OWASA

Mason Farm Wastewater Treatment Plant Hydraulic and Treatment Capacity Study

Orange Water and Sewer Authority



November 2010

HAZEN AND SAWYER
Environmental Engineers & Scientists

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Executive Summary

The Orange Water and Sewer Authority's (OWASA) Mason Farm Wastewater Treatment Plant (Mason Farm WWTP), an advanced treatment facility that discharges to Morgan Creek, will be required to meet stringent effluent limits required for all major dischargers to the Jordan Lake watershed. The Mason Farm WWTP is currently permitted to discharge up to 14.5 million gallons per day (mgd) on a maximum month basis. The expansion to 14.5 mgd addressed Jordan Lake limits anticipated at the time; however, the final rules require the Mason Farm WWTP to meet more stringent annual mass load allocations of 134,375 pounds per year and 10,188 pounds per year for total nitrogen and total phosphorus, respectively.



In light of the additional limitations on discharge of nutrients, OWASA engaged Hazen and Sawyer to perform a Hydraulic and Treatment Capacity Study for the existing facilities. The goal of this study is to determine the treatment and hydraulic capacity of existing facilities in order to identify process deficiencies that will hinder the plant's compliance with limits under the Jordan Lake Rules. Hazen and Sawyer then evaluated nutrient removal optimization alternatives, aeration capacity alternatives, secondary treatment capacity expansion alternatives, recycle stream treatment alternatives, and chemical feed optimization in order to develop recommendations for plant improvements.

Results of the liquid and solid stream facilities evaluations indicated the need for the following list of recommended plant improvements:

- Full scale pilot testing of nitrogen and phosphorus optimization recommendations

- Convert existing secondary treatment process to provide for step feed of primary effluent flow to the first and second cells of each aeration basin when operated in the 3:4 configuration
- Construct anaerobic digester heat exchanger improvements
- Conduct rotary press stress testing
- Provide chemical phosphorus removal feed point at filters
- Install on-line nutrient analyzers and integrate with plant control system
- Modify odor control design to accommodate step feed configuration
- Modify existing aeration system to include fine bubble diffused aeration and vertical mixers
- Provide new blowers compatible with fine bubble diffuser system
- Construct supplemental carbon feed facilities
- Consideration of filtrate flow equalization
- Consideration of filtrate phosphorus recovery facilities.

The estimated cost of recommended interim improvements to the Mason Farm WWTP is approximately $8.3 million over the next ten years.

The next logical expansion of the Mason Farm WWTP is to 18.5 mgd. This expansion is expected around 2030 based on current wastewater strength and predicted growth. Due to site constraints, it is expected that 18.5 mgd will be the build out capacity of the facility. Facilities preliminarily identified as part of the 18.5 mgd expansion include:

- Various hydraulic improvements
- New Primary Effluent Pump Station
- Anaerobic/Anoxic Tanks
- Modifications to existing Nutrified Sludge Tanks
- NRCY Pump Station
- New Secondary Clarifier #6
- Two New Denitrification Filter Cells
- 1 New UV Channel
- Additional thickening capacity
- Additional anaerobic digestion capacity (two digesters, each with a minimum volume of 510,000 gal)

- Emergency power generation
- Morgan Creek bypass improvements
- Construction of filtrate nitritation/denitritation facilities depending on future dewatering operations
- Construction of filtrate phosphorus recovery facilities depending on future dewatering operations

The estimated cost of capital costs to expand the Mason Farm WWTP to a capacity of 18.5 mgd is approximately $60 million (January 2010 dollars).

Section 1.0
Introduction and Background

1.1 Purpose of the Study

The purpose of this Hydraulic and Treatment Capacity Study is to evaluate the existing facilities at the Orange Water and Sewer Authority's (OWASA) Mason Farm Wastewater Treatment Plant (Mason Farm WWTP) in light of proposed limits resulting from the Jordan Lake Rules.

The Mason Farm WWTP is an advanced treatment facility which discharges treated effluent to Morgan Creek, a tributary to Jordan Lake and the Cape Fear River. The most recent expansion to the Mason Farm WWTP increased the permitted maximum month capacity of the plant to 14.5 million gallons per day (mgd). The design also provided for a rated hydraulic peak capacity of 43.5 mgd with all facilities in service. As part of the expansion project, the plant was also upgraded to address anticipated nutrient limits associated with the Jordan Lake Nutrient Strategy. The final Jordan Lake rules, which were more stringent than the anticipated rules, require that the Mason Farm WWTP meet effluent annual mass limits for Total Nitrogen (TN) and Total Phosphorus (TP) equivalent to performance at concentrations of 3.0 mg/l and 0.23 mg/l respectively, at the 14.5 mgd permitted capacity.

1.2 History of the Mason Farm WWTP

The Mason Farm WWTP, located on Old Mason Farm Road near UNC's Finley Golf Course, was originally constructed in 1948 as a 2.25 mgd trickling filter plant. The facility was upgraded in 1968 to 4.5 mgd and again in 1978 to 5.5 mgd. In 1978, conventional activated sludge basins were constructed for ammonia-nitrogen removal. Chlorine contact basins for effluent disinfection were also constructed at that time. The plant expanded to 8 mgd in 1983 with the addition of more activated sludge basins, final clarifiers, and a new chlorine contact basin, among other improvements. Further improvements were made in 1991 to meet phosphorus effluent limits and in 1994 to add liquid biosolids storage off-site, a project that allowed rerating of the plant from 8 mgd to 9 mgd. An additional 3 mgd of treatment capacity was added as part of a Financial Year 2000 expansion and renovation project. Another major project completed in 2007 expanded the plant's capacity to 14.5 mgd.

1.3 Hydraulic and Treatment Capacity Study Timeline

Hazen and Sawyer began evaluation of the Mason Farm WWTP in July 2009. Findings of the study were presented to OWASA in the form of technical memoranda which were then used to develop this report. Workshops were held to discuss important findings and gain input and direction from OWASA. Primary tasks, in order of completion, included:

- Development of uncalibrated process model in BioWin
- Hazen and Sawyer special sampling
- Hazen and Sawyer secondary clarifier stress testing
- Field survey of hydraulic control points
- Review and development of flow projections
- Hydraulic capacity evaluation using surveyed elevations
- Influent wastewater characterization based on historical and special sampling data
- Calibration of biological process model with historical data
- Evaluate existing unit process capacity for liquid and solids stream facilities
- Develop and evaluate scenarios that should be further evaluated for implementation in order to help the Mason Farm WWTP meet its nutrient mass load allocations
- Present study findings and recommendations

Section 2.0
Flow and Load Development

2.1 Flow and Load Evaluation

This section describes the results of evaluations of influent flows and wastewater loads at the Mason Farm WWTP. Historical data from January 2004 through December 2009 were used to characterize the Mason Farm WWTP influent and to develop the design flows and loads, which serve as the basis for the process evaluation. A detailed historical data analysis is provided in Appendix A.

2.2 Flow Projections

OWASA updated its Long-Range Water Supply Plan. The final Water Supply Plan Update dated April 8, 2010 was provided to Hazen and Sawyer for use in determining future wastewater flows. The projected water demands and wastewater flows were updated in this report. Hazen and Sawyer has reviewed this body of work and concurs with its conclusions regarding projected growth in wastewater flows. A summary of those findings are included in Appendix B. The draft plan includes water demand projections through 2060. Three demand projections are provided in the supply plan including the expected demand and adjusted projections assuming higher and lower growth scenarios. Table 2-1 summarizes the increase in demand from various customer types used to develop the expected future demand. Assumptions for each demand projection include:

<u>Expected Demand</u>

- No changes to OWASA's service area.
- Rate of development will increase to a rate of 563 new meter equivalents (MEs) per year within the next two to three years.
- The rate of development will decrease to a rate of 521 new MEs per year from 2029 through 2060 based on build-out of UNC Central Campus and UNC Hospitals by 2028.
- Future consumption rates are based on average rates observed from 2004 through

2007

- Raw water demand = 114% of total billed water
- OWASA Water Treatment Plant (WTP) process water will continue to be recycled.
- UNC Central Campus and UNC Hospitals development will increase linearly through 2028. No further growth is anticipated after 2028.
- Carolina North will develop at a constant rate from 2014 through 2060.
- 25% of Carolina North's water needs will be met through reclaimed/non-potable water sources.
- Reclaimed water demand at UNC Central Campus and UNC Hospitals will grow through 2028.

Lower Demand

- 15% less residential development through 2060
- 15% less non-UNC non-residential development through 2060
- 50% reclaimed/non-potable water use at Carolina North
- Water conservation will reduce existing and new water demands by the following:
    - All existing non-UNC customer demands decrease 0.10%/year, or 5% through 2060
    - All new non-UNC customer demands decrease 0.25 %/year, or 12% through 2060
    - All UNC customer demands decrease 0.15 %/year, or 7% through 2060

Higher Demand

- One new high density mixed use development (equivalent to approximately 150 single family homes) added per year through 2060.
- Additional non-residential, non-UNC growth is 25% greater than expected through 2060.
- UNC Central Campus and UNC Hospitals growth is 25% greater than expected through 2028.
- Carolina North demand is 25% higher than expected (assumes no reclaimed/non-potable water use).

Table 2-1
OWASA Long-Range Water Supply Plan Update - Projected Water Demand

	% 2008 Demand	New MEs/yr through 2028	New MEs/yr 2028 - 2060	Gal/month /ME	% of New Demand
Single Family Detached	34%	120	120	4,700	12%
Townhouse/Condos	5%	240	240	3,400	20%
Multifamily (master metered)	19%	68	68	14,400	24%
Non-Residential (non-UNC)	13%	61	61	9,900	15%
UNC Central Campus	24%	35	0	21,500	7%
UNC Hospitals	5%	8	0	37,900	3%
Carolina North	0%	31	31	23,300	17%
Totals	**100%**	**563**	**521**	**8,400**	**100%**

Projected potable and reclaimed water demands were used to develop projected annual average WWTP flows based on the following formula:

> Wastewater Flow = 1.14 x 0.924 x (Potable Demand (billed) + Reclaimed Water Flow)

The above equation is based on the average annual ratio of WWTP flow to finished water production of 0.924 between 1995 and 2008. Figure 2-1 provides a graph of the annual ratio through this period. A factor of 1.14 was applied to account for unbilled water to the collection system. Figure 2-2 presents the projected annual average WWTP flows associated with the expected, lower and higher potable water demand projections through 2060. Table 2-2 summarizes the projected WWTP flows.

Figure 2-1
WWTP Flow to WTP Finished Water Production Ratio – 1995-2008

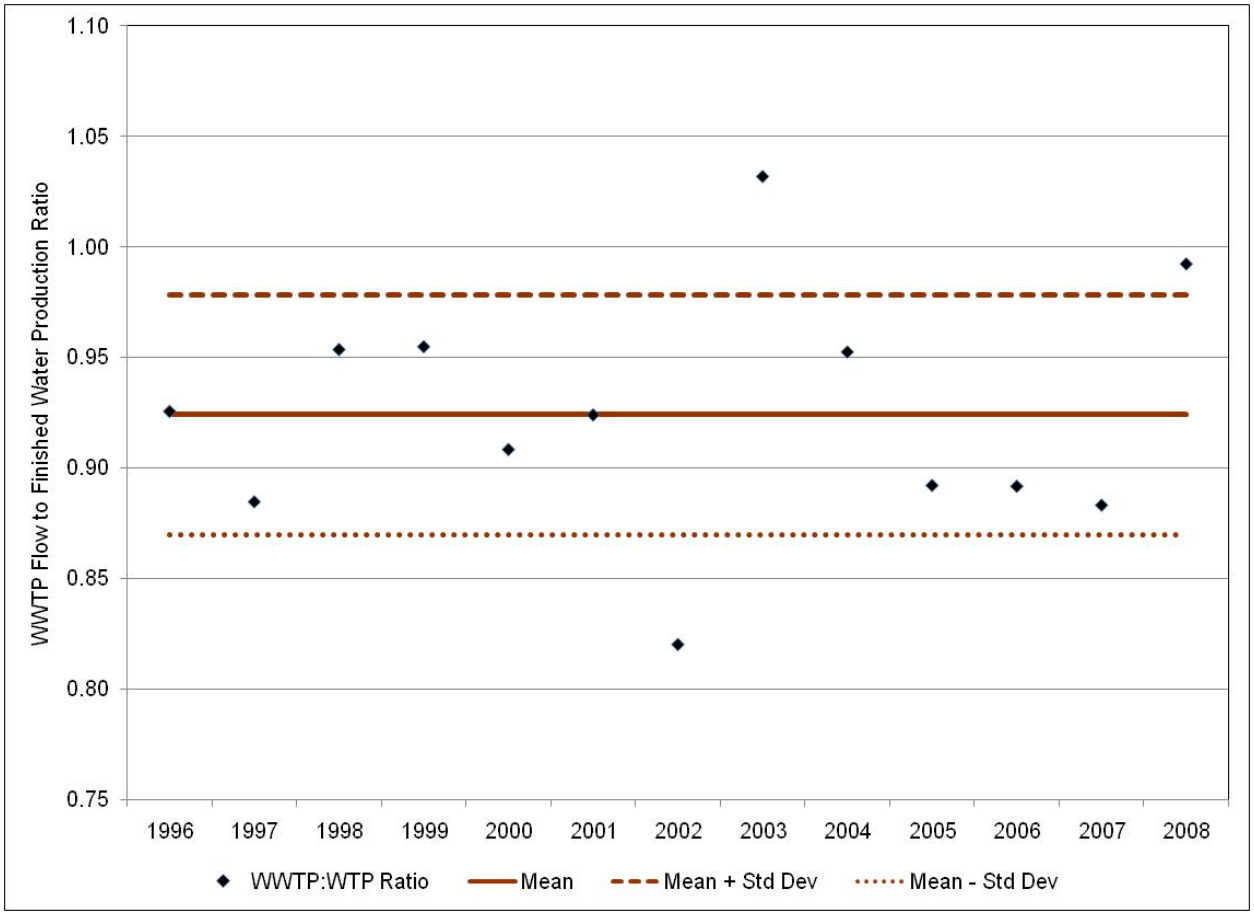


Figure 2-2
WWTP Flow Projections from the OWASA Long-Range Supply Plan Update (Includes November 17, 2009 Adjustments to FY '10 Billed and Unbilled Water Assumptions)

