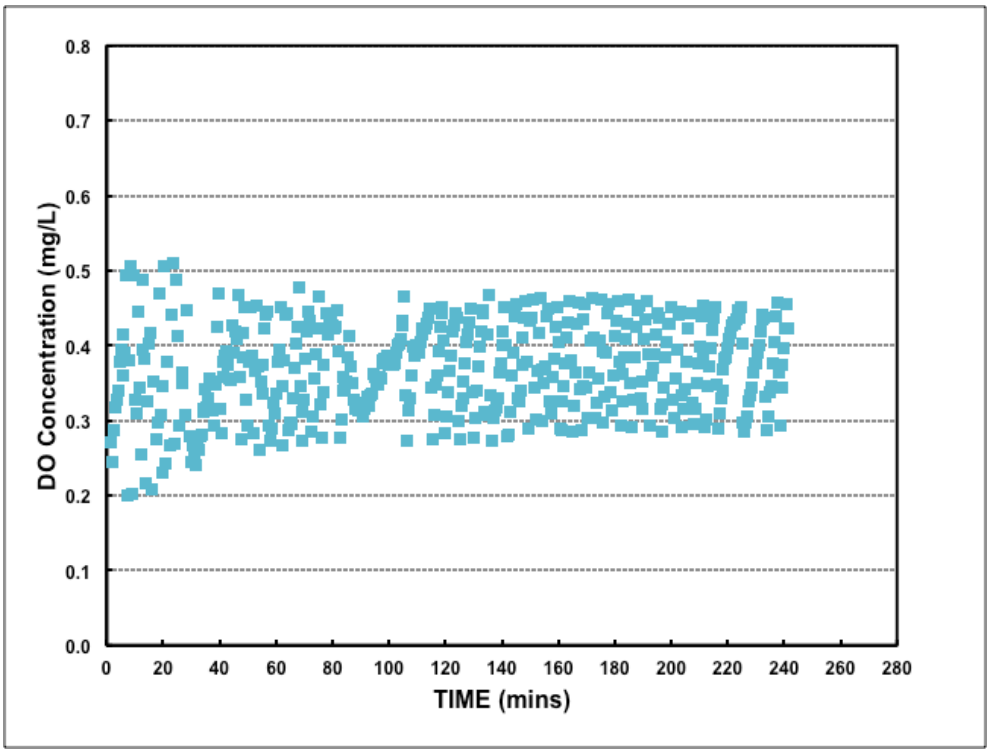

FIGURE 12: MEASURED DO CONCENTRATION IN SNR TEST #3.

3.1.4 SNR #4 (DO CONCENTRATION 0.2 MG/L)

The fourth SNR test was operated at a DO setpoint of 0.2 mg/L, which is approximately the average of the range of DO levels at which the aeration basins at the Southwest WRF are operated.

The plot of the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time is shown in Figure 13. It is clear from the data that nitrification was not observed during SNR Test #4. The TIN concentration remained relatively constant throughout the test.

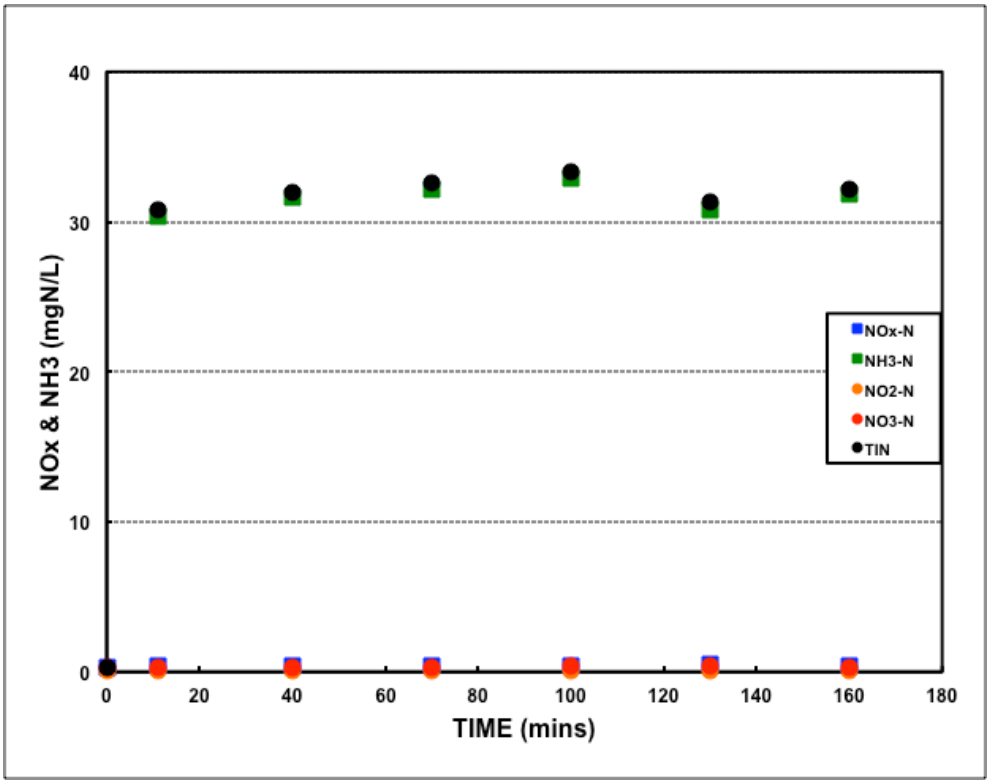

FIGURE 13: AMMONIA AND OXIDIZED NITROGEN CONCENTRATION VERSUS TIME FOR SNR TEST #4 (DO CONCENTRATION 0.2 MG/L).

The DO controller was programmed to maintain the DO concentration in the batch test between 0.15 and 0.25 mg/L. As shown in Figure 14, there was some overshoot in DO concentration over the initial period. However the overall average measured DO concentration during the test was 0.21 mg/L.

At the low DO in SNR #4 there was very little nitrification activity.

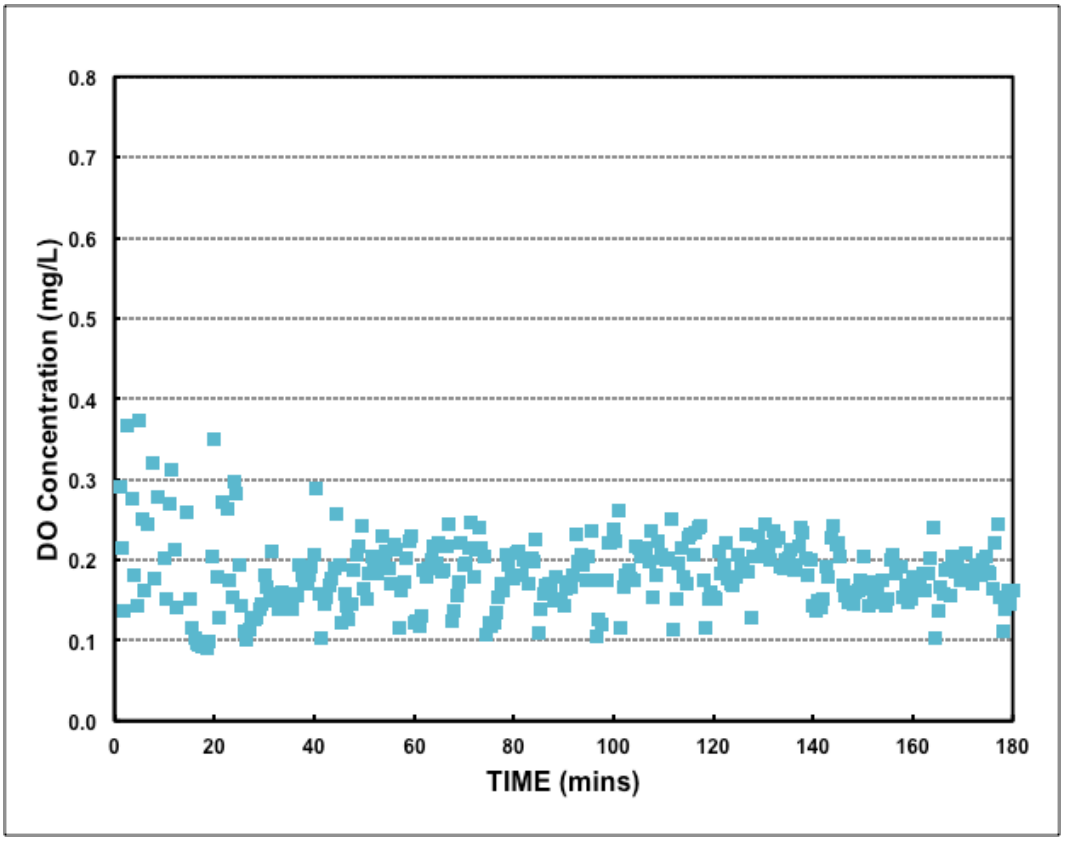

FIGURE 14: MEASURED DO CONCENTRATION IN SNR TEST #4.

3.1.5 SUMMARY OF SNR TESTS

Table 7 summarizes the SNR tests conducted.

TABLE 7: SUMMARY OF SNR TESTS CONDUCTED AT SOUTHWEST WRF

SNR #	AVG. DO (mg/L)	Batch Test MLVSS (mg/L)	SNH_3RR (mgN/gVSS /h)	SNO_XPR (mgN/gVSS /h)	SNO_3PR (mgN/gVSS /h)	SNO_3PR/SNO_XPR
1	5	2443	0.96	0.94	0.25	0.27
2	5	2280	0.98	1.04	0.30	0.29
3	0.37	2217	0.91	0.51	0.21	-
4	0.21	2300	0	0	0	-

As previously mentioned, the reading of 0.09 mg/L on the sc200 controller interface likely corresponds to zero DO with the Hach LDO probe. Thus the average measured DO concentration in SNR Test #4 of 0.21 mg/L probably was only 0.12 mg/L. Similarly, the reported average DO for SNR #3 of 0.37 mg/L likely was only 0.28 mg/L.

When assessing these results it is difficult to quantify the impact of DO on nitrification. At low DO NO_2 and/or NO_3 may be consumed through denitrification as was evident in SNR #3. Earlier it was noted that

NH_3 removal rate does not exactly reflect NO_2 production rate by AOB. However, SNH_3RR is at least a reasonable indicator of AOB activity. On this basis it is interesting to note that the SNH_3RR only decreases by approximately 7% from SNR #1 and #2 at high DO to SNR #3 at DO of 0.28 mg/L. Therefore one would conclude that the K_{DO} value for AOB is less than 0.28 mg/L. Because nitrite-shunt is occurring it is not possible to draw any conclusions on K_{DO} for the NOB.

3.2 SIMULTANEOUS NITRIFICATION-DENITRIFICATION TEST

Two SND tests were conducted; one with an initial target NO_3 concentration of 30 mgN/L, the other with an initial target NO_2 concentration of 30 mgN/L. In both tests, NH_3 was added at the start to set a target initial concentration of 25 mgN/L and the DO concentration was maintained at a setpoint of 0.4 mg/L.

Each test was seeded with a grab sample of 4 L of RAS from the Southwest WRF. Grab samples of raw influent from lines 741 A and 741 B were combined in a volumetric ratio of 3:1 in a bucket, to match the flow proportion received at the plant. In each SND test, 4 L of this combined influent was added. A small amount of supplemental phosphorus (in the form of potassium dihydrogen phosphate) was added to ensure nutrient limitations would not impact the test. Additional alkalinity in the form of sodium bicarbonate was added to ensure stable pH throughout the test. At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. Each test was aerated using aquarium air pumps and air stones, and a stand mixer provided mixing. The results from each test are summarized in the following sections.

3.2.1 SND #1 (DO CONCENTRATION 0.4 MG/L)

Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses, as shown in Figure 15.

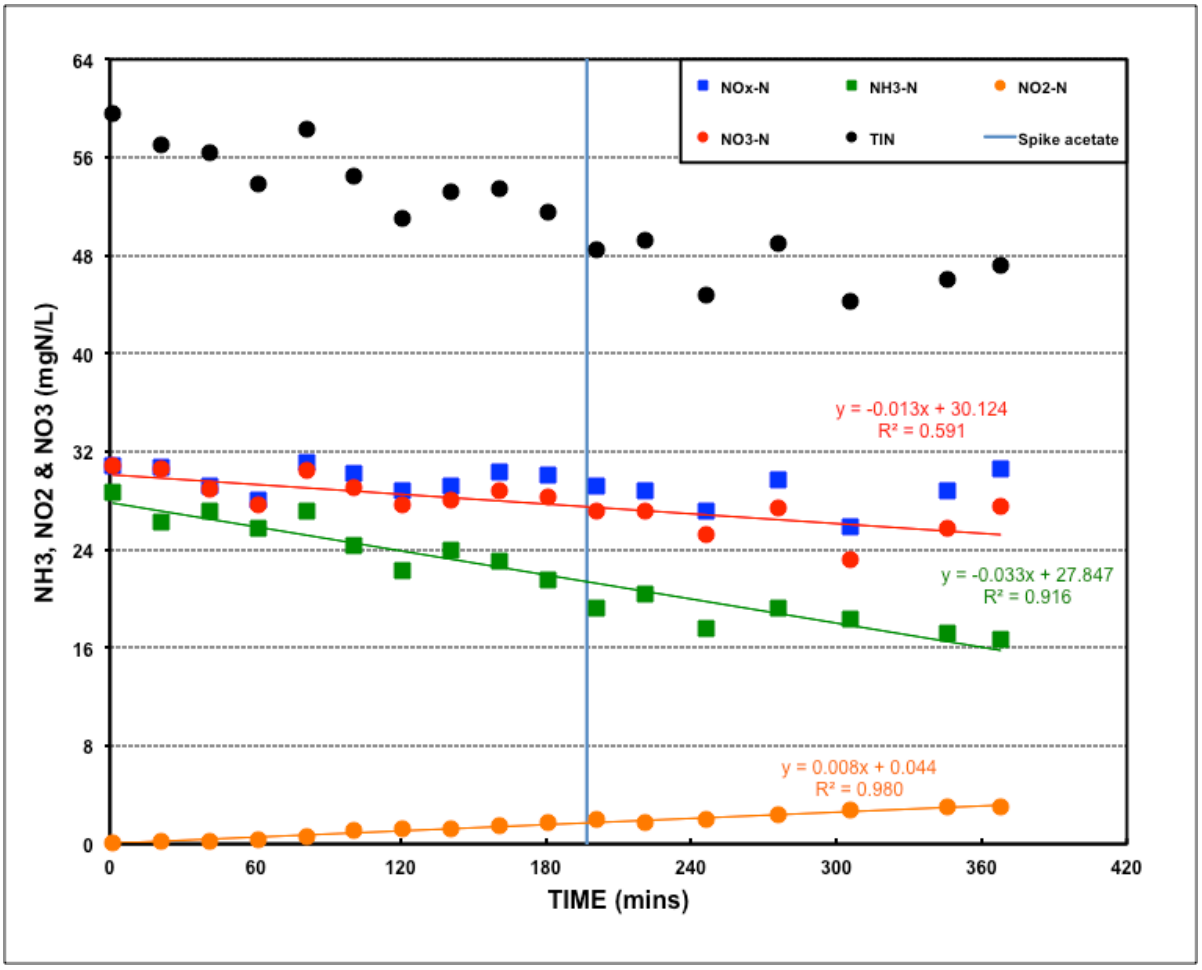

FIGURE 15: AMMONIA AND OXIDIZED NITROGEN CONCENTRATION VERSUS TIME FOR SND TEST #1 (DO CONCENTRATION 0.4 MG/L).

Linear regression analysis is used to estimate the removal rate of NH_3 and NO_3 accumulation rate of NO_2. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates, as shown in Table 8.

TABLE 8: SUMMARY OF RESULTS FOR SND TEST #1

VSS (mg/L)	2340
Average Test Temperature (°C)	23.2
NH_3RR (mgN/L/min)	0.033
SNH_3RR (mgN/gVSS/hr)	0.84
SNH_3RR corrected to 20°C (mgN/gVSS/hr)	0.67
NO_3RR (mgN/L/min)	0.013
SNO_3RR (mgN/gVSS/hr)	0.34
SNO_3RR corrected to 20°C (mgN/gVSS/hr)	0.31
NO_2AR (mgN/L/min)	0.009
SNO_2AR (mgN/gVSS/hr)	0.22
SNO_2AR corrected to 20°C (mgN/gVSS/hr)	0.18

The specific rates were corrected to 20°C using Equation [1]. An Arrhenius value of 1.072 was used for the SNH_3RR, an Arrhenius value of 1.029 was used for the SNO_3RR and an Arrhenius value of 1.060 was used for SNO_2AR.

As shown in Figure 15, a substantial decrease in the TIN (*i.e.* approximately 15 mgN/L) was observed. The NH_3 concentration decreased 12 mgN/L whereas the NO_2 concentration increased only 3 mgN/L. This clearly demonstrates the occurrence of the nitrite-shunt process. A small portion of NO_3 was also removed as the NO_3 concentration decreased approximately 2 mgN/L over the test.

The SNH_3RR (corrected to 20°C) is 0.67 mgN/gVSS/h, which is close to the measured value of 0.91 in SNR Test #3 (also operated at a DO concentration of 0.4 mg/L). The DO concentration was controlled to remain between setpoints of 0.35 and 0.45 mg/L using the Jenco membrane DO probe connected to the Jenco analyzer. Although the DO concentration was not automatically recorded, it was measured at the times the samples were taken. The average DO concentration throughout the test was 0.4 mg/L.

3.2.2 SND #2 (DO CONCENTRATION 0.4 MG/L)

Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses for NH_3 removal and oxidized nitrogen production, as shown in Figure 16.

Before commenting on the results, it is pertinent to note that in this test the Hach LDO probe was used whereas the Jenco probe was used in SND #1. The Hach probe response was very confusing over the first part of the test until approximately 180 minutes. The probe showed DO values much higher than the desired level of 0.4 mg/L, and the controller switched aeration off for that whole period. From 180 to 300 minutes the on/off aeration control appeared to be functioning as expected. During the first 180 minutes the LDO probe was removed several times, washed, and placed in sodium sulfite solution. Each time the DO would drop rapidly to about 0.1 mg/L. But when the probe was returned to the reactor it exhibited the unexpected high DO response. This was extremely confusing at the time.

In hindsight it appears that the higher NO_2 concentration causes interference with the LDO probe. This problem is confirmed from the results of SDR #1, an unaerated test with a high initial NO_2 concentration. Over the first 240 minutes of the test while NO_2 was present (see Fig. 18) the LDO probe reflected

significant DO levels (see Fig. 19). Once NO_2 essentially was removed, the probe maintained a constant reading of 0.09 mg/L (the probe "zero" as discussed earlier).

As a result of the LDO probe issue, and the non-functioning of the DO controller, the DO in SND #2 wass not maintained at the desired 0.4 mg/L setpoint. Therefore the results from this test do not reflect the desired SND conditions. Nevertheless, the results are presented below.

Keep in mind that DO was zero until at least 180 minutes and thereafter aeration switched on and off apparently to maintain the 0.4 mg/L setpoint. Even after the DO was raised the NH_3 concentration remained near constant and there was no accumulation of NO_2. Apparently no AOB activity was occurring. It can be speculated that the higher NO_2 concentration inhibited the AOB. However, this conclusion is not definitive given the uncertainty over the DO concentration.

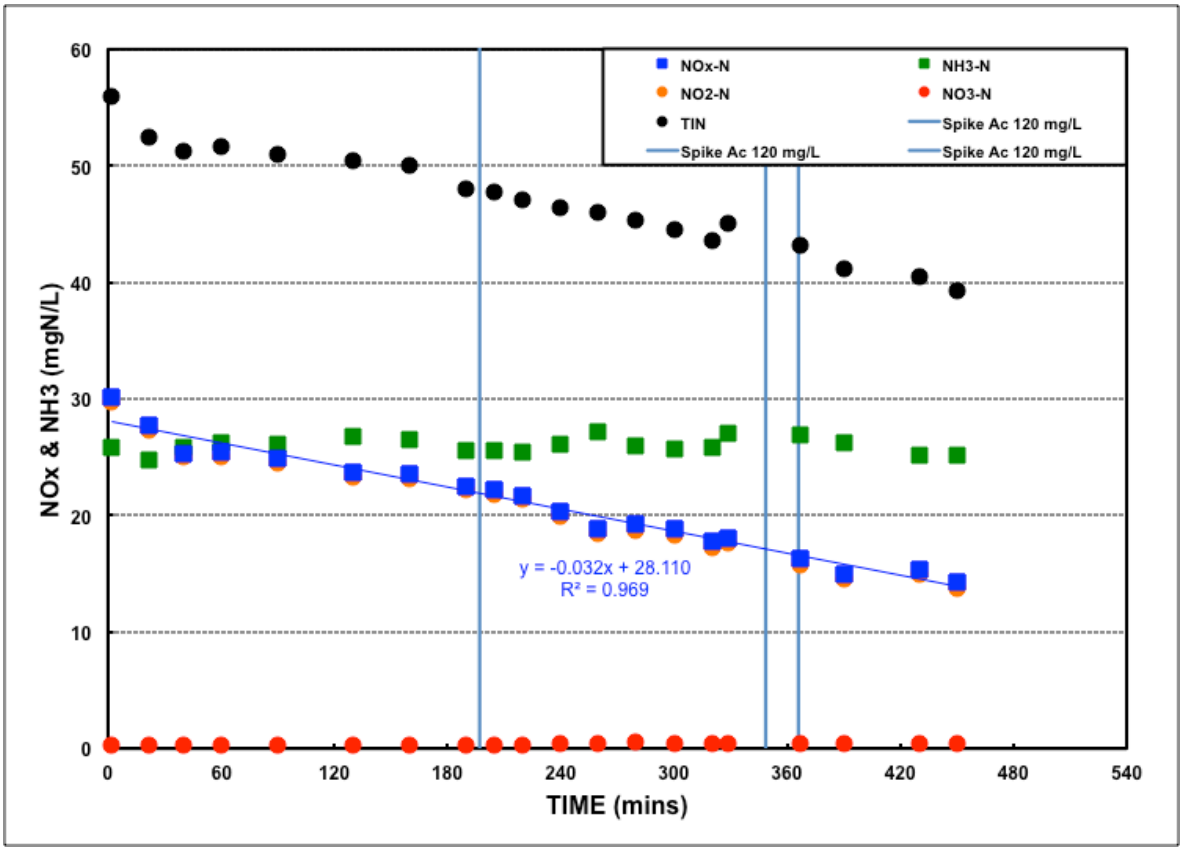

FIGURE 16: AMMONIA AND OXIDIZED NITROGEN CONCENTRATION VERSUS TIME FOR SND TEST #2 (DO CONCENTRATION 0.4 MG/L).