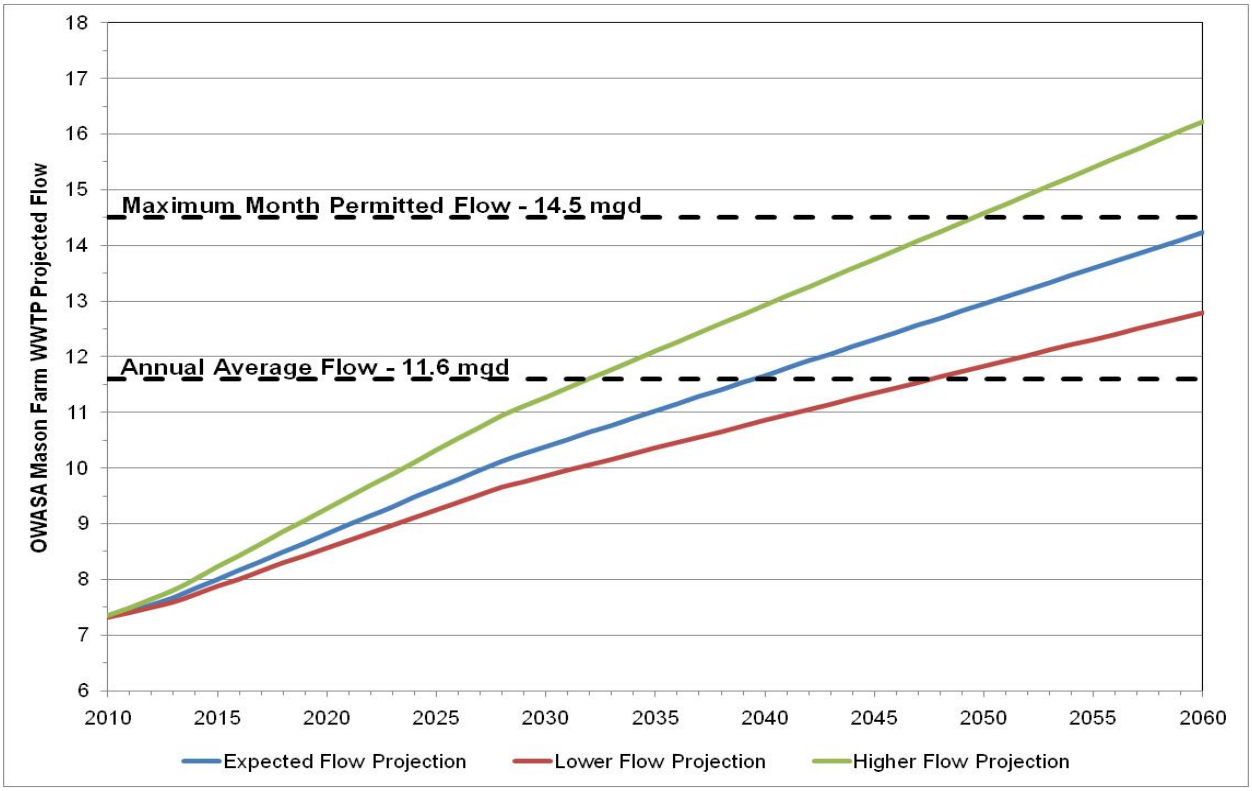


Table 2-2
Projected Annual Average WWTP Flows from the OWASA Long-Range Supply Plan Update (Includes November 17, 2009 Adjustments to FY '10 Billed and Unbilled Water Assumptions)

Year	Expected Flow (mgd)	Lower Flow (mgd)	Higher Flow (mgd)
2010	7.33	7.32	7.36
2020	8.82	8.57	9.27
2030	10.39	9.87	11.28
2040	11.67	10.86	12.93
2050	12.95	11.84	14.58
2060	14.24	12.80	16.23

The Mason Farm WWTP is currently rated to treat a maximum monthly flow (MMF) of 14.5 mgd. Based on the design maximum month peak factor of 1.25, developed in subsequent sections of this report, the annual average flow at which the maximum month flows are expected to reach 14.5 mgd is 11.6 mgd. This flow is not expected to be reached until after 2030, even if assuming the higher flow projections. The earliest expansion in plant capacity would be required by 2032 based on the higher demand projections. The Mason Farm WWTP has adequate permitted flow capacity through the 20-year planning period. The North Carolina Division of Water Quality (NCDWQ) provides guidance that WWTP owners should begin engineering planning when the WWTP reaches 80% of its permitted capacity, and begin construction of expansion facilities when the average flows reach 90% of the permitted capacity. For the Mason Farm WWTP, it is anticipated that 80% of the permitted capacity (or 11.6 mgd) will be reached after 2030 and likely closer to 2040. This NCDWQ guidance further validates the anticipation that the Mason Farm WWTP will not require liquid capacity expansion within the 20-year planning horizon.

Brown and Caldwell previously identified that the next expansion to the Mason Farm WWTP should increase the plant capacity to 18.5 mgd. This capacity is expected to be reached once annual average flows approach 14.8 mgd based on the 1.25 maximum month flow peak factor. This flow is not expected to occur until after 2060 based on the expected flow projections. A capital project to expand the facility to 18.5 mgd completed by 2040 should provide adequate capacity for the twenty years following the expansion. Hazen and Sawyer concurs with the previous consultant's findings that the next logical expansion to the Mason Farm WWTP would likely be to 18.5 mgd.

The wastewater projection methodology did not account for reductions in inflow and infiltration (I/I) in the collection system. OWASA has invested in inflow and infiltration reduction, and wastewater flows attributed to inflow and infiltration would be expected to decrease as improvements to the collection system are completed. OWASA recently commissioned an inflow and infiltration study by another consultant (CDM), and the results of the study are not yet available. It is anticipated that additional improvements will be recommended and OWASA will continue to invest in inflow/infiltration reduction.

The current wastewater projections assume inflow and infiltration will increase at the same rate as the increase in water demand. This is a conservative assumption as it does not account for specific projects to reduce inflow and infiltration or tighter collection systems associated with new construction. The wastewater flow projections should be revisited once the inflow/infiltration study is completed to assess the impacts of future reduction on wastewater flows.

The WWTP flow projections developed in the OWASA Long-Range Supply Plan Update include the projected reclaimed water use in addition to flow associated with the potable water demand. The methodology applies the same adjustment factors to the potable water and reclaimed water demand in developing wastewater flows. Reclaimed water is currently used to provide make-up water for cooling towers at the UNC Central Campus. Although a portion of the cooling water is blown-down and drains to the OWASA sanitary collection system, the remainder of the reclaimed water flow is evaporated. Future expansion of the reclaimed water system at the UNC Central Campus will provide additional cooling tower water, toilet flushing and irrigation. The current methodology for projecting reclaimed water impacts assumes that reclaimed water return to the collection system will be similar to current operations. The wastewater growth projections may be conservative if a significant portion of the future reclaimed water demand is not returned to the wastewater collection system in the future.

A major development planned within the Mason Farm WWTP service area is Carolina North. Reclaimed water usage is also proposed at the Carolina North campus, and the expected water demand projections assume that 25% of Carolina North's water needs will be met through reclaimed water or other non-potable water sources. OWASA submitted a letter to Mr. Roger Stancil, Chapel Hill Town Manager, on April 9, 2009 summarizing the "Potential Impacts of Carolina North on OWASA's Future Operations". Included in the letter was a discussion on potential reclaimed water applications at Carolina North and impacts of the development on the Mason Farm WWTP. The letter indicated that UNC planned to meet the water needs of Carolina North through rainwater harvesting and on-site wastewater treatment to provide reclaimed water for irrigation, toilet flushing, cooling tower make-up water and other non-potable uses. It also mentioned that the reclaimed water may be supplemented through scalping flow from a nearby OWASA sewer line and treating it on-site to reuse standards. The letter

concluded that it was unlikely that wastewater generated at Carolina North will have significant hydraulic impacts on the Mason Farm WWTP, but noted that additional analysis is required as plans for Carolina North develop.

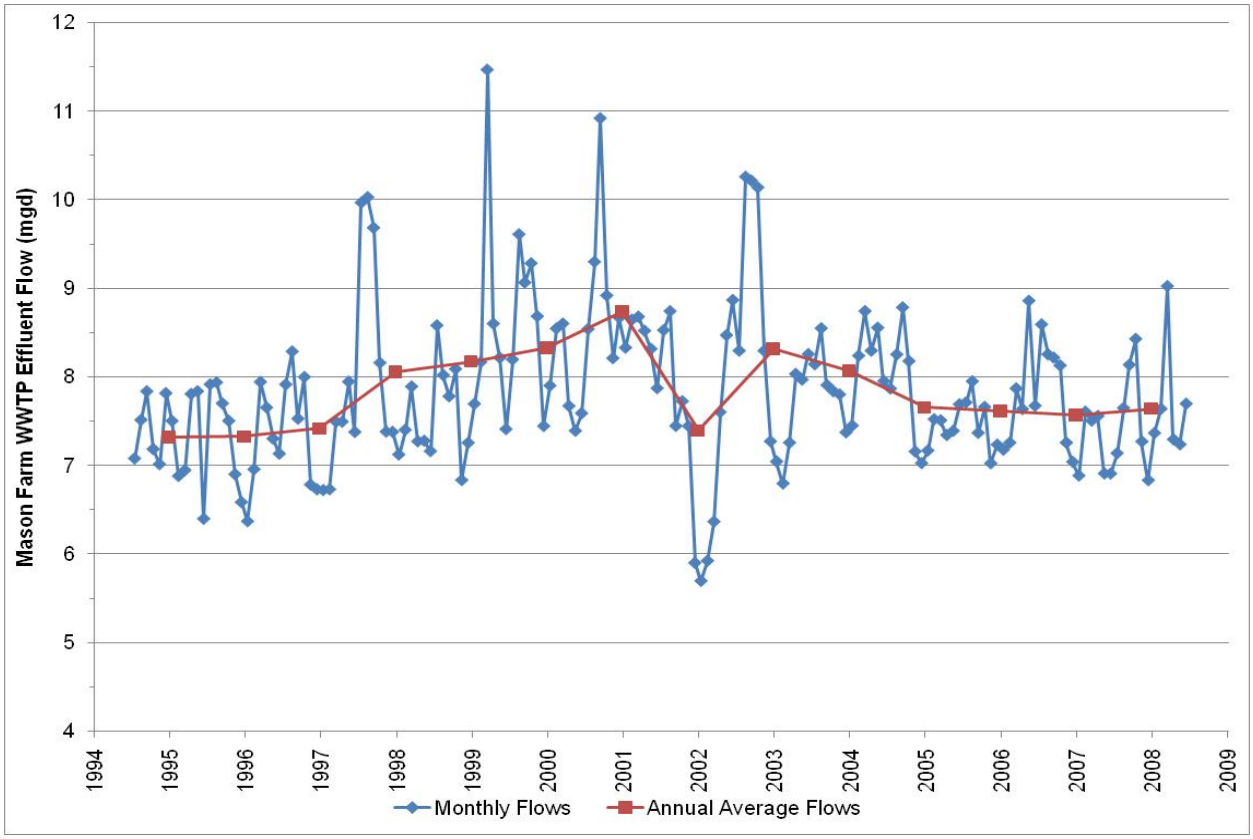
Despite these assumptions and the level of conservatism they bring to the wastewater flow projections, OWASA's flow projection methodology is very detailed and likely provides for a better estimate than more traditional flow projection approaches, which are typically based on applying a specific flow per equivalent dwelling unit (EDU) or population equivalent to population projections. Providing specific flow rates for each customer type based on recent flow data allows greater flexibility in assessing the impacts of future development since different rates of growth are applicable to each customer type. The relationship between the wastewater flow and the water demand should continue to be monitored and adjusted as appropriate for future wastewater demand projections since reductions in inflow and infiltration and increased reclaimed water usage may reduce the fraction of the potable water demand which ends up in the collection system.

The WWTP flow projections in the Long-Range Water Supply Plan Update appear conservative, and it is unlikely that the permitted design flow will be reached within the 20-year planning period. It is recommended that OWASA adopt the wastewater treatment flow projections in the Long-Range Water Supply Plan and base the analysis of need for improvements to the Mason Farm WWTP on these projections. It is also recommended that OWASA revisit the wastewater flow projections periodically, at least once every five years, and once the inflow and infiltration study is complete.

2.3 Flow Analysis

Figure 2-3 presents annual average and monthly average influent wastewater flows from 1995 through 2008. Wastewater flows increased from 1995 through 2001, and then decreased significantly in 2002 due to severe drought conditions. Wastewater flows increased in Spring 2003, and have been steady since 2005. The annual average flow from 2005 through 2008 was 7.6 mgd.

Figure 2-3
Annual Average and Monthly Average Influent Wastewater Flows



The Mason Farm WWTP is permitted to discharge a maximum month flow of 14.5 mgd; therefore it is expected that monthly flows will exceed the permitted flow well before annual average flows approach 14.5 mgd. The maximum month to annual average flow ratio was analyzed for each year from 1995 through 2008. The average maximum month flow ratio during this period was 1.18. The highest ratio occurred in 1999, when the average monthly flow during September was 1.4 times the annual average flow. September 1999 also had the highest average flow of any month from 1995 through 2008. The high reported flows in September 1999 coincide with Hurricane Floyd, which bought record rainfall and flooding to Eastern and Central North Carolina.

Excluding September 1999, the highest maximum month to annual average flow ratio was 1.25, with three separate years (1998, 2001 and 2003) having a similar ratio. It is recommended that a maximum month to annual average flow ratio of 1.25 be applied to the design criteria for the Mason Farm WWTP due to the infrequency of storm events similar in severity to Hurricane Floyd. Applying the 1.25 ratio to the permitted maximum month average daily flow of 14.5 mgd results in an annual average capacity of 11.6 mgd; therefore an annual average design flow of 11.6 mgd has been selected to develop the design criteria at the current permitted flow capacity.