Linear regression analysis is used to estimate the removal rate of NO_X. Dividing this rate by the batch volatile suspended solids concentration yields the specific rate, as shown in Table 9.

TABLE 9: SUMMARY OF RESULTS FOR SND TEST #2

VSS (mg/L)	2300
Average Test Temperature (°C)	23.1
NO_XRR (mgN/L/min)	0.032
SNO_XRR (mgN/gVSS/hr)	0.83
SNO_XRR corrected to 20°C (mgN/gVSS/hr)	0.76

The specific rate was corrected to 20°C using Equation [1] with an Arrhenius value of 1.029.

As shown in Figure 16, a substantial decrease in the TIN (*i.e.* approximately 17 mgN/L) was observed. The NH_3 concentration remained relatively constant whereas the NO_2 concentration decreased 15 mgN/L. This test therefore shows denitritation.

The measured DO concentration in the second SND test is shown in Figure 17.

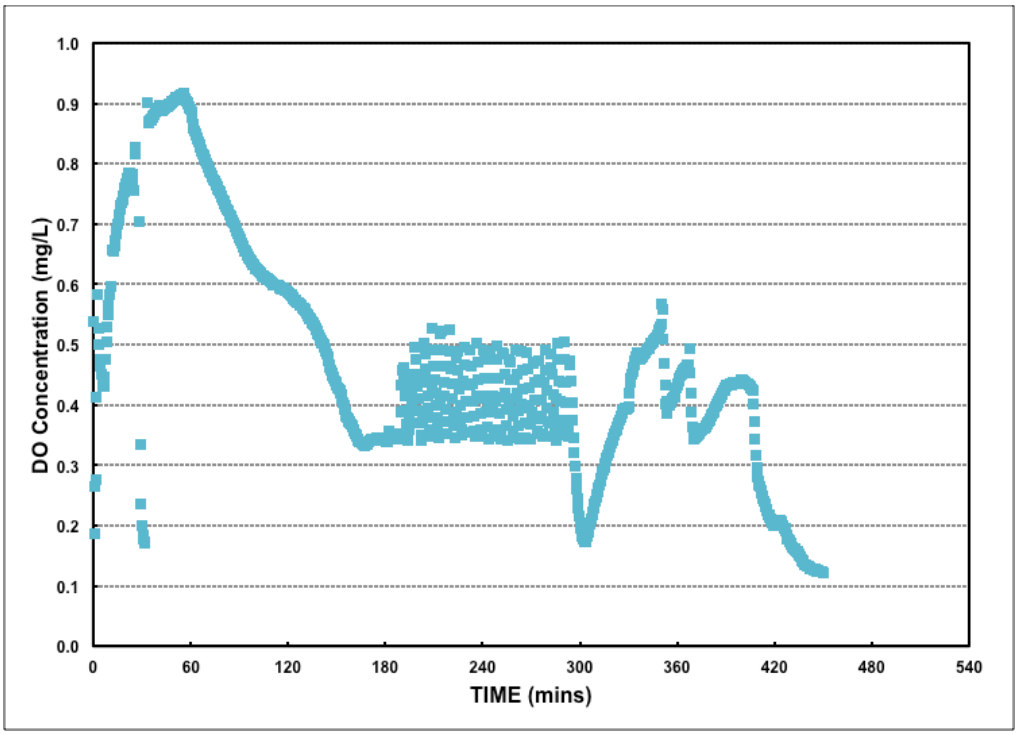

FIGURE 17: MEASURED DO CONCENTRATION IN SND TEST #2.

3.3 SPECIFIC DENITRIFICATION RATE TESTS

Three SDR tests were conducted using acetate as the electron donor and NO_2 or NO_3 as electron acceptors. Each test was seeded with a grab sample of 8 L of RAS from the Southwest WRF. In all three tests, the RAS sample initially contained negligible concentrations of NO_2 and NO_3. At the start of the test, acetate was dosed to an initial concentration of 400 mgCOD/L. Nitrite was dosed at the start of each test. Once NO_2 was depleted, NO_3 was dosed. This allowed for comparison of the SNO_2RR in the absence of NO_3 with the SNO_3RR in the absence of NO_2.

A stand mixer provided mixing. The mixing speed was adjusted so that the liquid was adequately mixed while avoiding the creation of a vortex that could entrain air into the liquid. The liquid surface was covered

with ping-pong balls to further limit the surface transfer of oxygen. The first two SDR tests were unaerated. In order to investigate the effect of low DO concentration on denitrification, the third test was operated at a DO concentration of 0.4 mg/L. Aeration was provided in the third test using aquarium air pumps and air stones. The phosphate concentration was measured throughout each test to ensure nutrient limitations would not impact the test. The results from each test are summarized in the following sections.

3.3.1 SDR #1 (UNAERATED)

In SDR Test #1, NO_2 was dosed at the start of the test to set a target initial concentration of 22 mgN/L. Once the NO_2 was depleted, NO_3 was dosed to set a target concentration of 15 mgN/L. At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Approximately every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for ammonia-, nitrite-, and nitrite-nitrogen.

Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses, as shown in Figure 18.

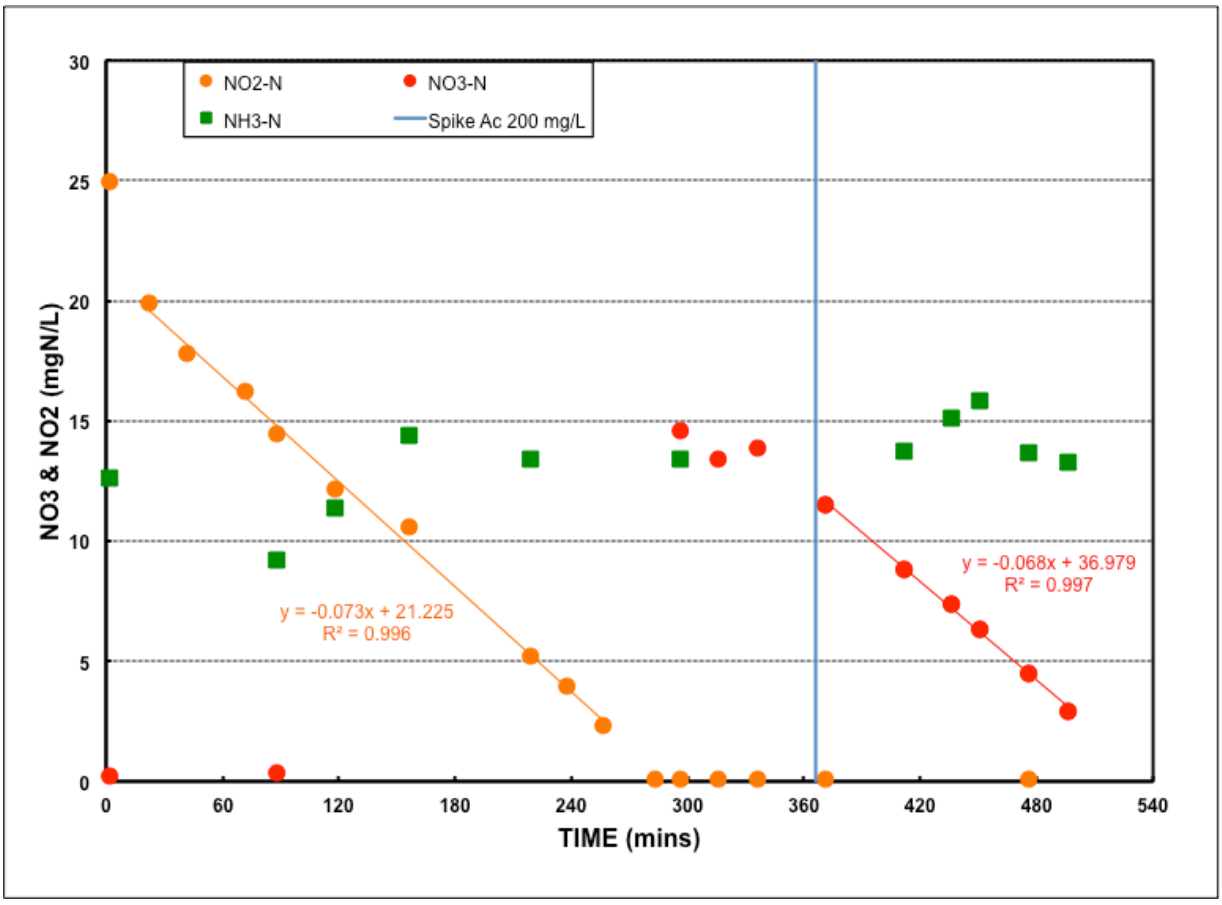

FIGURE 18: NITRATE AND NITRITE REMOVAL VS. TIME IN SDR TEST #1 (UNAERATED).

Linear regression analysis is used to estimate the removal rate of NO_2 and NO_3. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates, as shown in Table 11.

TABLE 11: SUMMARY OF RESULTS FOR SDR TEST #1

VSS (mg/L)	3973
Average Test Temperature (°C)	22.9
NO_3RR in absence of NO_2 (mgN/L/min)	0.068
SNO_3RR in absence of NO_2 (mgN/gVSS/hr)	1.03
SNO_3RR in absence of NO_2 corrected to 20°C (mgN/gVSS/hr)	0.95
NO_2RR in absence of NO_3 (mgN/L/min)	0.073
SNO_2RR in absence of NO_3 (mgN/gVSS/hr)	1.10
SNO_2RR in absence of NO_3 corrected to 20°C (mgN/gVSS/hr)	1.01

The specific rates were corrected to 20°C using Equation [1]. An Arrhenius value of 1.029 was used for the SNO_3RR and SNO_2RR.

The SNO_3RR in the absence of NO_2 was essentially equivalent to the SNO_2RR in the absence of NO_3. However, it was expected that the former rate would be approximately 60% of the latter rate because the oxygen states of NO_3 and NO_2 are +5 and +3, respectively. Hence, per unit COD oxidized, NO_3 is reduced to N_2 gas at a rate that is 3/5 (*i.e.* 60%) the rate that NO_2 is reduced to N_2 gas.

As shown in Table 10, the SNO_3RR and SNO_2RR (corrected to 20°C) were 0.95 and 1.01 mgN/gVSS/hr. Both of these rates are relatively slow. By comparison, in the example SDR test presented in Section 2.3, the SNO_3RR in the absence of NO_2 and SNO_2RR in the absence of NO_3 (both corrected to 20°C) were 7.16 and 11.36 mgN/gVSS/hr, respectively.

In assessing these results, it is pertinent to note that there were problems with the Hach LDO probe. As with SND #2, it appears that interference due to nitrite occurred. Figure 18 shows the decreasing nitrite concentration from the start to approximately 240 minutes. Figure 19 shows the measured DO response. The probe showed significant DO values over this period despite the reactor not being aerated. During this period the LDO probe was removed several times, washed, and placed in sodium sulfite solution. Each time the DO would drop rapidly to about 0.1 mg/L. But when the probe was returned to the reactor it exhibited the unexpected high DO response. This was extremely confusing at the time. However, with hindsight it is evident that presence of nitrite causes the LDO probe to report incorrect data. Therefore the information in Fig. 19 should be ignored; the DO concentration was effectively zero throughout the test.

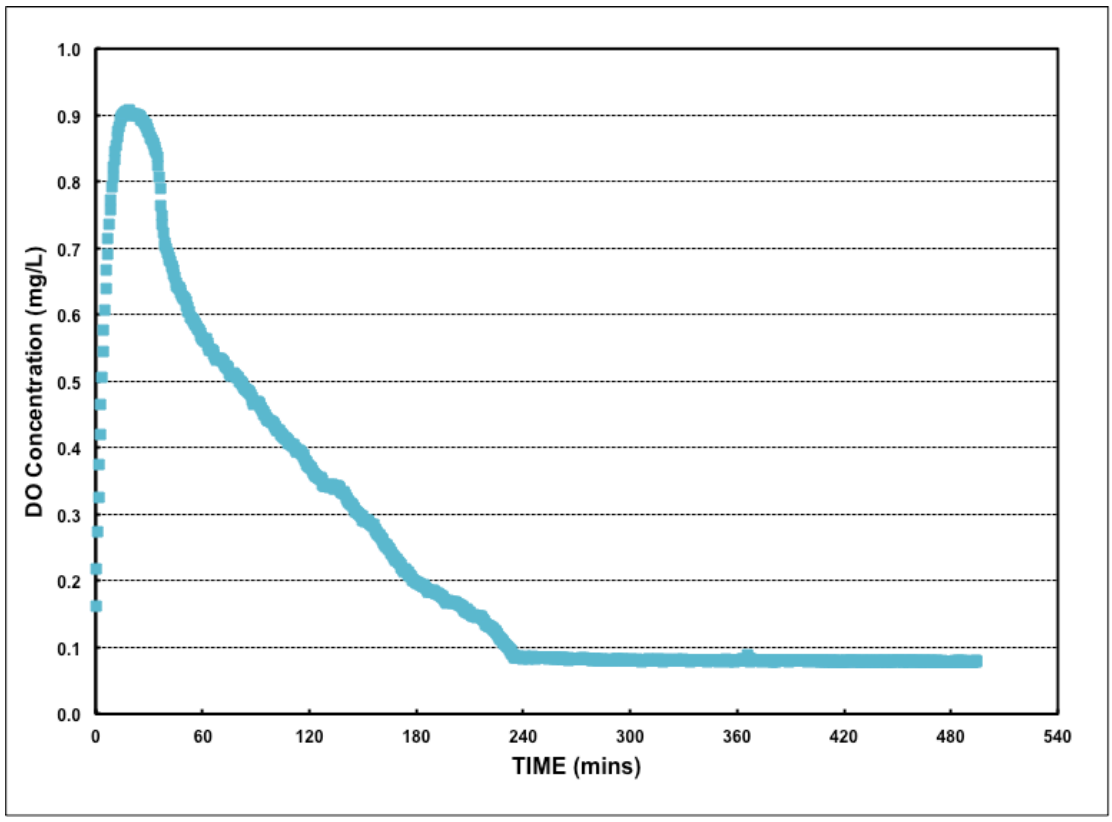

FIGURE 19: MEASURED DO CONCENTRATION IN SDR TEST #1 DEMONSTRATING NITRITE INTERFERENCE.

3.3.2 SDR #2 (UNAERATED)

In SDR Test #2, NO_2 was dosed three separate times during the first half of the test to target concentrations of 10, 10 and 22 mgN/L. Once the final dose of NO_2 was depleted, NO_3 was dosed two separate times to target concentrations of 20 and 22 mgN/L. Multiple doses of NO_2 and NO_3 were required to obtain a sufficient number of data points for the linear regression; the denitrification rates were found to be faster than SDR Test #1. At the beginning of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Approximately every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for ammonia-, nitrite-, and nitrite-nitrogen.

Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses, as shown in Figure 20.

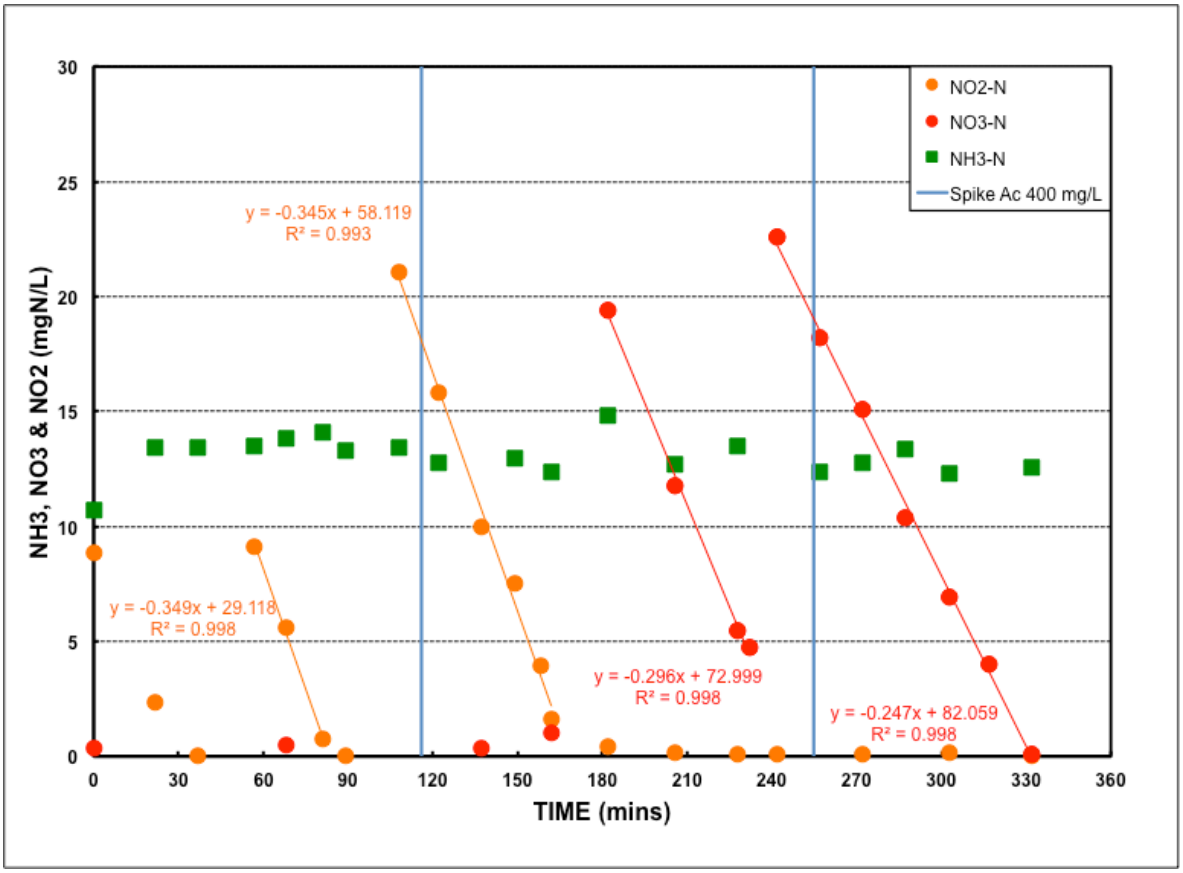

FIGURE 20: NITRATE AND NITRITE REMOVAL VS. TIME IN SDR TEST #2 (UNAERATED).

Linear regression analysis is used to estimate the removal rate of NO_2 and NO_3. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates, as shown in Table 12.

TABLE 12: SUMMARY OF RESULTS FOR SDR TEST #1

VSS (mg/L)	4038
Average Test Temperature (°C)	23.3
NO_3RR in absence of NO_2 (mgN/L/min)	0.247
SNO_3RR in absence of NO_2 (mgN/gVSS/hr)	3.67
SNO_3RR in absence of NO_2 corrected to 20°C (mgN/gVSS/hr)	3.34
NO_2RR in absence of NO_3 (mgN/L/min)	0.345
SNO_2RR in absence of NO_3 (mgN/gVSS/hr)	5.13
SNO_2RR in absence of NO_3 corrected to 20°C (mgN/gVSS/hr)	4.67

The specific rates were corrected to 20°C using Equation [1]. An Arrhenius value of 1.029 was used for the SNO_3RR and SNO_2RR.

The SNO_3RR in the absence of NO_2 was 72% of the SNO_2RR in the absence of NO_3. This is close to the expected value of 60% based on the oxygen states of NO_3 and NO_2. As shown in Table 11, the SNO_3RR

and SNO_2RR (corrected to 20°C) were 3.34 and 4.67 mgN/gVSS/hr, respectively. These rates are considerably faster than the respective rates of 0.95 and 1.01 mgN/gVSS/hr, measured in SDR Test #1.

The DO concentration was measured at the times the samples were taken using the Jenco membrane DO probe and analyzer. The measured DO concentration was at 0.02 mg/L or less throughout the test, indicating that fully anoxic conditions were maintained.

3.3.3 SDR #3 (DO CONCENTRATION 0.4 MG/L)

The presence of low DO in the reactor in SDR Test #1 resulted in lower denitrification rates compared to SDR Test #2 which was truly anoxic. In order to further investigate the effect of low DO on denitrification, SDR Test #3 was operated at a DO setpoint of 0.4 mg/L. This setpoint is close to the upper limit of the DO range at which the aeration basins at the Southwest WRF are operated.

In SDR Test #3, NO_2 was dosed at the start of the test to set a target initial concentration of 10 mgN/L. Because nitrification occurred during the test, NH_3 was removed and hence NH_3 was dosed to a target concentration of 22 mgN/L at 100 minutes into the test. At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Every 20 to 50 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for ammonia-, nitrite-, and nitrite-nitrogen.

Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses, as shown in Figure 21.