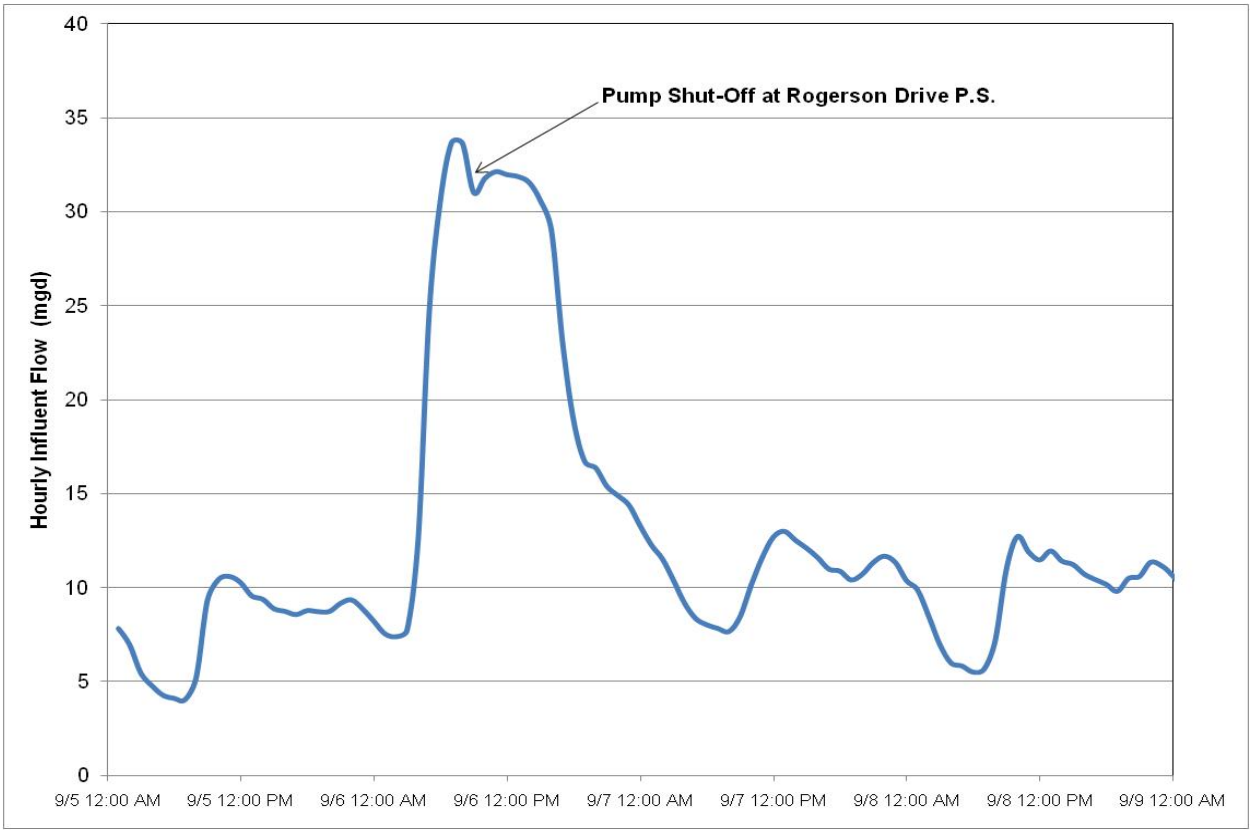
Additional flow data from 2004 through 2008 were evaluated to determine peak day flow factors. Table 2-3 summarizes the influent flows during this period.

Table 2-3
Historical Flow Data – 2004-2008

Year	Average (mgd)	Max Month (mgd)	Max Week (mgd)	Max Day (mgd)
2004	8.06	8.74	9.73	14.14
2005	7.65	8.78	9.73	15.51
2006	7.61	8.86	10.11	15.68
2007	7.56	8.59	10.06	13.45
2008	7.64	9.02	11.53	22.54
Average	**7.70**	**8.80**	**10.23**	**16.27**

The peak day flow during the five year period occurred in September 2008 (Tropical Storm Hanna) and corresponds to a significant storm event. The reported daily flow was 22.54 mgd, which is nearly three times the annual average flow observed in 2008. Hourly flow data for this peak flow event were evaluated, and a flow hydrograph, presented in Figure 2-4, was developed. One of the pumps at the Rogerson Drive Pump Station cut off at 8 am on September the 6th, and the pump station effluent flow was reduced from 19 mgd to 17 mgd while the wet well continued to fill; however there were no reported overflows during this storm event. The impact of the pump shut off on the influent flow to the Mason Farm WWTP can be observed in Figure 2-4.

Figure 2-4
September 2008 Peak Flow Hydrograph

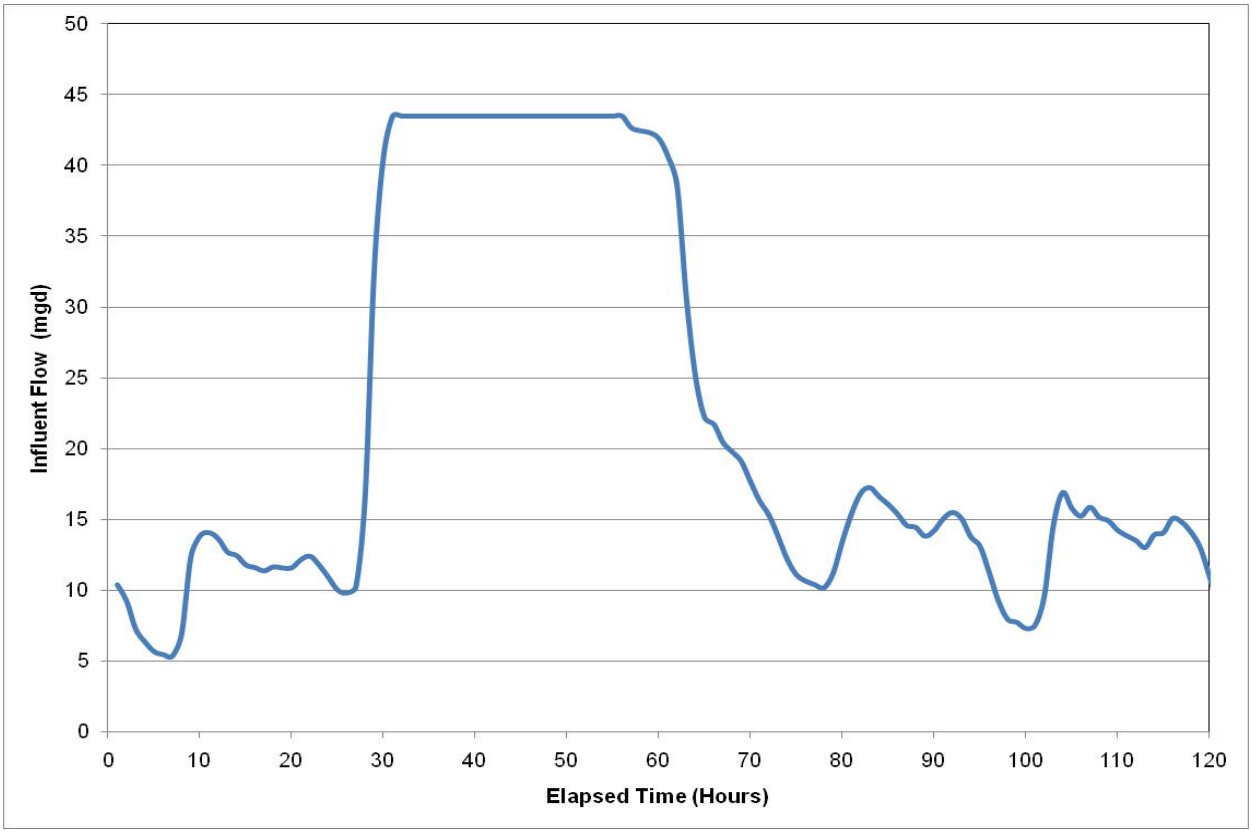


The peak hour flow during the storm event was 33.7 mgd, resulting in peaking factors of 1.5 of the maximum day flow and 4.4 of the 2008 annual average flow. The peak hour flow occurred prior to the pump shut-off at the Rogerson Drive Pump Station. The Mason Farm WWTP is designed for a peak hydraulic flow of 43.5 mgd, which is three times the permitted maximum month capacity. Note that this is significantly more conservative than North Carolina Division of Water Quality (NCDWQ) guidelines, which require a minimum peak factor of 2.5 applied to the permitted design flow. Applying the 1.25 design maximum month to annual average ratio to the 4.4 peak hour to annual average ratio results in a peak hour to maximum month flow ratio of 3.5, which is greater than the current 3.0 design peaking factor.

The projected annual average flow in 2030 is 10.22 mgd. Applying a peak hour factor of 4.4 results in a potential peak hour flow of 45 mgd by 2030, which is slightly above the current design peak hour flow of 43.5 mgd. Hydraulic peaking factors generally decrease as average flows to a WWTP increase, and Hazen and Sawyer would expect that the current 43.5 mgd hydraulic capacity is sufficient through 2030. OWASA should continue to monitor infiltration and inflow in the collection system and consider whether investments in improvements to decrease wet weather flows would reduce the potential for hydraulic and process problems associated with peak flow events.

A design storm hydrograph was developed based on the September 2008 flow event and is presented in Figure 2-5. The storm is based on the projected 2030 annual average flow of 10.22 mgd. The peak hourly flow was limited to 43.5 mgd. The September 2008 storm had a very sharp peak where flows quickly diminished after several hours of peak flow. This is atypical of peak wet weather flow events at domestic WWTPs, where the peak flow is sustained for a longer period. In addition the pump shut-off at the Rogerson Drive Pump Station may have contracted the peak flow. The design storm hydrograph includes a peak flow that is sustained for a period of 24 hours to more closely resemble a typical storm event. The design storm hydrograph will be used in conjunction with BioWin process modeling and computational fluid dynamic (CFD) clarifier modeling during the evaluation of the current plant capacity and treatment alternatives to determine necessary modifications required to protect the biological process during wet weather events.

Figure 2-5
Design Storm Hydrograph



2.4 Influent Wastewater Characteristics

Influent data from 2004 through August 2009, reported in the monthly discharge monitoring reports (DMRs) submitted to the State, were evaluated to develop design influent wastewater characteristics. Influent five-day carbonaceous biochemical oxygen demand ($cBOD_5$), total suspended solids (TSS), volatile suspended solids (VSS), ammonia-nitrogen (NH_3-N) and total phosphorus (TP) concentrations were reported five days per week. Total Kjeldahl nitrogen (TKN) was reported once per month from 2004 through February 2007, and the reporting frequency was increased to weekly beginning in March 2007. The reported $cBOD_5$ concentrations are based on an inhibited test and will need to be adjusted to a "true" BOD_5 concentration for design.

The influent samples are collected by a flow paced composite sampler located downstream of preliminary treatment. Samples are collected on a 24-hour basis. The influent composite did not provide a representative sample of the true raw influent wastewater entering the Mason Farm WWTP prior to December 2007 since septage and plant recycle flows, including process drains, digester overflows and solids processing filtrate streams, were returned upstream of the sampler. The influent composite sampler was relocated upstream of the rotary press (which began operation in October 2007) filtrate return, and other potential recycle streams including digester overflows and plant drains were redirected downstream of the sampler in December 2007. Septage is still received upstream of the influent composite sampler, and its impacts are included in the raw influent composite samples throughout the historical period evaluated.

Figures 2-6 through 2-9 present monthly average influent $cBOD_5$, TSS, NH_3-N and TP concentrations and loads, respectively, from 2004 through 2008.