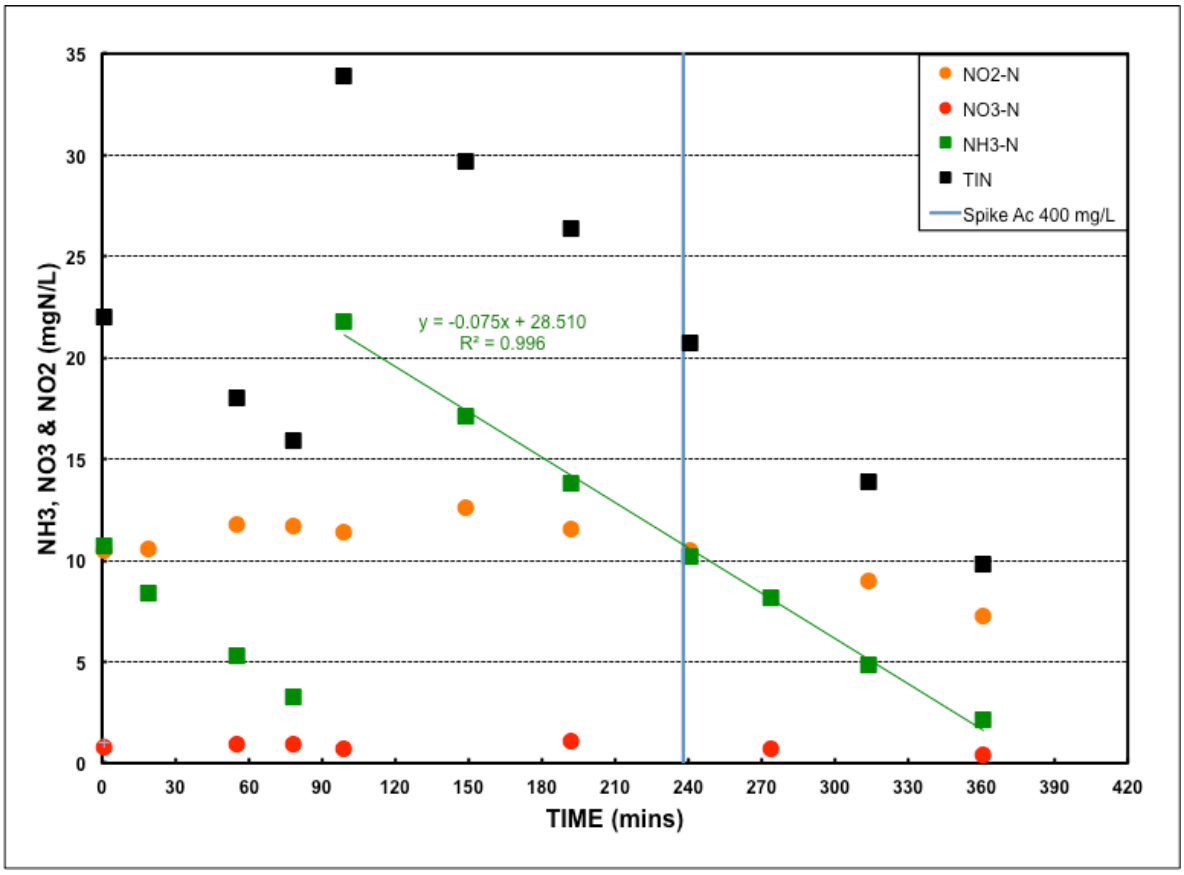

FIGURE 21: AMMONIA, NITRATE, NITRITE AND TIN VS. TIME IN SDR TEST #3 (DO CONCENTRATION 0.4 MG/L).

Linear regression analysis is used to estimate the removal rate of NH_3. Dividing this rate by the batch volatile suspended solids concentration yields the specific rate, as shown in Table 13.

TABLE 13: SUMMARY OF RESULTS FOR SDR TEST #3

VSS (mg/L)	4100
Average Test Temperature (°C)	23.4
NH_3RR (mgN/L/min)	0.075
SNH_3RR (mgN/gVSS/hr)	1.09
SNH_3RR corrected to 20°C (mgN/gVSS/hr)	0.86

The specific rate was corrected to 20°C using Equation [1]. An Arrhenius value of 1.072 was used for the SNH_3RR. As shown in Figure 21, the TIN steadily decreased. [It should be noted that the TIN increased at 100 minutes due to the dose of NH_3 added at that time]. The NH_3 was removed at a temperature-corrected SNH_3RR of 0.86 mgN/gVSS/h while the NO_2 concentration remained relatively constant (*cf.* SNR #3 where SNH3RR was 0.91 at a DO of 0.4 mg/L). This clearly demonstrates the occurrence of the nitrite-shunt process. The dose of acetate added at 240 minutes perhaps improved denitritation as the NO_2 concentration decreased by approximately 3 mgN/L from 240 minutes until the end of the test.

The DO controller was programmed to maintain the DO concentration in the batch test between 0.35 and 0.45 mg/L. As shown in Figure 22, the measured DO concentration actually exceeded this range. However the overall average measured DO concentration during the test was 0.39 mg/L.

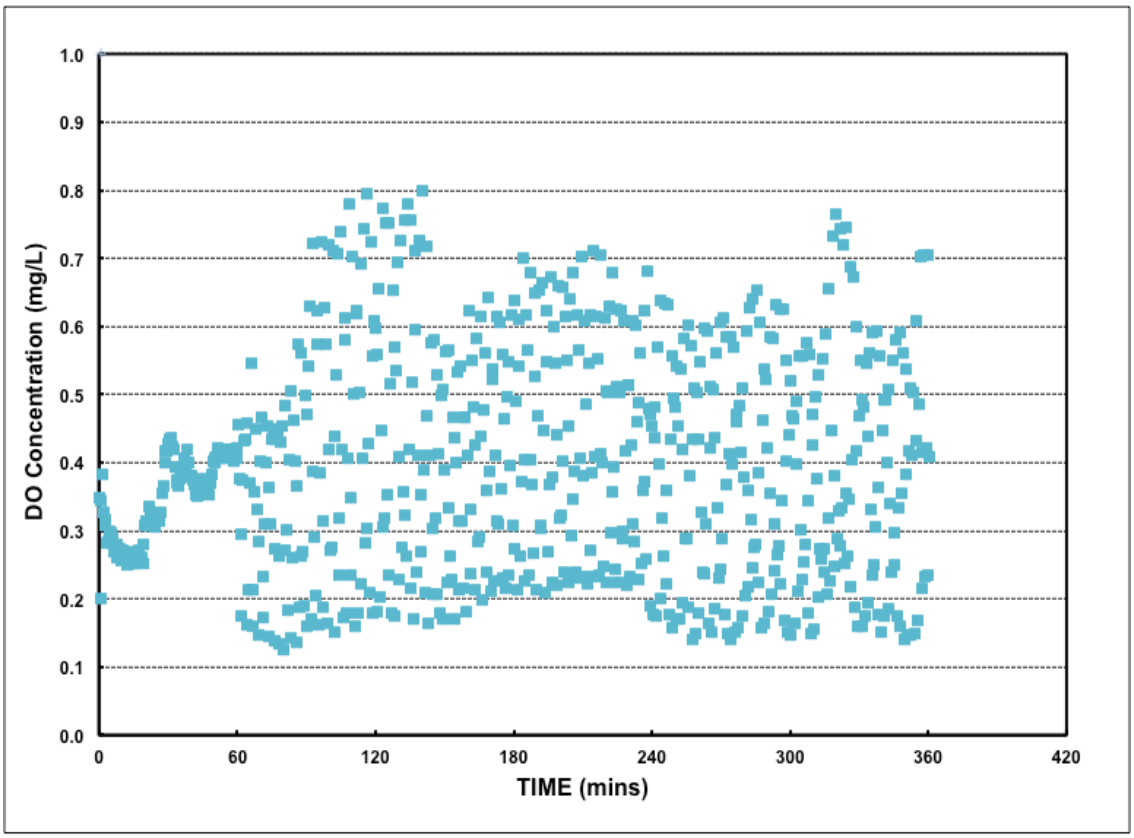

FIGURE 22: MEASURED DO CONCENTRATION IN SDR TEST #3.

As previously mentioned, the reading of 0.09 mg/L on the sc200 controller interface in fact corresponds to "zero" DO. Thus the average measured DO concentration in SNR Test #4 of 0.39 mg/L is in fact 0.30 mg/L.

3.3.4 SUMMARY OF SDR TESTS

Table 14 summarizes the SDR tests conducted.

TABLE 14: SUMMARY OF SDR TESTS CONDUCTED AT SOUTHWEST WRF

SNR #	Avg. DO (mg/L)	Batch Test MLVSS (mg/L)	SNO_2RR in absence of NO_3 (mgN/gVSS/h)	SNO_3RR in absence of NO_2 (mgN/gVSS/h)	SNH_3RR (mgN/gVSS/h)
1	0	3973	1.01	0.95	N/A
2	0	4038	4.67	3.34	N/A
3	0.39	4100	N/A	N/A	0.86

The SDR test data did not conform to other testing that we have conducted at other plants using acetate. The one difference is that those other plants did not incorporate biological P removal. In hindsight, perhaps selecting acetate as substrate was not a good choice. In the case of the Southwest WRF sludge, adding acetate would result in phosphate release and acetate uptake by PAOs. This leaves a question mark over how much acetate was available for denitrification by OHOs.

Nevertheless the SDR tests confirmed that both NO_2 and NO_3 can be denitrified by the Southwest WRF mixed liquor. Also, denitrification occurred when DO was raised to 0.4 mg/L, confirming that nitrite-shunt is happening.

3.4 TEST TO DETECT PRESENCE OF ANAMMOX BACTERIA

A single batch test was carried out to determine whether Anammox bacteria are present in the activated sludge at the Southwest WRF. The test was seeded with a grab sample of 8 L of RAS from the Southwest WRF. Ammonia and NO_2 were dosed to initial target concentrations of 30 and 10 mgN/L, respectively. Because NO_2 was removed during the test, a second dose of NO_2 was added at 56 minutes to a target concentration of 25 mgN/L. A stand mixer provided mixing. The mixing speed was adjusted so that the liquid was adequately mixed while avoiding the creation of a vortex that could entrain air into the liquid. The liquid surface was covered with ping-pong balls to further limit the surface transfer of oxygen.

At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Approximately every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for ammonia-, nitrite-, and nitrite-nitrogen. Plotting the ammonia-, nitrate-, and nitrite-nitrogen data *versus* time results in linear responses, as shown in Figure 23.

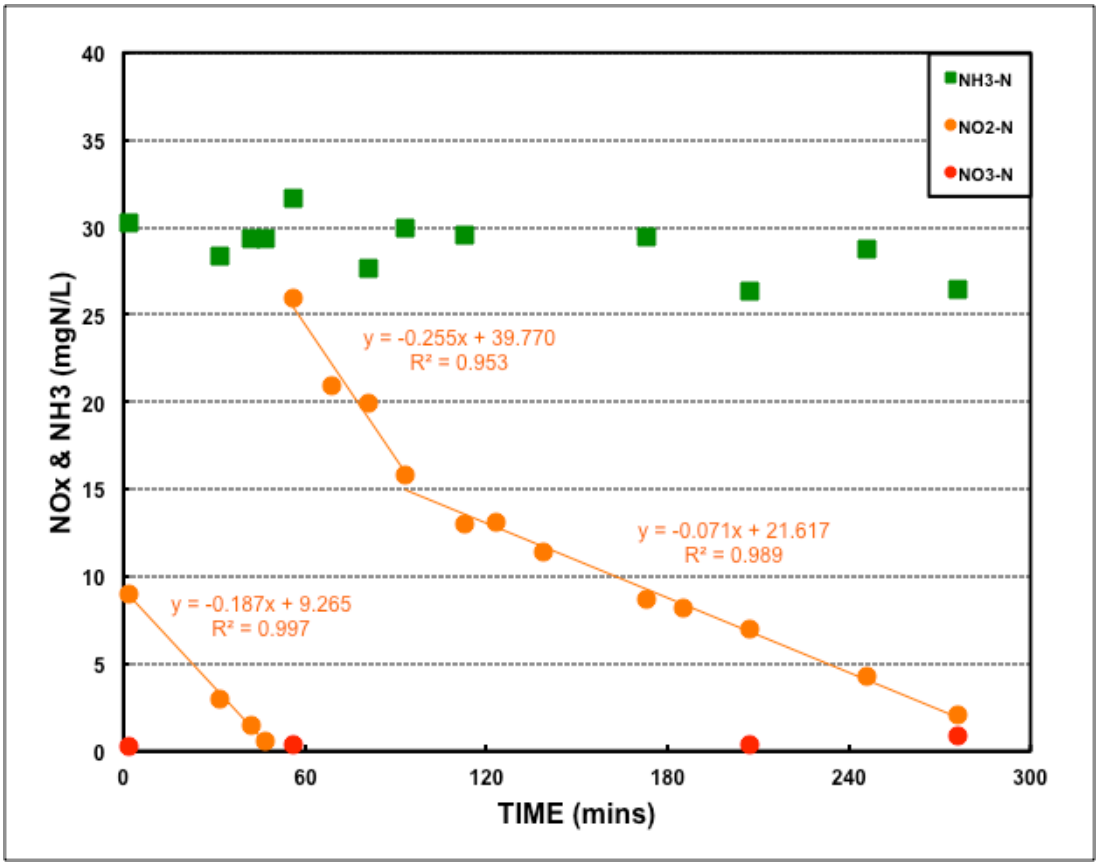

FIGURE 23: AMMONIA, NITRITE AND NITRATE VERSUS TIME FOR ANAMMOX TEST (UNAERATED)

Linear regression analysis is used to estimate the removal rates of NO_2. Dividing these rates by the batch volatile suspended solids concentration yields the specific rates, as shown in Table 15. Over the first 100 minutes of the test the removal rate of NO_2 was very rapid even though there was no external carbon addition. It is likely that a significant amount of hydrolysis and fermentation of COD had occurred in the RAS sample prior to the test, and that the rapid initial rate was due to denitritation with RBCOD and/or acetate.

TABLE 15: SUMMARY OF RESULTS FOR ANAMMOX TEST

VSS (mg/L)	4173
Average Test Temperature (°C)	23.0
NO_2RR "K1" in Absence of NO_3 (mgN/L/min)	0.221*
SNO_2RR "K1" in Absence of NO_3 (mgN/L/hr)	3.18
SNO_2RR "K1" in Absence of NO_3 corrected to 20°C (mgN/gVSS/hr)	2.92
NO_2RR "K2" in Absence of NO_3 (mgN/L/min)	0.071
SNO_2RR "K2" in Absence of NO_3 (mgN/L/hr)	1.03
SNO_2RR "K2" in Absence of NO_3 corrected to 20°C (mgN/gVSS/hr)	0.94

*The NO_2RR value 0.221 mgN/L/min is the average of the two measured slopes, 0.187 and 0.255 mgN/L/min.

The specific rates were corrected to 20°C using Equation [1] with an Arrhenius value of 1.029. It is interesting to note that the rate of NO_2 removal changed after about 90 minutes from a faster "K1" rate (2.92 mgN/gVSS/h) to a slower "K2" rate (0.94 mgN/gVSS/h).

As shown in Figure 23, NH_3 remained relatively constant while NO_2 was steadily removed during the test. This suggests that Anammox bacteria were not present in the sludge, as these bacteria would have removed NH_3 and NO_2 at equal rates.

3.5 PHOSPHORUS RELEASE AND UPTAKE TESTS

Three phosphorus release and uptake tests were conducted during the investigation. In addition, orthophosphate and soluble COD were measured during many of the SNR, SND, SDR and Anammox tests. These results will be described after the results of the three P-release and uptake tests.

Each P-release and uptake test was seeded with a grab sample of 8 L of RAS from the Southwest WRF. Before starting each test, the reactor was aerated using aquarium pumps and air stones for approximately 30 minutes to lower the initial orthophosphate concentration as much as possible. The aeration system was then turned off. The mixing speed of the stand mixer was adjusted so that the liquid was adequately mixed while avoiding the creation of a vortex that could entrain air into the liquid. The liquid surface was covered with ping-pong balls to further limit the surface transfer of oxygen. At the start of the test, acetate was dosed to a target concentration of 200 mgCOD/L (Tests #1 and #2) and 80 mgCOD/L (Test #3).

All three tests were operated under unaerated conditions until the release of phosphate with respect to time reached a plateau. An electron acceptor was then introduced into the reactor. Tests #1 and #2 were aerated at DO setpoints of 5 and 0.4 mg/L; Test #3 was dosed with NO_2 to a target concentration of 25 mgN/L. The results from each test are summarized in the following sections.

3.5.1 P-RELEASE & UPTAKE TEST #1 (FULLY AERATED AFTER P-RELEASE)

At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for orthophosphate and soluble COD. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Approximately every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for orthophosphate and soluble COD. As shown in Figure 24, the release of orthophosphate slowed down after approximately 60 minutes and appeared to plateau by 160 minutes. At this time, the test was then fully aerated at a target DO concentration of 5 mg/L. Aeration was continued until the uptake of phosphate was complete.

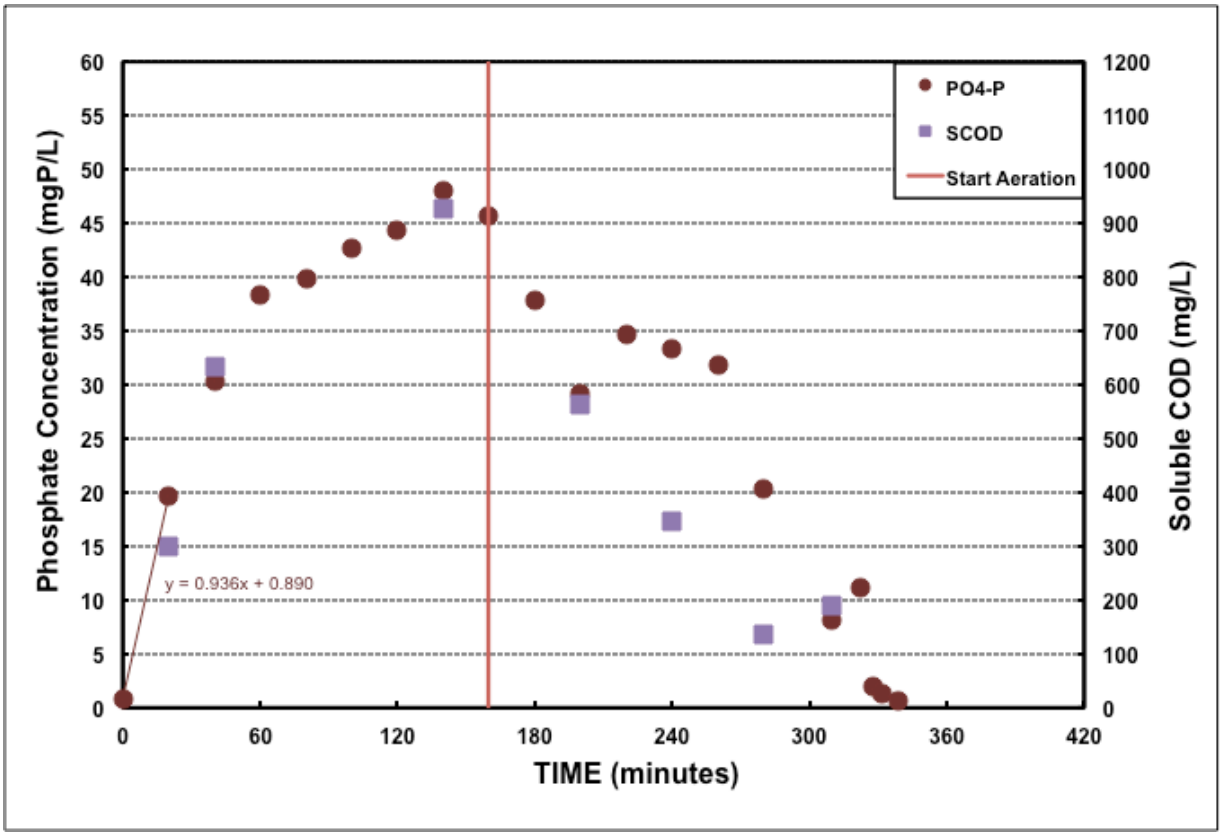

FIGURE 24: PHOSPHATE AND SOLUBLE COD VERSUS TIME FOR P-RELEASE & UPTAKE TEST #1 (FULLY AERATED AFTER P-RELEASE)

Linear regression analysis is used to estimate the maximum P-release rate. Dividing this rate by the batch volatile suspended solids concentration yields the specific rate, as shown in Table 16.

TABLE 16: SUMMARY OF RESULTS FOR P-RELEASE & UPTAKE TEST #1

VSS (mg/L)	4250
Maximum PO_4 Release Rate (mgP/L/min)	0.936
Specific Maximum PO_4 Release Rate (mgP/gVSS/hr)	13.21

As shown in Figure 24, the soluble COD increased during the P-release. This was likely because VFAs were being generated by fermentation at a rate faster than the rate at which acetate and propionate were utilized for P-release. Once aeration commenced, these VFAs were then used by OHOs. The aeration period lasted 179 minutes. During that time, the orthophosphate concentration decreased 46 mgP/L. The P-uptake over the aerobic phase was therefore 46 mgP/L/179 min = 0.26 mgP/L/min. The specific P-uptake rate was therefore 3.67 mgP/gVSS/h.

3.5.2 P-RELEASE & UPTAKE TEST #2 (DO CONCENTRATION 0.4 MG/L AFTER P-RELEASE)

At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for orthophosphate and soluble COD. In addition, a suspended solids analysis was carried out on the solids

retained on each filter paper. Approximately every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for orthophosphate and soluble COD. As shown in Figure 25, the release of orthophosphate slowed down after approximately 80 minutes and appeared to plateau by 160 minutes. At this time, the test was then aerated at a target DO concentration of 0.4 mg/L. Aeration was continued until the uptake of orthophosphate was complete.