Figure 2-6
Influent $cBOD_5$ Concentrations and Loads

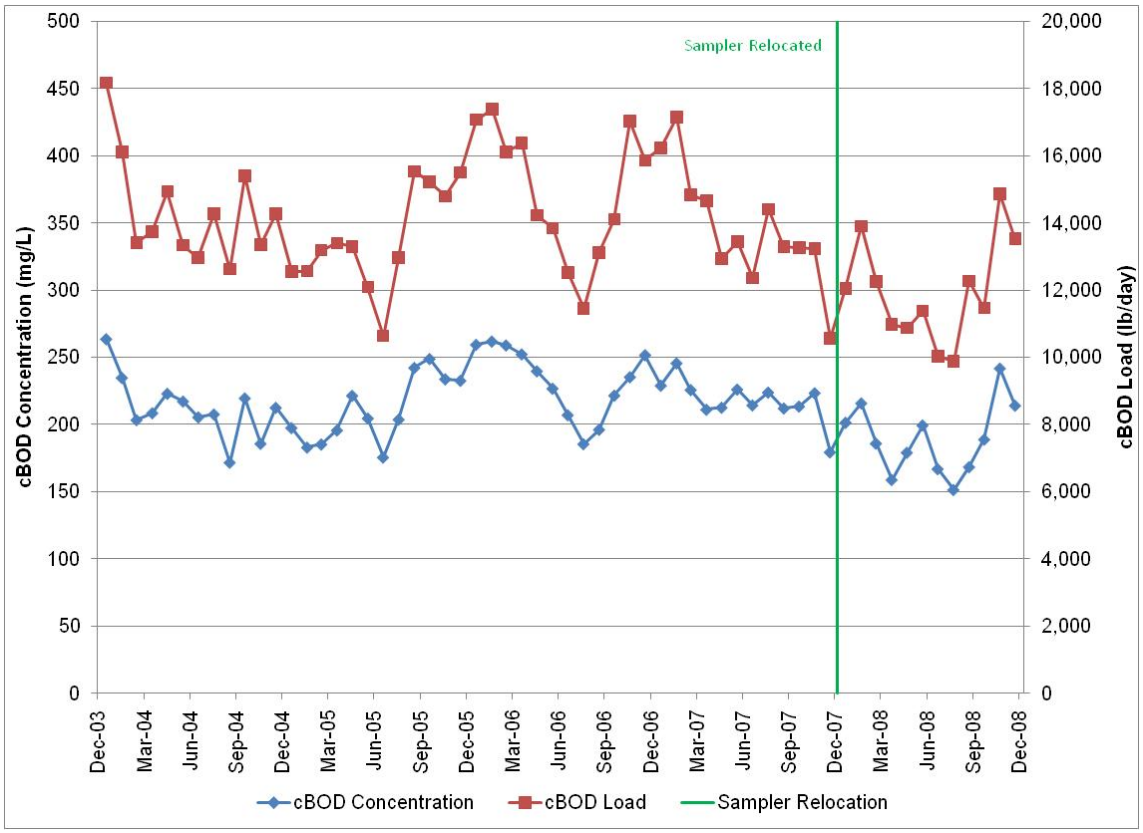


Figure 2-7
Influent TSS Concentrations and Loads

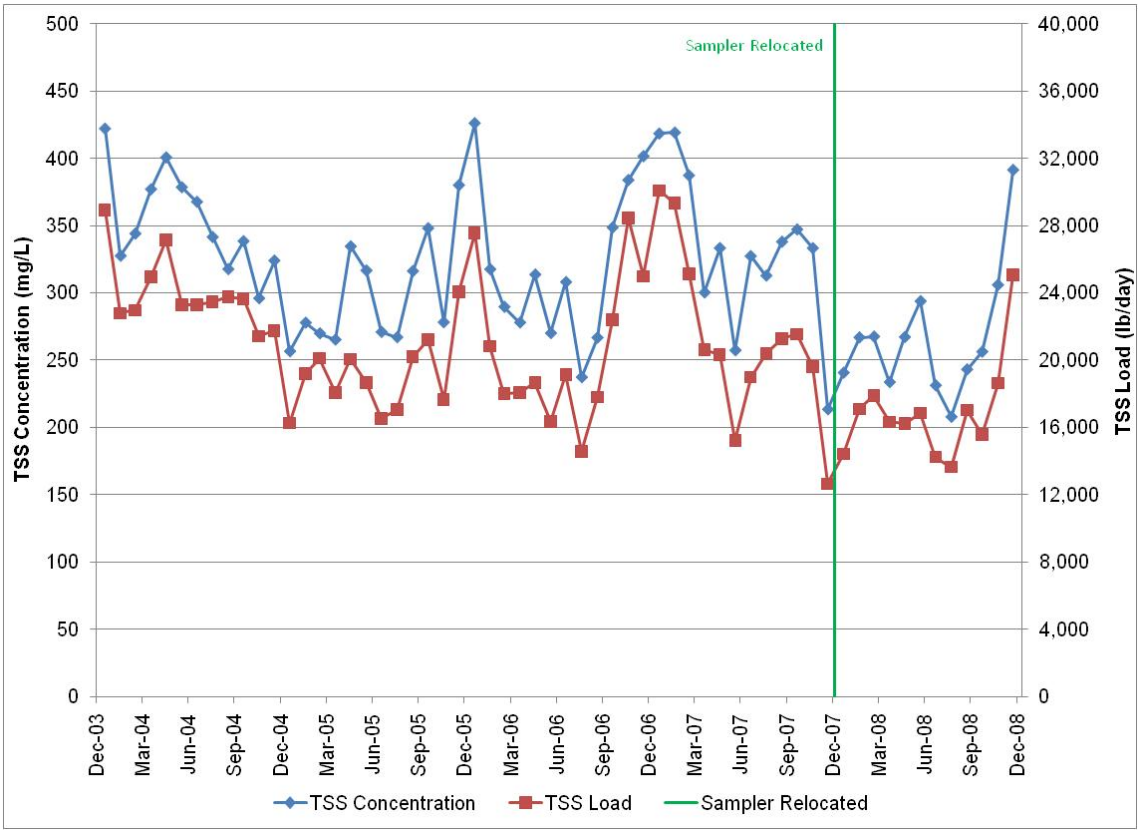


Figure 2-8
Influent NH_3-N Concentrations and Loads

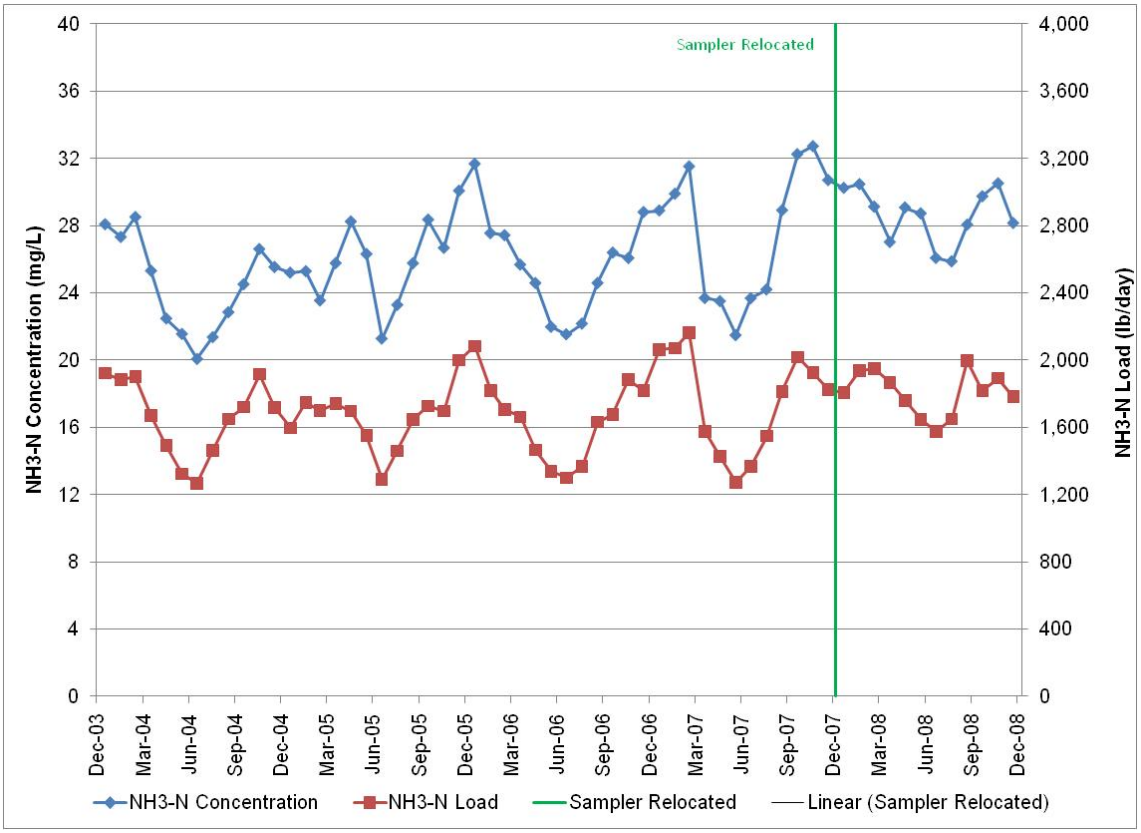
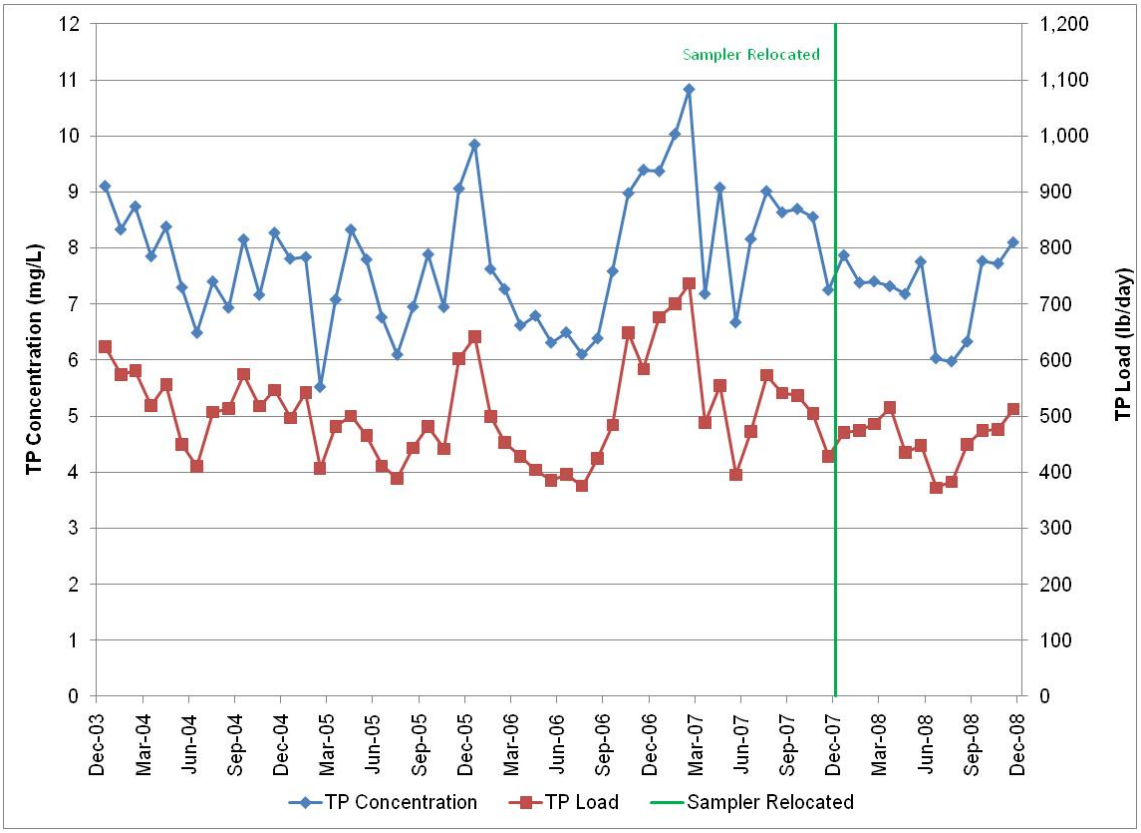


Figure 2-9
Influent TP Concentrations and Loads



A statistical evaluation of the available influent data was performed, and annual average concentrations and loads, load peaking factors and typical influent ratios were determined. Concentration values outside of two standard deviations from the mean were excluded from the computation of average values and influent ratios. Minimum and maximum peaking factors were developed based on excluding concentration values outside of three standard deviations.

Tables 2-4 and 2-5 summarize the annual average concentration and load data from 2004 through 2009. The average concentrations and loads from January 2004 through December 2007 were calculated and compared to the average concentrations and loads from January 2008 through August 2009 to compare influent wastewater quality before and after plant recycle flows were removed from the influent sample.

Table 2-4
Annual Average Influent Concentrations – 2004-2009

Year	$cBOD_5$ (mg/L)	TSS (mg/L)	VSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
2004	211	353	317	24.5	41.2	7.8
2005	210	298	267	25.8	36.1	7.3
2006	231	315	285	25.6	37.0	7.2
2007	218	332	303	27.2	45.1	8.5
2008	186	261	239	28.8	39.5	7.1
2009[1]	211	274	247	27.4	36.5	7.1
2004-2007 Avg	**218**	**325**	**293**	**25.5**	**42.5[2]**	**7.7**
2008/2009 Avg	**196**	**267**	**242**	**28.2**	**38.2**	**7.1**

[1]*Through August 2009*

[2]*Average of all data from 2004-2007. This value is different than the average of the annual average values since sampling frequency was increased in March 2007*

Table 2-5
Annual Average Influent Loadings – 2004-2009

Year	$cBOD_5$ (lb/d)	TSS (lb/d)	VSS (lb/d)	NH3-N (lb/d)	TKN (lb/d)	TP (lb/d)
2004	14,277	23,951	21,541	1,657	2,864	531
2005	13,432	19,046	17,047	1,647	2,255	468
2006	14,792	20,154	18,293	1,635	2,242	461
2007	13,841	21,163	19,373	1,728	2,794	538
2008	11,797	16,550	15,143	1,819	2,532	448
2009[1]	13,868	18,194	16,376	1,793	2,361	466
2004-2007 Avg	**14,100**	**21,200**	**19,100**	**1,660**	**2,680**	**510**
2008/2009 Avg	**12,700**	**17,300**	**15,700**	**1,810**	**2,470**	**460**

[1]*Through August 2009*

Reported average influent $cBOD_5$ and TSS loads have decreased by 11% and 18%, respectively, since plant recycle flows were removed from the influent composite samples, indicating that recycle streams were likely a significant source of solids reported in the influent sample. Although influent TKN concentrations have decreased slightly, influent ammonia concentrations have increased by 11% since recycle flows were removed. Influent total phosphorus concentrations have also decreased slightly since 2007.

Historical load peaking factors with and without plant recycle stream influences are summarized in Table 2-6 and Table 2-7, respectively. The load peaking factors are fairly typical of a municipal wastewater treatment plant the size of the Mason Farm WWTP and are generally consistent between the two periods evaluated. Maximum month $cBOD_5$ and maximum month and day TP loads between 2004 and 2007 are greater than typically observed values, but are more representative of typical wastewater patterns in 2008 and 2009. Maximum month and maximum week TKN data is not provided since TKN was only reported monthly through most of the historical period evaluated.

Table 2-6
Historical Load Peaking Factors – 2004-2007

	$cBOD_5$	TSS	VSS	NH3-N	TKN	TP
Minimum Day	0.48	0.27	0.28	0.59	0.66	0.47
Annual Average	1.00	1.00	1.00	1.00	1.00	1.00
Maximum Month	1.38	1.50	1.48	1.31	-	1.54
Maximum Week	1.67	2.13	2.12	1.48	-	2.04
Maximum Day	1.93	3.44	3.36	1.70	1.85	3.26

Table 2-7
Historical Load Peaking Factors – 2008/2009

	$cBOD_5$	TSS	VSS	NH3-N	TKN	TP
Minimum Day	0.42	0.34	0.34	0.60	0.63	0.48
Annual Average	1.00	1.00	1.00	1.00	1.00	1.00
Maximum Month	1.30	1.52	1.51	1.16	-	1.19
Maximum Week	1.85	2.15	2.01	1.45	-	1.45
Maximum Day	2.35	3.31	3.29	1.69	1.63	2.18

Table 2-8 summarizes the average observed influent ratios, which are typical of municipal wastewater. The ammonia to TKN (NH_3-N:TKN) ratio has increased since the recycle streams were removed, which may be due to the elimination of particulate organic nitrogen returned with the plant recycle streams. The $cBOD_5$:TKN ratio and $cBOD_5$:TP ratio are borderline for reliable nutrient removal performance without supplemental carbon addition. These ratios will decrease through primary treatment as particulate BOD is removed, and the primary effluent will likely be carbon limited. This issue is discussed in the evaluation of the existing plant capacity and treatment alternatives.

Table 2-8
Average Influent Wastewater Ratios

	2004-07	2008-09
$cBOD_5$:TKN	5.21	5.04
$cBOD_5$:NH_3-N	8.72	7.09
$cBOD_5$:TP	29.1	28.3
$cBOD_5$:TSS	0.67	0.73
NH_3-N:TKN	0.63	0.74
VSS:TSS	0.89	0.91

The wastewater evaluation suggests that the plant recycle streams contain significant solids, which impacted the influent $cBOD_5$, TSS and TKN concentrations reported prior to 2008. Process simulations to evaluate plant capacity and nutrient removal alternatives include modeling of the solids handling process and impacts from these sidestreams. Basing raw influent characteristics on data prior to 2008 may lead to "double counting" of the impacts from the solids sidestreams. The influent characteristics between January 2008 and August 2009, which preclude the influence of plant recycle flows, were used as the basis of design for the modeling efforts.

The influent composite sample and design wastewater characteristics still include the impacts of septage. Historical information on septage flows and strength was used during the process modeling to evaluate the impacts septage has on the capacity of the Mason Farm WWTP to meet current and future effluent quality objectives.

2.5 Wastewater Temperatures

The reported weekly influent wastewater temperatures were evaluated and are summarized in Table 2-9. An annual average temperature of 20°C is recommended for design, which is typical of wastewater treatment plants in the region. A minimum wastewater temperature of 13°C is proposed for evaluating the biological process under worst-case winter temperature conditions.