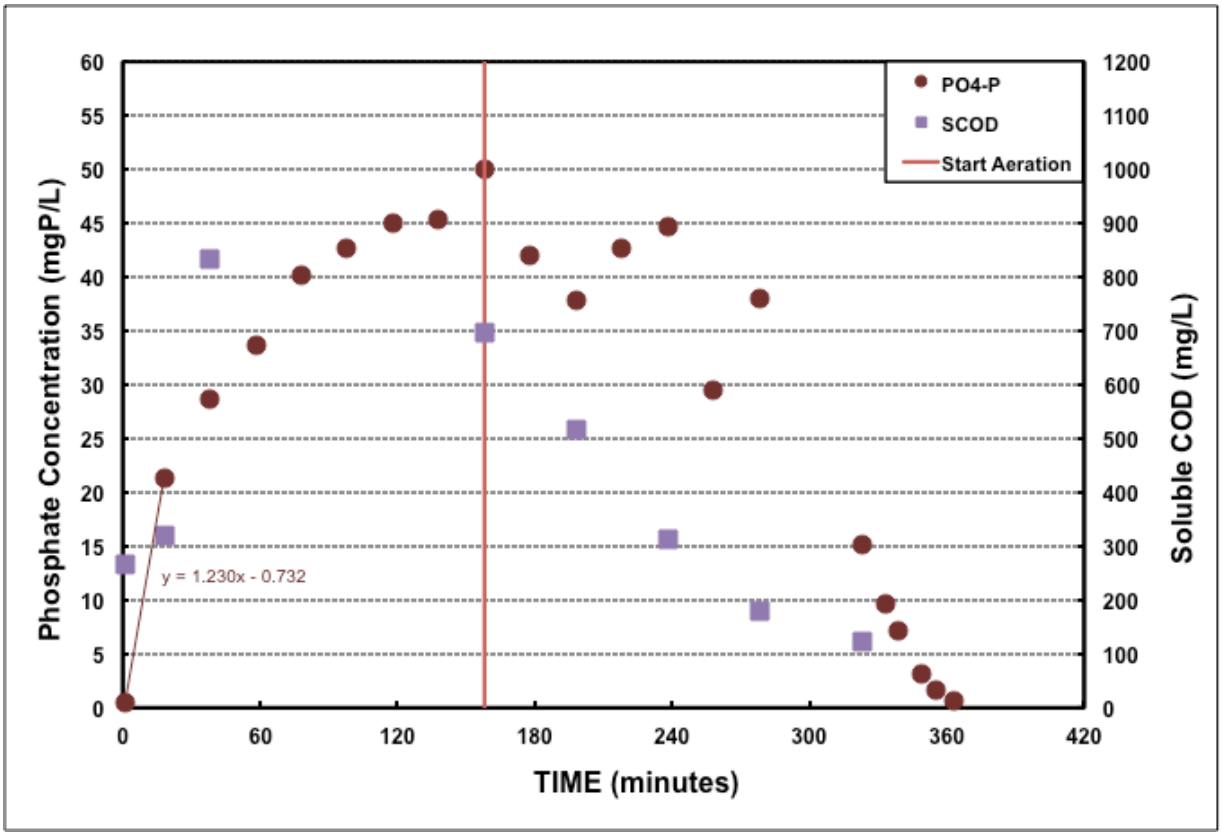

FIGURE 25: PHOSPHATE AND SOLUBLE COD VERSUS TIME FOR P-RELEASE & UPTAKE TEST #2 (DO CONCENTRATION 0.4 MG/L AFTER P-RELEASE)

Linear regression analysis is used to estimate the maximum P-release rate. Dividing this rate by the batch volatile suspended solids concentration yields the specific rate, as shown in Table 17.

TABLE 17: SUMMARY OF RESULTS FOR P-RELEASE & UPTAKE TEST #2

VSS (mg/L)	4370
Maximum PO_4 Release Rate (mgP/L/min)	1.23
Specific Maximum PO_4 Release Rate (mgP/gVSS/hr)	16.89

The specific maximum phosphate release rate in Test #2 was 16.89 mgP/gVSS/hr, which is higher than the rate of 13.21 mgP/gVSS/hr measured in Test #1. Both tests were operated identically during the unaerated period. Similar to the first P-release & uptake test, the soluble COD increased during the P-release in this test.

Once aeration commenced, the orthophosphate concentration remained relatively constant in the reactor for approximately 80 minutes, and then was removed. By comparison, once aeration commenced in Test

#1, phosphate was continually taken up. The aeration period lasted 205 minutes in Test #2. During that time, the orthophosphate concentration decreased 50 mgP/L. The P-uptake was therefore 50 mgP/L/205 min = 0.24 mgP/L/min. The specific P-uptake rate was therefore 3.30 mgP/gVSS/h. This rate is essentially equivalent to the P-uptake rate measured in Test #1 (*i.e.* 3.67 mgP/gVSS/h). This indicates that aerating the sludge at the low DO concentration of 0.4 mg/L does not hinder the uptake of phosphate.

During the aeration period, the DO controller was programmed to maintain the DO concentration between 0.35 and 0.45 mg/L. As shown in Figure 26, the measured DO concentration actually exceeded this range. However the overall average measured DO concentration during the test was 0.37 mg/L.

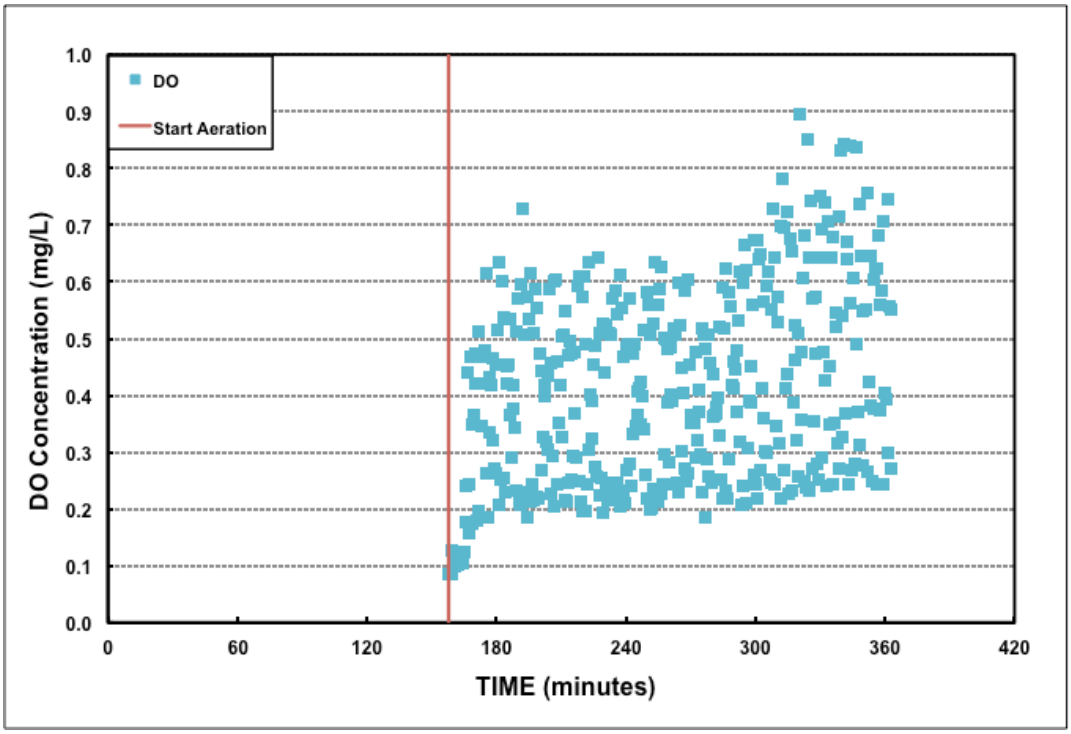

FIGURE 26: MEASURED DO CONCENTRATION IN P-RELEASE & UPTAKE TEST #2.

As previously mentioned, the reading of 0.09 mg/L on the sc200 controller interface in fact corresponds to "zero" DO. Thus the average measured DO concentration in SNR Test #4 of 0.37 mg/L is in fact 0.28 mg/L. This level is well above the K_{DO} of 0.05 mg/L typically assumed for PAOs, which explains why P-uptake proceeded unhindered during the aeration phase of Test #2.

3.5.3 P-RELEASE & UPTAKE TEST #3 (DOSE NO_2 AFTER P-RELEASE)

At the start of the test, two 25 mL aliquots were removed, separately filtered, and analyzed for orthophosphate and soluble COD. In addition, a suspended solids analysis was carried out on the solids retained on each filter paper. Approximately every 20 minutes over the duration of the test, a 15 mL aliquot was removed, filtered and analyzed for orthophosphate and soluble COD. As shown in Figure 27, the release of orthophosphate slowed down after approximately 24 minutes. At 89 minutes, the test was then dosed with NO_2 to a target concentration of 25 mgN/L.

If PAOs in the mixed liquor are able to use NO_2 as an electron acceptor with concomitant uptake of phosphate, then a decline in both phosphate and NO_2 would be anticipated after adding NO_2. However, phosphate was not taken up and in fact continued to be released at a similar rate as during the anaerobic phase. The NO_2 was therefore likely denitrified by the OHOs. Since dosing NO_2 did not result in P-uptake, the batch test was fully aerated at a DO setpoint of 5 mg/L after 308 minutes. Although this

resulted in P-uptake, it proceeded at a very slow rate (*i.e.* 6 mgP/L in 155 min = 0.04 mgP/L/min). Dividing this rate by the measured VSS concentration in the batch test (shown in Table 18) yields a specific P-uptake rate of 0.57 mgP/gVSS/h. This P-uptake rate is much slower than the rates of 3.67 and 3.30 mgP/gVSS/h observed in Tests #1 and #2, respectively. This slower rate was likely due to the fact that the stored PHA in the PAOs was nearly depleted.

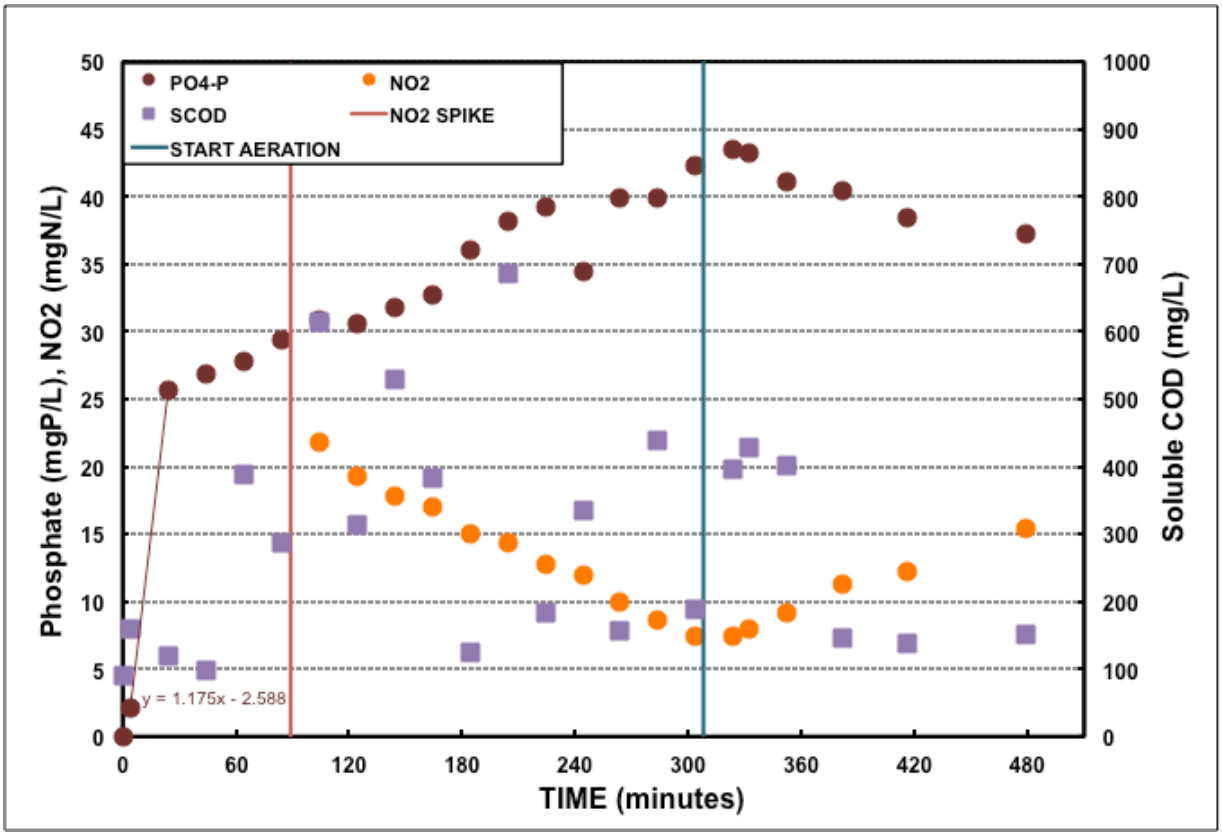

FIGURE 27: PHOSPHATE, NO2 AND SOLUBLE COD VERSUS TIME FOR P-RELEASE & UPTAKE TEST #3 (NO2 DOSED TO 25 MGN/L AFTER P-RELEASE)

Linear regression analysis is used to estimate the maximum P-release rate. Dividing this rate by the batch volatile suspended solids concentration yields the specific rate, as shown in Table 18.