Table 2-9
Annual Temperature Data – 2004-2008

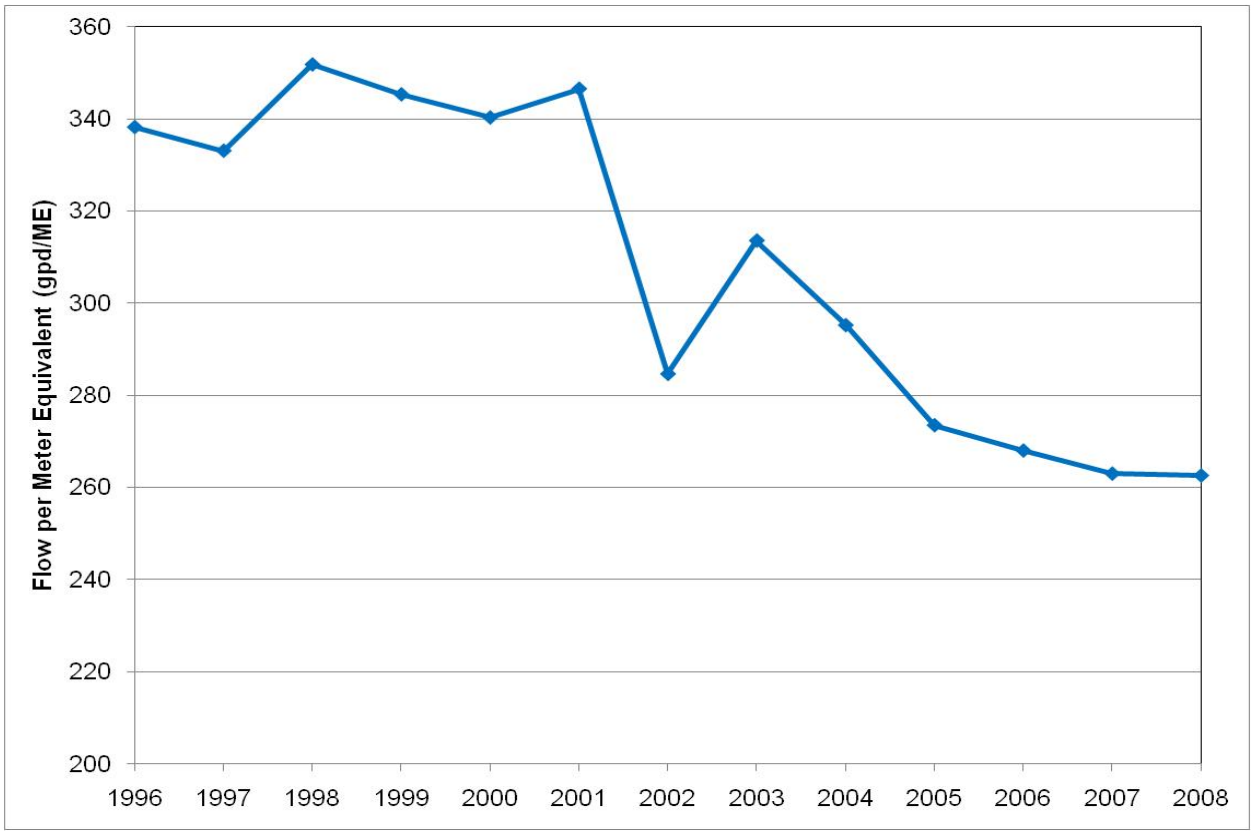
Year	Annual Average (°C)	Minimum Month (°C)	Minimum Day (°C)	Maximum Month (°C)	Maximum Day (°C)
2004	20.1	14.4	14.0	24.8	25.8
2005	20.1	15.0	13.2	25.7	26.0
2006	20.5	15.5	15.0	26.1	26.3
2007	20.8	14.8	14.6	26.5	27.0
2008	20.5	16.2	15.4	25.7	26.5
Average	**20.4**	**15.2**	**14.4**	**25.8**	**26.3**

2.6 Impact of Water Conservation

Influent wastewater concentrations are expected to increase in the future as water efficient appliances and fixtures are installed and infiltration and inflow are addressed. The evaluation of the historical influent flows and loads between 2004 and 2009 do not indicate a strengthening of wastewater concentrations, but this may be difficult to discern due to the previous influence of recycle streams on the influent samples. Increased wastewater concentrations could lead to a WWTP reaching its load limitation prior to reaching its hydraulic capacity, which could increase permit compliance challenges prior to the plant's permitted hydraulic capacity being reached.

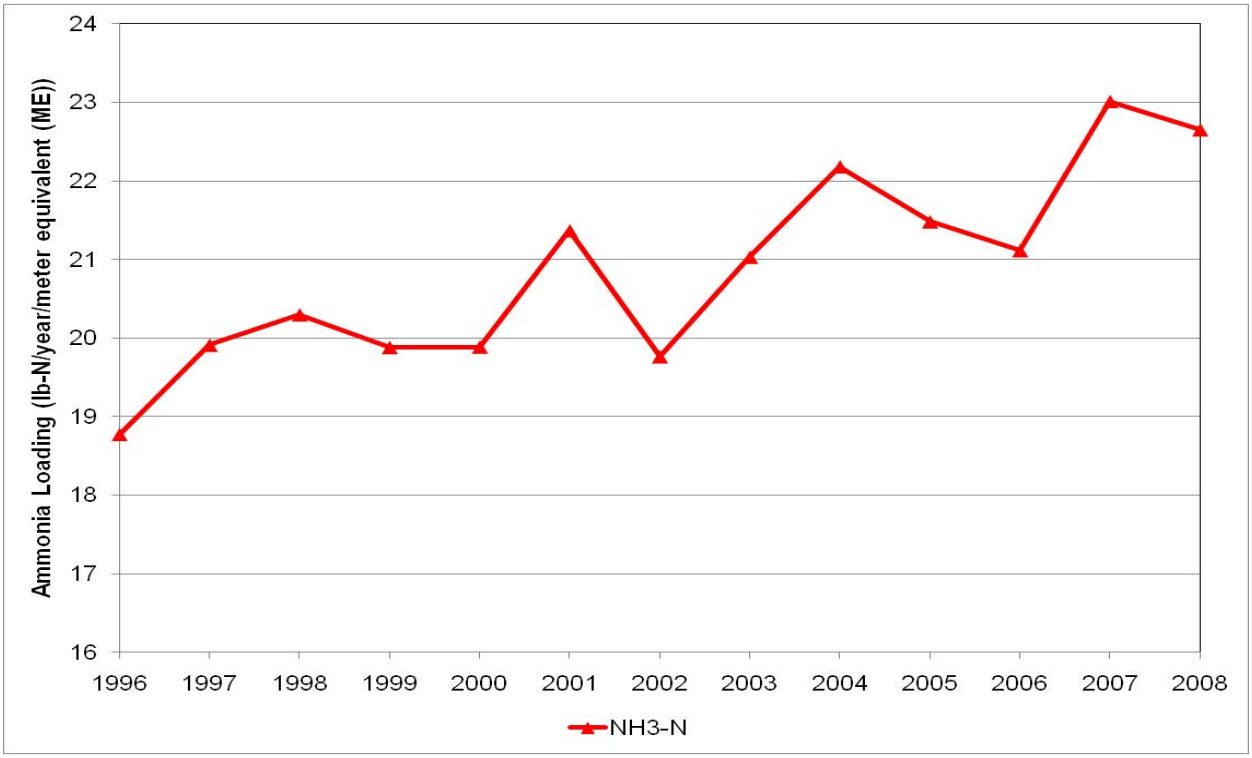
Annual wastewater flow and loading data from 1996 through 2008 were analyzed to determine if there were any long term trends indicating increased wastewater strength due to the conservation measures already in place in the Mason Farm WWTP service area. The annual average influent flow was compared to the number of meter equivalents served by OWASA, and a wastewater flow per meter equivalent was developed to determine the extent of water conservation over the past 13 years. Figure 2-10 indicates significant water conservation methods are in place in the OWASA service area as the flow per meter equivalent has decreased by almost 25% in the past 13 years. Figure 2-10 also suggests that the rate of water conservation may have stabilized over the past few years.

Figure 2-10
Flow per Meter Equivalent



A similar evaluation was performed for $cBOD_5$, TSS, NH_3-N and TP to determine the load of each constituent per meter equivalent from 1996 through 2008. The $cBOD_5$, TSS and TP loads per meter equivalent were relatively consistent and did not suggest a strengthening of wastewater loads; however the NH_3-N load per meter equivalent increased an average of 1.7% annually as shown in Figure 2-11.

Figure 2-11
Ammonia Load per Meter Equivalent



Since particulate matter associated with plant recycle streams likely accounted for a portion of the reported influent $cBOD_5$, TSS and TP prior to 2008, it is difficult to discern whether the actual influent wastewater concentrations for these parameters have increased. Post-digestion solids including digester overflows and filtrate from dewatering equipment trials may have also influenced the reported influent ammonia concentrations, but a definite increase in ammonia load to the plant is suggested by the data. An annual 1.7% increase in ammonia load per meter equivalents from 2008 values results in an expected ammonia concentration of approximately 40 mg/L in 2030, which is similar to that observed at other wastewater treatment plants treating high strength domestic wastewater resulting from water conservation practices and inflow/infiltration reduction.

An annual 1.7% linear increase from 2008 loads per meter equivalents was also applied to the influent $cBOD_5$, TSS and TP concentrations to develop high strength wastewater characteristics for 2030. Table 2-10 compares the resulting influent concentrations at 2030 to the 2008 concentrations. The resulting influent wastewater concentrations are 35-40% greater than the reported 2008 concentrations and likely represent a "worst case" increase in wastewater strength.

Table 2-10
Projected 2030 Wastewater Concentrations Based On Water Conservation

Year	$cBOD_5$ (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
2008 Concentration	186	261	28.8	39.5	7.1
Revised for Conservation	258	365	39.2	53.0	9.9

2.7 Design Influent Criteria Summary

Tables 2-11 and 2-12 summarize the design flows and concentrations under annual average and maximum month conditions, respectively, at 2030. Maximum month conditions are based on the maximum month flow peaking factor of 1.25 and maximum month load factors provided in Table 2-12. Maximum month peak factors for ammonia, TKN and TP were adjusted to 1.30 to match the $cBOD_5$ factor between 2008 and 2009. The current average influent cBOD5 concentration has been divided by 0.94 to provide for a "true" BOD5 value based on detailed wastewater characterization performed from August 16 through August 22, 2009. The design criteria provided assumes that there will be no new significant industrial loads and the influent wastewater will continue to be primarily domestic. It is also assumed that septage flows and loads will remain similar in proportion to current operations.

Design influent criteria are provided for two scenarios; current wastewater strength and increased wastewater strength due to water conservation, increased reclaimed water usage and reduced inflow/infiltration. The purpose of providing these two sets of characteristics is to bracket the potential range of wastewater loadings to the plant through 2030. In addition both the expected demand and higher demand flow projections will be evaluated to provide a range of potential plant loadings in the year 2030 and determine when expansion of capacity and/or improvements to the Mason Farm WWTP can be expected.

Table 2-11
Summary of Design Influent Criteria at 2030 – Annual Average Conditions

	Flow (mgd)	Temp. (°C)	BOD_5 (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
Expected Flow, Current WW Strength	10.4	20	210	270	30	40	7.1
Expected Flow, Increased WW Strength	10.4	20	274	365	39	53	9.9
Higher Flow, Current WW Strength	11.3	20	210	270	30	40	7.1
Higher Flow, Increased WW Strength	11.3	20	274	365	39	53	9.9

Table 2-12
Summary of Design Influent Criteria at 2030 – Maximum Month Conditions

	Flow (mgd)	Temp. (°C)	BOD_5 (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
Maximum Month Factor	1.25	-	1.30	1.50	1.30	1.30	1.30
Expected Flow, Current WW Strength	13.0	13	218	324	31	42	7.4
Expected Flow, Increased WW Strength	13.0	13	285	438	41	55	10.3
Higher Flow, Current WW Strength	14.1	13	218	324	31	42	7.4
Higher Flow, Increased WW Strength	14.1	13	285	438	41	55	10.3

Design influent criteria was also established for annual average and maximum month conditions at the current 14.5 mgd permitted design capacity and are provided in Table 2-13.

Table 2-13
Summary of Design Influent Criteria at 14.5 mgd Design Flow

	Flow (mgd)	Temp. (°C)	BOD_5 (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
Annual Average, Current WW Strength	11.6	20	210	270	30	40	7.1
Annual Average, Increased WW Strength	11.6	20	274	365	39	53	9.9
Maximum Month, Current WW Strength	14.5	13	218	324	31	42	7.4
Maximum Month, Increased WW Strength	14.5	13	285	438	41	55	10.3

BioWin process modeling will also be conducted at current flows and loads to establish process criteria for optimized nutrient removal at current and near-term conditions.

2.8 Effluent Permit Limits and Treatment Objectives

The Mason Farm WWTP is permitted under National Pollutant Discharge Elimination System (NPDES) Permit number NC0025241. The current permit is effective from March 1, 2007 through July 31, 2011. Two sets of effluent limits are provided in the permit. The first set of effluent limits was effective from the permit start date until the plant was expanded above 12.0 mgd. The second set of permit limits became effective once the plant was expanded beyond 12.0 mgd to the current permitted capacity of 14.5 mgd. The Mason Farm WWTP is currently operating under the second set of limits as provided in Table 2-14.

Table 2-14

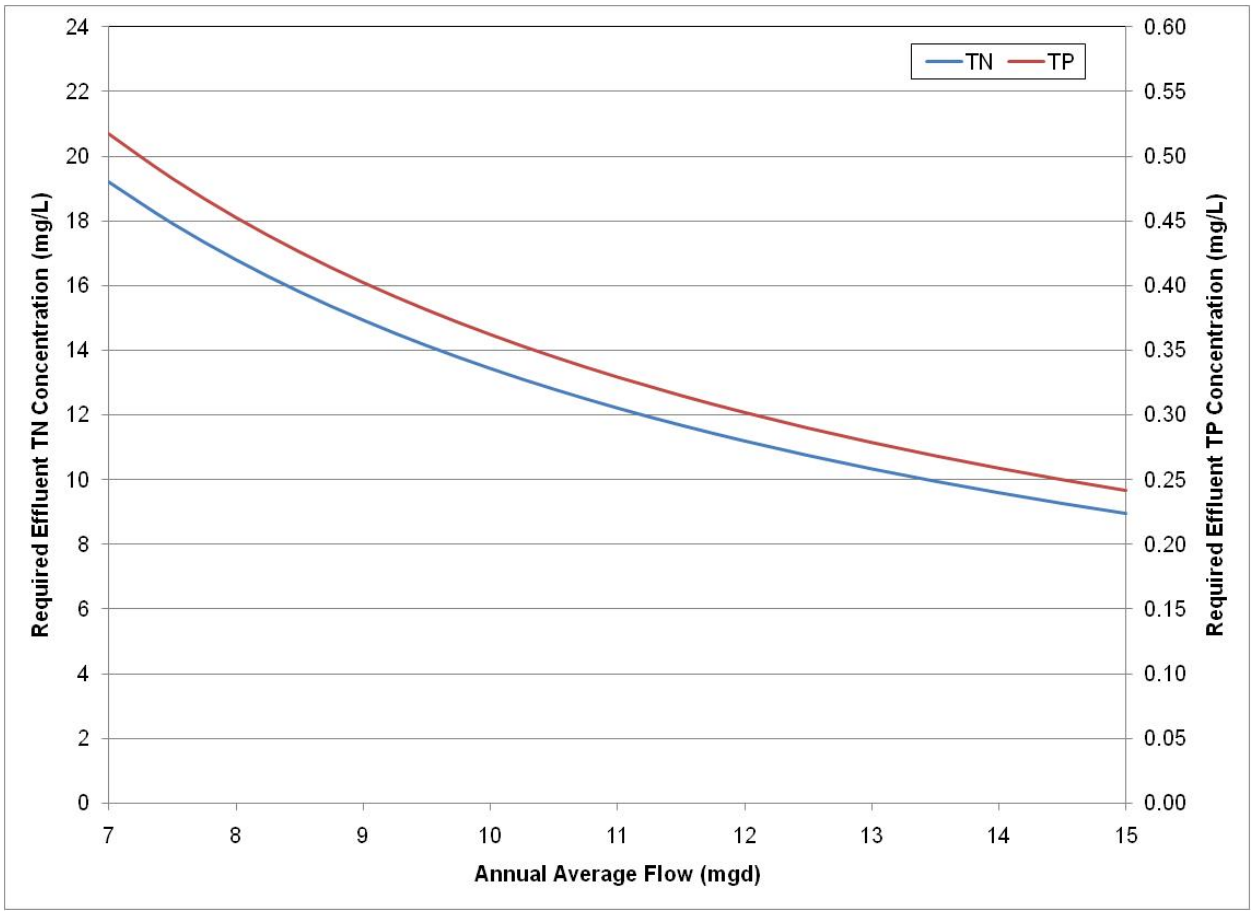
Current Permitted Effluent Limits

	Value	Period	Sampling Frequency
Flow (mgd)	14.5	Monthly	Continuous
$cBOD_5$ (mg/L) (April-October)	4.0	Monthly	Daily
$cBOD_5$ (mg/L) (November-March)	8.0	Monthly	Daily
TSS (mg/L)	30.0	Monthly	Daily
NH_3-N(mg/L) (April-October)	1.0	Monthly	Daily
NH_3-N(mg/L) (November-March)	2.0	Monthly	Daily
Total Kjeldahl Nitrogen (mg/L)	Report	-	Weekly
Nitrate/Nitrite (mg/L)	Report	-	Weekly
Total Phosphorus (mg/L)	Report	-	Weekly
Total Phosphorus (lbs/year)	11,030	Annual	Monthly
Total Nitrogen (mg/L)	Report	-	Weekly
Total Nitrogen (lbs/year)	409,448	Annual	Monthly
Dissolved Oxygen (mg/L)	> 6.0	Monthly	Daily
pH	6 – 9	Monthly	Daily
Fecal Coliform (cfu/100 mL)	200	Monthly	Daily
Total Residual Chlorine (max. µg/L)	18	Daily	Daily