TABLE 18: SUMMARY OF RESULTS FOR P-RELEASE & UPTAKE TEST #3

VSS (mg/L)	4237
Maximum PO_4 Release Rate (mgP/L/min)	1.175
Specific Maximum PO_4 Release Rate (mgP/gVSS/hr)	16.64

The specific maximum phosphate release rate in Test #3 was 16.64 mgP/gVSS/hr, which is essentially equivalent to the rate of 16.89 mgP/gVSS/hr measured in Test #2.

3.5.11 SUMMARY OF P-RELEASE & UPTAKE TESTS

Table 27 summarizes the P-release and uptake tests conducted.

TABLE 27: SUMMARY OF P-RELEASE & UPTAKE TESTS CONDUCTED AT SOUTHWEST WRF

Test #	Avg. DO (mg/L)	Max P-Uptake Rate (mgP/gVSS/h)	Max P-Release Rate (Ac) (mgP/gVSS/h)	Max P-Release Rate (Inf) (mgP/gVSS/h)
P 1	0 then 5	3.67	13.21	N/A
P 2	0 then 0.4	3.30	16.89	N/A
P 3	0 then 0	N/A	16.64	N/A

In hindsight, perhaps more definitive P release and uptake patterns would have been observed had mixed liquor drawn at the end of the plant aeration basin been used instead of RAS. Evidently a significant amount of hydrolysis and fermentation is occurring in the sludge blanket of the clarifiers. As a consequence, the RAS already contained soluble COD and continued to generate soluble COD over the first phase of the tests.

4.0 CONCLUSIONS

The St. Petersburg Southwest WRF is performing exceptionally well using a simple process configuration with no mixed liquor recycle. The key factors of this simple configuration include a large anaerobic zone (*i.e.* the unaerated zone is 25% of the total reactor volume) and accurate DO control at low DO levels (*i.e.* over a DO concentration range of 0.1 to 0.4 mg/L) to achieve a significant degree of nitrite-shunt. Operating at such a low DO concentration substantially reduces the aeration costs at the plant. The Southwest WRF is only required to meet an effluent NO_3 limit; however, it is achieving a TIN of approximately 3.5 mgN/L and a very low effluent orthophosphate concentration of around 0.1 mgP/L. Based on the results of the P-release and uptake tests, biological phosphorus removal performance at the Southwest WRF is very good, despite the low DO concentrations in the aerated reactors.

5.0 RECOMMENDATIONS

The St. Petersburg Southwest WRF is achieving excellent effluent quality using a simple process configuration. In the previous section the factors contributing to the successful performance of the Southwest WRF were identified. Since the plant is operating so well, it may be beneficial to apply the design and operating mode of the Southwest WRF at other plants. However, the following factors must first be considered:

- The St. Petersburg Southwest WRF treats raw influent. Many other plants incorporate primary treatment, which will reduce the amount of carbon available for denitrification. As a result, it may not always be possible to achieve such low effluent NO_2 and NO_3 concentrations.
- The average temperature of the RAS sampled from the Southwest WRF during this investigation was 23.5°C. At lower temperatures the NH_3 removal by the AOB may be reduced, especially since the reactors are operated at very low DO levels to promote denitrification. It may be possible to compensate for temperature-reduced growth rates by increasing the SRT of the plant.
- It should be determined whether the Southwest WRF continues to operate well at high summer temperatures or whether the plant operation should be modified. It has been shown that the activity of PAOs is reduced at higher temperatures (around 30°C) ostensibly due to GAO

proliferation. If phosphorus removal does not deteriorate of the Southwest WRF when mixed liquor temperatures rise to 30°C, then perhaps it can be speculated that low DO operation constrains GAO growth.

- The importance of SRT to the success of the plant operation should be better established. The high temperature at the Southwest WRF allows shorter SRT. In turn, shorter SRT means more N to WAS sludge, so less N to be converted to NO_X and less NO_X to be denitrified.
- It should be further investigated whether the DO levels can be better optimized at the Southwest WRF to reduce the effluent NH_3 concentration closer to 1 mgN/L. This could perhaps be achieved by elevating the DO slightly either at the beginning or the end of the aeration train. Measurements should be carried out to determine the NH_3 profile data along the length of the tank.
- Is would be interesting to know whether it possible to reduce the volume fraction of the anaerobic zone from 25% to 15% without compromising the biological phosphorus removal process.

6.0 REFERENCES

WERF (Water Environment Research Foundation) (2003) ***Methods for wastewater characterization in activated sludge modeling.*** Project 99-WWF-3, ISBN 1-893664-71-6. Alexandria, Virginia.

Bye, C.M., Jones, R.M., and Dold, P.L. ***Pragmatic nitrification inhibition testing for robust plant design.*** Water Environment Federation 85th Annual Conference and Exposition, New Orleans, LA, USA, September 29 - October 3, 2012.

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Appendix C: Aeration Calculations

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	A	B	C	D	E	F	G	H
8	Parameter	Unit	Membrane				Total	Comments
9	Zone Number		**Aer-1A**	**Aer-1B**	**Aer-1C**	**Post-Aer**		
10	Zone Volume	Mgal	1.140	1.140	0.570	0.570	3.420	
14	Avg AOR	kgO2/d	7,590	4,033	1,130	1,019	13,772	
15	Max AOR	kgO2/d	10,663	5,619	1,643	1,566	19,491	
16	Min AOR	kgO2/d	6,346	2,692	593	478	10,109	
17	Alpha		0.45	0.50	0.55	0.55		Sanitaire 9" membrane diffusers
18	Altitude	m	61	61	61	61		Approx 200 ft
19	Water Temp	°C	22.7	22.7	22.7	22.7		Max temp from 90d itinerary
20	Air Temp	°C	32.0	32.0	32.0	32.0		Assumption
21	Avg DO conc.	mg/l	0.4	0.2	0.1	0.1		Values from Modeling
22	Max DO conc.	mg/l	0.4	0.2	0.1	0.1		
23	Min DO conc.	mg/l	0.4	0.2	0.1	0.1		
24	Liquid Depth in Tank	ft	15.3	15.3	15.3	15.3		depth of reactors
25	Diffusers' Distance from Bottom	ft	1.00	1.00	1.00	1.00		Approx. 1 ft less
26	Submergence Depth	m	4.36	4.36	4.36	4.36		
27	Avg Submergence Depth	ft	14.30					
28	O2 in air	kgO2/kg air	0.23	0.23	0.23	0.23		
29	Air density	kg air/m3	1.30	1.30	1.30	1.30		
30	Rel Humidity		0.50	0.50	0.50	0.50		
31	Beta		0.95	0.95	0.95	0.95		
32	Theta		1.02	1.02	1.02	1.02		
33	Barom. Press.	mm Hg	755.1	755.1	755.1	755.1		
34	Blower Dischg	mm Hg	1,150	1,150	1,150	1,150		
35	SWVP	mm Hg	20.50	20.50	20.50	20.50		
36	Avg AOR / SOR		0.412	0.468	0.521	0.521		
37	Max AOR / SOR		0.412	0.468	0.521	0.521		
38	Min AOR / SOR		0.412	0.468	0.521	0.521		
39	Avg SOTE		0.306	0.299	0.288	0.293		
40	Max SOTE		0.290	0.282	0.260	0.265		
41	Min SOTE		0.311	0.313	0.314	0.319		
42	Avg Calculated SOTE		0.306	0.299	0.288	0.293	0.297	
43	Max Calculated SOTE		0.290	0.282	0.260	0.265		
44	Min Calculated SOTE		0.311	0.313	0.314	0.319		
48	Avg Air Flow (US Units)	scfm	5,300	2,529	662	586	9,077	
49	Maximum Air Flow (US Units)	scfm	7,833	3,743	1,065	999	13,640	
50	Minimum Air Flow (US Units)	scfm	4,348	1,612	319	253	6,532	
54	Avg Inlet Air Flow	cfm	5,648	2,695	706	625	9,674	
55	Max Inlet Air Flow	cfm	8,348	3,989	1,135	1,064	14,537	
56	Min Inlet Air Flow	cfm	4,634	1,718	340	270	6,962	
57	Diffusers	No.	5,000	2,000	400	400	7,800	
58	Avg Air Flow per Diffuser	scfm	1.06	1.26	1.66	1.47		< 1.35 scfm
59	Max Air Flow per Diffuser	scfm	1.57	1.87	2.66	2.50		< 3.75 scfm
60	Min Air Flow per Diffuser	scfm	0.87	0.81	0.80	0.63		> 0.5
61	Diffuser Code		2	2	2	2		
62	Diffuser Type Factor		0.1000	0.1000	0.1000	0.1000		
63	Rel.Diff.Area, At/Ad (9"Diffs.)		5.20	13.00	32.49	32.49	20.79	4.5 < At/Ad < 20
64	Diffuser density (% floor)		19.2	7.7	3.1	3.1		
65	Avg. Mixing Air Flow		0.50	0.24	0.12	0.11		> 0.12 scfm/ft^2 for fine pore systems without primaries; > 0.05 to 0.08 scfm/ft^2 for systems with primaries
66	Number Basins		1	1	1	1		
67	Blower Efficiency		0.80					
68	Coupling Efficiency		0.95					
69	Motor Efficiency		0.95					
70	Avg Power	kW	165	79	21	18	282	Wire to water
71	Max Power	kW	243	116	33	31	424	Wire to water
72	Min Power	kW	135	50	10	8	203	Wire to water
73	Avg Power.	hp	221	105	28	24	378	Wire to water
74	Max Power	hp	326	156	44	42	568	Wire to water
75	Min Power	hp	181	67	13	11	272	Wire to water
76								

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