The Mason Farm WWTP is currently required to limit effluent TN and TP discharges based on annual mass limits. These annual mass limits correspond to effluent TN and TP concentrations of 9.3 mg/L and 0.25 mg/L, respectively, at the permitted 14.5 mgd design flow. Required effluent TN and TP concentrations at the current annual average flow of 7.7 mgd are 17.5 mg/L and 0.47 mg/L, respectively. Figure 2-12 illustrates the necessary decrease in effluent nutrient concentrations as flows increase.

Figure 2-12

Required Annual Average Nutrient Concentrations – Current Permit



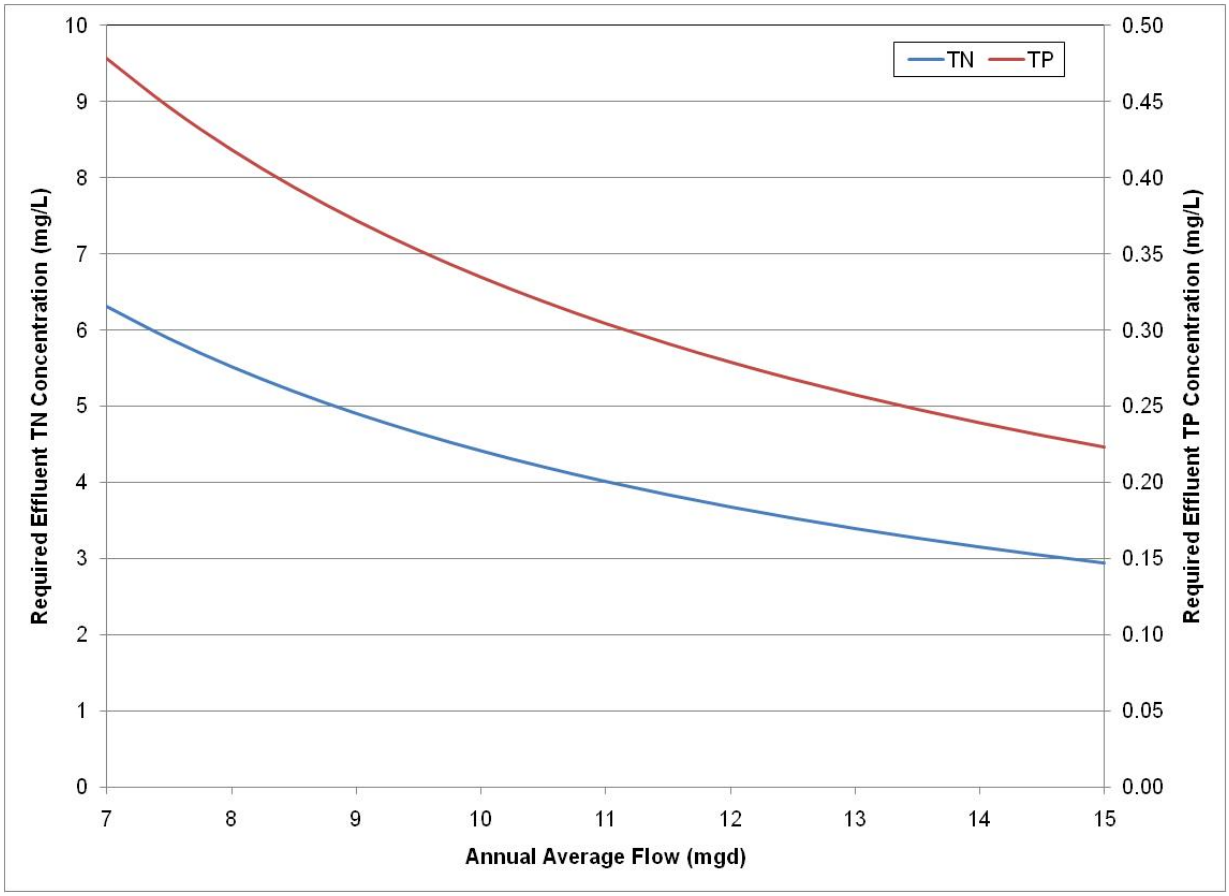
The Jordan Lake nutrient regulations will require that the Mason Farm WWTP meet annual effluent nutrient TN and TP mass limits of 134,375 lb/year and 10,188 lb/year, respectively. These mass limits correspond to effluent TN and TP concentrations of 3.04 mg/L and 0.23 mg/L, respectively, at the 14.5 mgd design flow. The Jordan Lake nutrient reduction rules for wastewater treatment plant discharges were enacted on August 11, 2009, and the Mason Farm WWTP will be required to comply with the new TP standards in calendar year 2010 and with the TN standards by 2016. Figure 2-13 presents required effluent nutrient concentrations versus flow assuming the Jordan Lake nutrient mass limits.

The Jordan Lake Rules allow for an association to be formed to achieve group compliance by multiple dischargers into Jordan Lake. While no association exists as of

the development of this study, there is potential for the dischargers to form such a group. The regulatory community has made a distinction between the Haw River arm and the New Hope arm of Jordan Lake that the Mason Farm WWTP discharges into. Group compliance would likely be implemented for the New Hope arm of the lake separately from the other dischargers in the Haw River arm. Only the Mason Farm WWTP, the Durham County WWTP and the South Durham Water Reclamation Facilities are the only municipal dischargers into the New Hope arm. Because each of the New Hope dischargers will have to meet stringent nutrient wasteload allocations based on total nitrogen concentrations of 3.0 mg/L and total phosphorus concentrations of 0.23 mg/L at their permitted flows, there will be limited available mass available to trade between the dischargers especially as flows increase in the future. Notwithstanding, it is recommended that the New Hope dischargers pursue group compliance as a tool to assist each of the communities in meeting their nutrient wasteload allocations.

Figure 2-13

Required Annual Average Nutrient Concentrations – Jordan Lake Mass Limits



A future expansion to a permitted maximum month capacity of 18.5 mgd was identified in previous analyses and confirmed. Future TN and TP load allocations are not expected to be increased upon expansion of the Mason Farm WWTP, and the current mass load allocations correspond to TN and TP concentrations of 2.39 mg/L and 0.18 mg/L, respectively, at the future 18.5 mgd design flow. Note that this assumption does not address possible future additional, more stringent regulatory requirements. This report will address the ability of the existing facilities to meet the existing NPDES discharge limits and Jordan Lake nutrient reduction requirements through 2030. The ability to meet these objectives at the current permitted maximum month flow of 14.5 mgd will also be addressed.

Section 3.0
Hydraulic Capacity Analysis

3.1 Background Information

Hydraulic evaluations of the existing facilities were based on flow conditions presented in Table 3-1. Figure 3-1 shows the site plan for the existing Mason Farm WWTP facilities and major process piping.

Table 3-1
Design Flows

Flow Condition	Design Flow (mgd)	RAS Flow (mgd)
Annual Average Flow	10.35	10.35
Maximum Monthly Flow	14.5	14.5
Peak Wet Weather Flow	43.5	14.5

Design flow projections are discussed in detail in Section 2. Return activated sludge (RAS) flow is the rate at which settled solids are pumped (returned) from the secondary clarifiers to the head of the aeration basins. RAS pumping facilities are typically designed to return approximately 100% of the design plant influent flow which would equate to 14.5 mgd (the maximum month flow) for the Mason Farm WWTP. During normal operation, the RAS rate is often less than 100% of plant influent flow. Under peak flow conditions, secondary clarifier performance is enhanced if higher RAS rates are used. Because RAS is pumped from the secondary clarifiers to the head of the aeration basin, this portion of the plant must have adequate hydraulic capacity to handle the combined plant influent and RAS flows. According to the Mason Farm WWTP Process Control Plan, the current RAS pumping facilities have a maximum capacity of 18 mgd. However, for the purposes of this evaluation it is assumed that a peak RAS rate of 14.5 mgd will used under peak wet weather flow conditions.

3.2 Hydraulic Profile Calculations

Hydraulic profile calculations were performed using *HazenPro*, an Excel-based hydraulic model developed by Hazen and Sawyer specifically for treatment plant hydraulic profile analysis. Information on existing facilities was acquired from record drawings from past construction projects and equipment operation and maintenance manuals, and where possible, was verified in the field. Critical elevations were verified with a field survey.

Hydraulic profiles through a treatment plant are calculated beginning at the downstream end of the plant and work upstream, with the starting point being the water surface elevation in the receiving stream (Morgan Creek). Hydraulic profile calculations, which are included as Appendix D, consist of a series of calculations of head loss in each hydraulic element (piping, channels, gates, weirs, flumes, etc.), with the head loss in each element impacting the element immediately upstream. HazenPro consists of Excel spreadsheet modules for each type of hydraulic element which are linked together to form a continuous hydraulic profile. The theoretical bases for the HazenPro hydraulic profile calculations are as follows:

Piping: Head loss in piping consists of friction losses and minor losses. Friction losses are calculated using the Hazen-Williams equation. Hazen and Williams C-values are based on those typically found in pipes that have been in service for many years. Minor losses in pipes are due to bends, valves, entrance and exit losses, etc. Minor loss k-values used are typical published values.

Channels: Head loss in channels consists of friction losses and minor losses. Friction losses are calculated using the Manning equation. Friction losses depend on the roughness of the channel, as measured by the Manning "n" value. The calculation for friction loss in a channel is an iterative solution because the depth and velocity of flow varies along the length of the channel. Minor losses in a channel are calculated similar to those in a pipe.

Gates/Ports: Gates and ports can be submerged (typical for sluice gates) or have free surface flow (typical for a slide gate). Losses through gates and ports are calculated using the orifice equation, with orifice coefficients determined based on the geometry of the port.

Weirs: Weirs are often used to maintain a constant level in a process unit and can have different forms (sharp-crested, v-notch, etc). Weir equations vary based on the type of weir provided.

Flumes: Flumes are used to measure flow and, at several locations at the Mason Farm WWTP, to equally or proportionally distribute flow to process units. The Mason Farm WWTP has both parshall and cutthroat flumes. Flume flow characteristics have been established based on extensive lab testing, and equations have been derived which can be used in the spreadsheet calculations to determine hydraulic performance.

The equations used in the hydraulic calculations are widely accepted and have been in use for many years. However, the accuracy of the equations is impacted by the selection of values for the coefficients used in the model. Some of these coefficients, such as the Hazen and Williams C-value, change over time. A 20-yr old pipe will not have the same C as a new pipe due to deterioration of the pipe wall or biological growth on the pipe walls. Hazen and Sawyer uses conservative values for coefficients that are intended to predict performance when pipes, etc. are in a future impaired condition. Hydraulic constants and coefficients used for the Mason Farm WWTP profile are shown in detailed calculations in Appendix D.

A limited amount of model calibration was performed for the Mason Farm WWTP profile. Calibration involves taking water surface elevation measurements under known flow conditions and comparing the measurements to those predicted by the model. There are several factors which make it difficult to calibrate a hydraulic profile model:

- The conservative “future condition” friction coefficients used in the model as described above, do not always correspond well to current conditions.
- Head losses in a hydraulic profile are usually very small under average flow conditions. Higher flows are generally necessary to create enough head loss to determine the accuracy of a model.
- Pipes and channels often accumulate some solids under average flow conditions and the accumulated solids impact hydraulic characteristics. These solids are typically flushed under high flow conditions.
- At the Mason Farm WWTP, unit processes upstream of the aeration basins are covered for odor control, making it difficult to access many locations to check water surface elevations.

For the Mason Farm WWTP, the calibration effort was limited to those locations where there was significant uncertainty about the details of the hydraulic flow path. These locations are discussed in more detail later in this section.

3.3 Hydraulic Evaluation

The goal of the hydraulic evaluation is to identify hydraulic constraints or “bottlenecks” in the existing plant that may impact plant performance by submerging critical hydraulic control points or, in extreme cases, by overtopping process tank walls. Critical hydraulic control points are defined as those hydraulic elements where the water level is somehow fixed or those points that, if submerged, could lead to significant process upsets. For the Mason Farm WWTP, the critical hydraulic control points and their locations are shown in Table 3-2.

Table 3-2
Hydraulic Control Points

Hydraulic Control Point	Location (Facility)
Plant influent flow measurement parshall flume	Headworks
Primary clarifier influent distribution flumes	Splitter Box 1
Primary clarifier effluent weirs	Primary Clarifiers
Wetwell Level	Intermediate Pump Station
Mixed liquor flow distribution flumes	Aeration Basins
Secondary clarifier effluent weirs	Secondary Clarifiers
Filter influent flow distribution flumes	Effluent Filter Complex
UV effluent gates	Effluent Filter Complex
Morgan Creek level	Morgan Creek

The segment of the hydraulic profile between each set of hydraulic control points was analyzed to predict performance under the design flow conditions, to determine the maximum hydraulic capacity of the segment, identify bottlenecks in the existing unit processes and yard piping, and recommend improvements to allow the segment to reliably pass the proposed peak wet weather flow. As would be expected, no hydraulic constraints were identified under average or maximum month flow conditions. Under peak flow conditions, the segments were analyzed: 1) with all process units in service, and 2) with the largest unit out of service. The capacity of the segment with one unit out of service was evaluated for informational purposes only. It is expected that all process units would be placed in service should peak flow conditions arise. Profile segments are identified in Table 3-3, with the corresponding capacity with all process units in service. Segments are labeled A through I as they appear in the hydraulic profile calculations included in Appendix D.

Table 3-3
Hydraulic Profile Segments

Segment	Description	Hydraulic Capacity
J	Headworks Influent Channel to Influent Flow Measurement Flume	70 mgd
I	Influent Flow Measurement Flume to Splitter Box 1	56 mgd
H	Splitter Box 1 to Primary Clarifiers	43.5 mgd [1]
G	Primary Clarifier Effluent Weirs to Intermediate Pump Station	33.5 mgd [2]
F	Intermediate Pump Station	50 mgd (43.5 mgd firm)
E	Aeration Basins	51 mgd
D	Mixed Liquor Distribution Flumes to Secondary Clarifier Effluent Weirs	43.5 mgd [1]
C	Secondary Clarifier Effluent Weirs to Filter Influent Distribution Flumes	46 mgd
B	Filter Influent Distribution Flumes to UV Disinfection	52 mgd
A	UV Disinfection Effluent Gates to Outfall	70 mgd

1. Distribution flumes are submerged at 43.5 mgd peak hour flow
2. Maximum flow with no PC Bypass. With PC bypass, capacity is approximately 50 mgd

Individual segments are evaluated assuming there are no impacts from hydraulic constraints downstream. The assumptions used to determine peak flow capacity for each segment are discussed in the following sections.

3.3.1 Headworks Influent Channel to Influent Flow Measurement Flume

This segment includes the influent bar screens, grit collectors, and influent flow measurement flume. The three influent screens, each rated at 23.3 mgd, provide a firm capacity of 46.6 mgd and a total capacity of 69.9 mgd. The two grit collectors, each rated at 50 mgd, provide a firm capacity of 50 mgd and a total capacity of 100 mgd. The 5' wide parshall flume has a maximum capacity of 55 mgd. No hydraulic constraints were identified in the channels serving these units at 69.9 mgd.

3.3.2 Influent Flow Measurement Flume to Splitter Box 1

This segment includes the piping between the Headworks and Splitter Box No. 1(SB1) which consists of several parallel piping flow paths. This network of pipes has a capacity of approximately 56 mgd without submerging the upstream parshall flume, assuming a high water level (0.5 feet of freeboard) in SB1. A parshall flume may be submerged up to 70% without affecting its flow measurement accuracy. The capacity of the piping from the Headworks to SB1 exceeds the capacity of the piping downstream of SB1.

3.3.3 Splitter Box 1 to Primary Clarifiers

This segment includes the primary clarifier (PC) influent flow distribution flumes, the piping between SB1 and each clarifier, and the clarifier mechanisms. Operations and Maintenance (O&M) manuals showing detailed information on the PC influent piers and influent ports were not available for PC1 and PC2. Losses through these units were modeled based on information available on the original construction drawings and based on typical arrangements for this type of equipment. Hydraulic restrictions were identified in this segment. PC1 and PC3 have 24" influent lines which result in high head loss under peak conditions as compared to the 30" line serving PC2. PC3 also has a newer mechanism than the other units and, according to the O&M manual, is equipped with an energy dissipating inlet similar to the newer secondary clarifiers. The influent port configuration on this unit creates more head loss than a typical primary clarifier influent arrangement. The combined effect of the smaller influent piping and the energy dissipating inlet make the flow path to PC3 the worst case flow path for this segment of the profile.

With all units in service, the flow distribution flume for PC3 will begin to exceed its transitional submergence at a flow of approximately 37 mgd. At 37 mgd, the water level upstream of the flumes is within one foot of overtopping the wall of SB1. At 43.5 mgd, the submergence of the PC3 flume is not severe enough to significantly impact flow distribution, as summarized in Table 3-4 below. At peak flow the freeboard in SB1 is only about 6".

Table 3-4
Primary Clarifier Flow Distribution

Clarifier	Theoretical Flow Distribution	Predicted Flow Distribution
1	33.3%	34%
2	33.3%	34%
3	33.3%	32%

If one clarifier is out of service during peak flow conditions, a primary clarifier bypass of 17 mgd is required. It would be possible to modify the PC3 influent well configuration to minimize head loss, but little can be done to address the 24" influent piping located below the slab of PC1 and PC3.

3.3.4 Primary Clarifier Effluent Weirs to Intermediate Pump Stations

This segment includes the piping between the primary clarifiers and the intermediate pump stations (IPS1 and IPS2). Primary effluent pipes from PCs 1-3 combine in Splitter Box 2 (SB2). SB2 contains three weirs and gates which are capable of splitting flows to the IPS and the two abandoned trickling filters. The primary flow path is the 30" pipe between SB2 and the IPS. This 30" line is inadequate to carry the full 43.5 mgd peak flow and operators staff currently divert some flow through Trickling Filter 2 during high flow events. The gate to Trickling Filter 2 remains partially open to avoid stagnant wastewater in the trickling filter. The hydraulic model indicates that that the capacity of the 30" pipe is over 30 mgd.

If one PC is out of service during peak flow conditions, a partial diversion of flow through the trickling filter is not adequate to prevent submergence of the clarifier effluent weirs. Under these conditions, the bypass piping from the Headworks directly to the IPS, which has a capacity of approximately 17 mgd, must be utilized. The PC bypass line is isolated by a downward opening slide gate at the headworks facility. It is possible to leave this gate open to allow for an automatic bypass at higher flows. If the gate is fully open, a bypass would occur whenever the plant flow exceeds 30 mgd. It should be noted that the gate is located immediately downstream of the parshall flume and it is possible that a premature bypass would occur due to turbulence. Experimentation by plant staff is recommended to determine if this will be an issue.

It is recommended that piping modifications be made to parallel the existing 30" line such that flow does not need to be diverted to the abandoned trickling filter if OWASA would prefer not to route excess flows through the trickling filter structure. Assuming the existing trickling filter influent and effluent piping is in good condition, this piping could be modified to bypass the trickling filter and the lab building and provide a suitable parallel flow path. The final routing of these piping improvements should consider the locations of future facilities on the site. The hydraulic capacity of this flow path could also be increased by removing the weir wall in SB2. This weir performs no function since the trickling filters have been abandoned and the restriction it creates causes the PC effluent weirs to be submerged at lower flows. If these two modifications were made, this segment would have the capacity to pass the 43.5 mgd peak with one PC out of service; however, PC1 and PC2 effluent weirs would still be slightly submerged due to the limited capacity of the effluent launders.

3.3.5 Intermediate Pump Station

The Mason Farm WWTP has two Intermediate Pump Stations (IPS1 and IPS2). IPS 1 is equipped with three 50 horsepower (hp) pumps, each with a nominal capacity of 3850 gpm at 36' total dynamic head (TDH). IPS 2 is equipped with three 60 hp pumps, each with a nominal capacity of 3600 gpm at 46' TDH. In addition, three engine driven pumps (two Godwin pumps and one Griffin pump) have been installed adjacent to the existing wet wells. Each engine driven pump has a capacity of approximately 4500 gpm at 46' TDH. The combined capacity of the two stations and the engine driven pumps is approximately 50 mgd (43.5 mgd with the largest unit out of service) which is adequate for the peak flow condition.

3.3.6 Aeration Basins

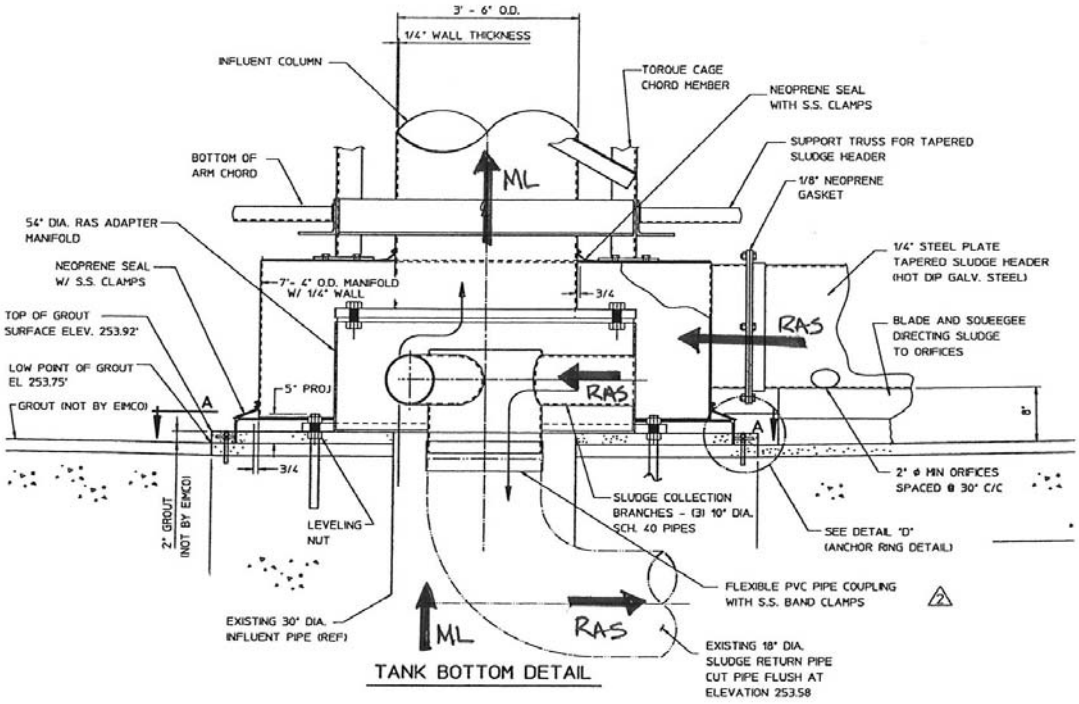
The aeration basins have the flexibility to operate in several different configurations. For this analysis, the basins were assumed to be operating in the 3:4 mode (three trains with four cells in series). In this mode a total mixed liquor flow of 66 mgd can be passed with all trains in service and 44 mgd with one train out of service. Assuming 14.5 mgd of RAS flow, this equates to a plant flow of 51.5 mgd and 29.5 mgd respectively. At these flow conditions there is only 6" of freeboard in the influent channel. The aeration basins will not see these flow conditions as the current design has provisions for bypassing primary effluent to cells 5A and 5B under peak flow conditions, providing relief not only hydraulically, but also from a process perspective. Appropriate use of this wet weather step feed (contact stabilization) capability is discussed in Section 4. The aeration basin influent channel has a large cross-sectional area and flow velocities in the channel are low (less than 0.5 feet per second at peak flow). It was not possible to confirm influent gate elevations by survey but, assuming the gates are all installed at the elevations indicated by the record drawings, flow distribution to the aeration basin trains is very good. Nutrified sludge is distributed to each train from a 30" header through individual 8" and 6" lines. This configuration allows for flexibility in distributing sludge based on the aeration basin process configuration. Capacity and flow distribution issues for this system do not impact the overall hydraulic capacity analysis; however, this system will be evaluated further in Section 4.

3.3.7 Mixed Liquor Distribution Flumes to Secondary Clarifier Effluent Weirs

This segment includes the mixed liquor distribution cutthroat flumes, secondary clarifier (SC) influent piping, and clarifier mechanisms. Under normal flow conditions (when not excessively submerged), the flumes provide for flow distribution to the SC's proportional to the throat width of the flume. A flume is submerged when the water level downstream of the flume backs up above the crest of the flume. A cutthroat flume of this size (9' length) may be submerged up to 80%, the "transitional submergence" level, without affecting its flow characteristics. If submerged above 80%, the flume will no longer provide for proportional flow distribution to the clarifiers.

Hydraulic constraints were identified in this segment including clarifier influent piping and clarifier mechanisms. The mechanisms for SC1, SC2, and SC3 have been modified from the original installations. Originally, these units were of the "organ pipe" type sludge removal mechanisms where the return activated sludge (RAS) withdrawal pipe runs within the influent column. When this type of unit is converted to the hydraulic header type removal system, the sludge withdrawal header must be tied into the RAS pipe located within the center column as shown in Figure 3-2.

Figure 3-2
SC1 RAS Withdrawal Modifications



This modification can result in a hydraulic restriction in the center column. An O&M manual showing details on this modification was available for SC1 but not for SC2 and SC3. SC2 and SC3 were modeled assuming a similar configuration to SC1, but without detailed drawings, this is provides only an estimate of actual performance. The influent piping to SC3 is longer than that serving SC2 and will restrict flow to this unit. During the field stress testing of the units, the effluent box downstream of the flume serving SC3 backed up significantly more than that for SC2, exceeding the transitional submergence of the flume at a mixed liquor flow of approximately 6 mgd. This corresponded well to the predictions of the hydraulic model.

With all units in service, the flume serving SC3 will begin to submerge when plant flow exceeds approximately 25 mgd. As flows continue to increase, the same submergence effect will be experienced at SC4, SC1, SC2, and SC5, in that order. When the flumes are submerged beyond the transitional submergence level, the mixed liquor will follow the path of least resistance. The SC with influent piping of larger diameter or shorter length will be favored. At the peak flow of 43.5 mgd, the flumes serving all clarifiers except SC5 are submerged. Table 3-5 compares the theoretical flow distribution based on unsubmerged flumes with the flow distribution with submerged flumes as predicted by the hydraulic model.

Table 3-5
Secondary Clarifier Flow Distribution at 43.5 mgd

Clarifier	Theoretical Flow Distribution	Predicted Flow Distribution
1	22.2%	22%
2	11.1%	12%
3	11.1%	9%
4	22.2%	21%
5	33.3%	36%

Under peak flow conditions, SC2 and SC5 will be loaded approximately 10% more heavily than the theoretical distribution and SC3 would be under loaded by nearly 20%. SC5 is the deepest of the 5 units and is better able to handle the higher loading; however the higher loading to SC2 (one of the shallowest units) could result in poor settling performance and solids loss from the unit. The mixed liquor piping to SC3 could be replaced or paralleled to increase flow to this unit and provide some relief to SC2. The potential need for improvements to SC flow distribution was reviewed further after completion of secondary clarifier computational fluid dynamics (CFD) modeling that is discussed in detail in Appendix H.

3.3.8 Secondary Clarifier Effluent Weirs to Filter Influent Distribution Flumes

This segment includes the individual secondary clarifier effluent pipes, secondary effluent header piping, and filter influent piping. With all secondary clarifiers in service, no hydraulic constraints were identified under peak flow conditions. However, if SC5 is out of service, other clarifier effluent weirs begin to submerge when flow exceeds 35 mgd, possibly resulting in solids loss from the clarifiers. Other units could be taken out of service with much less impact on peak flow capacity as summarized in Table 3-6.

Table 3-6
Secondary Effluent Piping Capacity

Secondary Clarifier Out of Service	Segment Capacity with Unit Out of Service
1	37 mgd
2	39 mgd
3	39 mgd
4	36 mgd
5	33 mgd

3.3.9 Filter Influent Flumes to Ultraviolet Disinfection

This segment includes the filter influent flumes, effluent filters, and filter bypass. Hydraulic calculations are based on one filter being in backwash mode. The Process Control Plan states that the peak flow capacity of each filter is 8.65 mgd, resulting in a filtration capacity of 52 mgd with all filters in service and 43.5 with one filter out of service. No hydraulic constraints were identified under these flow conditions. At flows greater than the rated capacity, the filter bypass gate would need to be opened to allow flow to bypass directly to the UV disinfection influent channel. In full open position, the gate can pass a peak flow of approximately 82 mgd.

3.3.10 Ultraviolet Disinfection to Outfall

This segment includes the UV effluent channel, post aeration tanks, and outfall piping to Morgan Creek. When Morgan Creek is at its normal water level of 247 feet, the gravity outfall piping has capacity to pass a peak flow of 101 mgd, and can pass 43.5 mgd when the water level in Morgan Creek is at elevation 251 (four feet above normal level). Under high flood conditions, the water level in Morgan Creek exceeds the design water levels in the post aeration basins and effluent pumping is required. Each effluent pump has a published capacity of 17.5 mgd, and the effluent pump station can pass 70 mgd with all units in service and 52.5 mgd with one unit out of service.

3.4 Summary

The attached figures summarize the constraints identified in the hydraulic evaluations under different flow scenarios. Figures are as follows:

Figure 3-3	Peak Flow, All units in Service, No Primary Clarifier Bypasses
Figure 3-4	Peak Flow, All units in Service, With Primary Clarifier Bypass
Figure 3-5	Peak Flow, Largest Unit out of Service, With Primary Clarifier Bypass
Figure 3-6	Hydraulic Profile, All units in Service, With Primary Clarifier Bypass

The hydraulic constraints identified include primary clarifier influent and effluent piping, the Splitter Box 2 weir, and secondary clarifier influent piping. The existing primary clarifier bypass line provides a flow path that relieves the constraints in the primary influent and effluent piping, and no further action related to the primary clarifier piping is required to pass the 43.5 mgd peak flow if use of this bypass piping is an acceptable approach for conveying intermittent wet weather peak conditions through the plant. Modifications described below in Table 3-7 would allow additional flow to pass through the primaries under peak flow conditions.

Table 3-7
Potential Hydraulic Improvements for Primary Clarifiers

Hydraulic Constraint	Recommended Improvement	Estimated Cost
PC Effluent Piping	Modify existing trickling filter piping downstream of Splitter Box 2 to provide parallel flow path	$62,000
Splitter Box 2	Remove weir wall to prevent PC weir submergence	$30,000

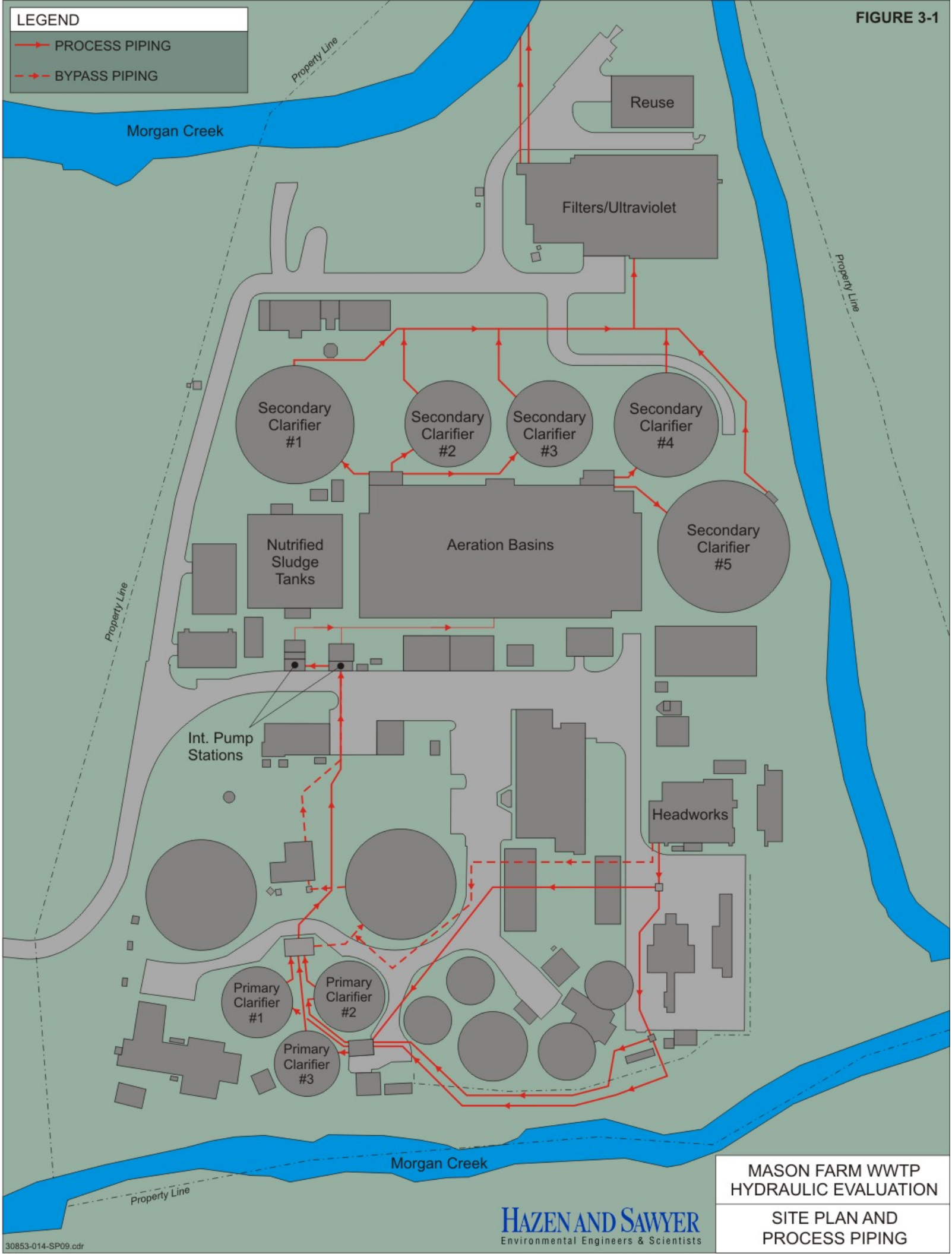
The hydraulic constraint with the greatest potential to affect the plant's treatment performance is related to the secondary clarifier influent piping. As described above, these hydraulic limitations affect flow distribution to the secondary clarifiers and result in overloading of SC2, one of the shallowest clarifiers. The CFD analysis of the secondary clarifiers indicates that a significant increase in loadings to SC2 due to poor flow distribution would be problematic. Parallel piping to SC3 could improve the flow distribution between SC2 and SC3 to improve performance of SC2 under peak flow conditions. However, it is recommended that investment in this modification, the cost of which is estimated in Table 3-8 below, could be deferred until the peak flows are nearing the design peak flow of 40 mgd. Flow distribution to the secondary clarifiers should be given a higher priority than other potential hydraulic improvements because of its greater impact on treatment performance.

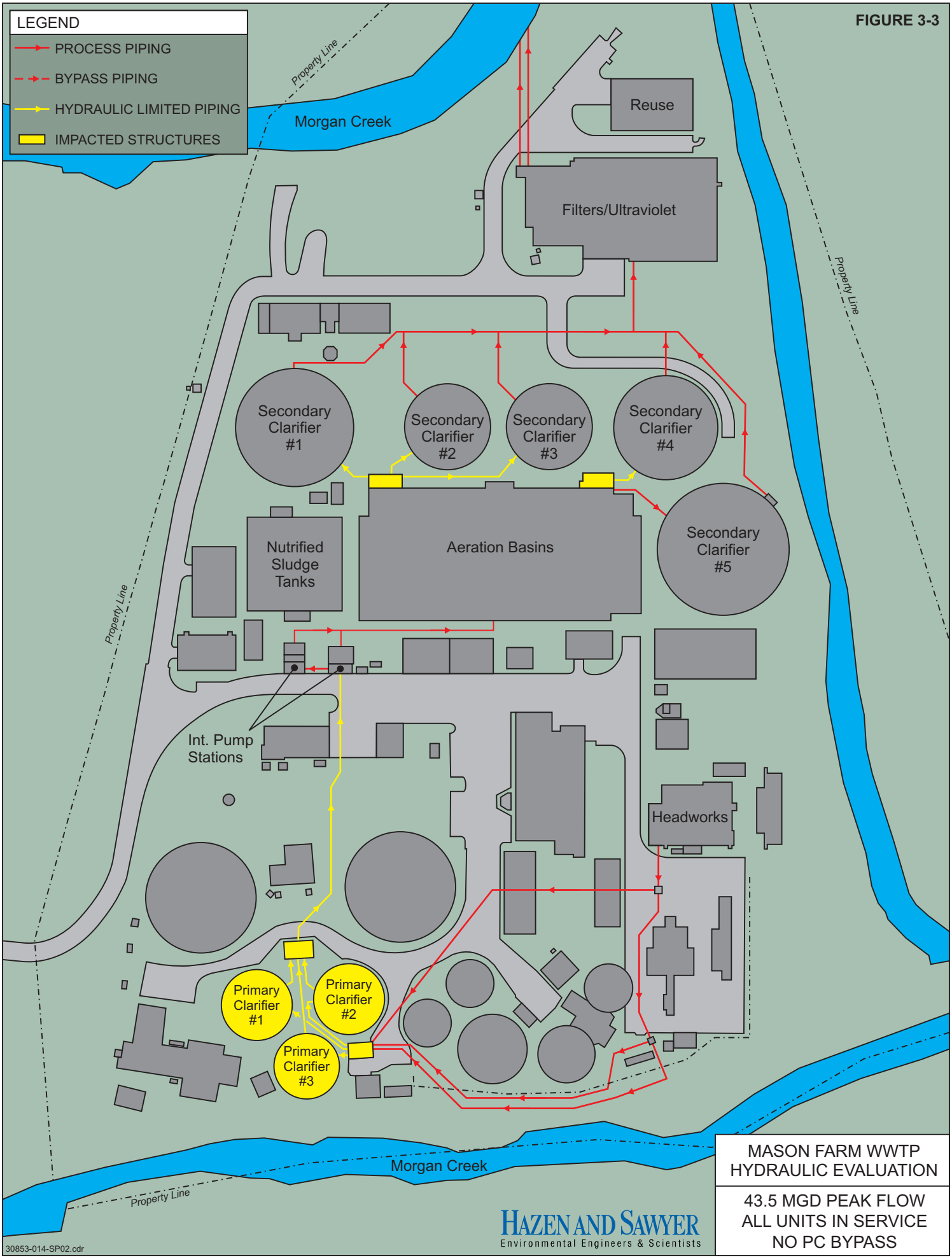
Table 3-8
Potential Hydraulic Improvements For Secondary Clarifiers

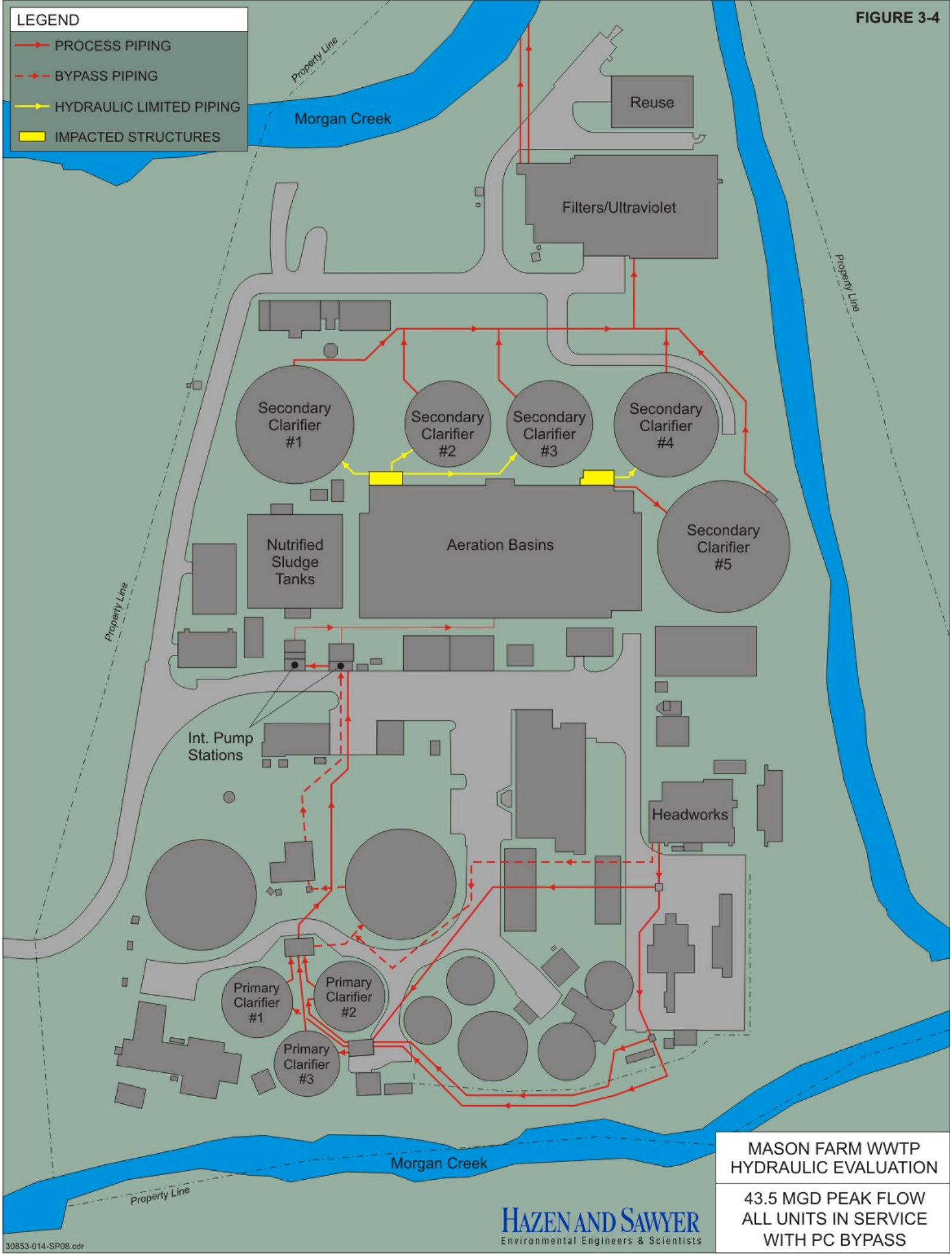
Hydraulic Constraint	Recommended Improvement	Estimated Cost
SC Influent Piping	Replace or parallel influent piping to SC3	$48,000

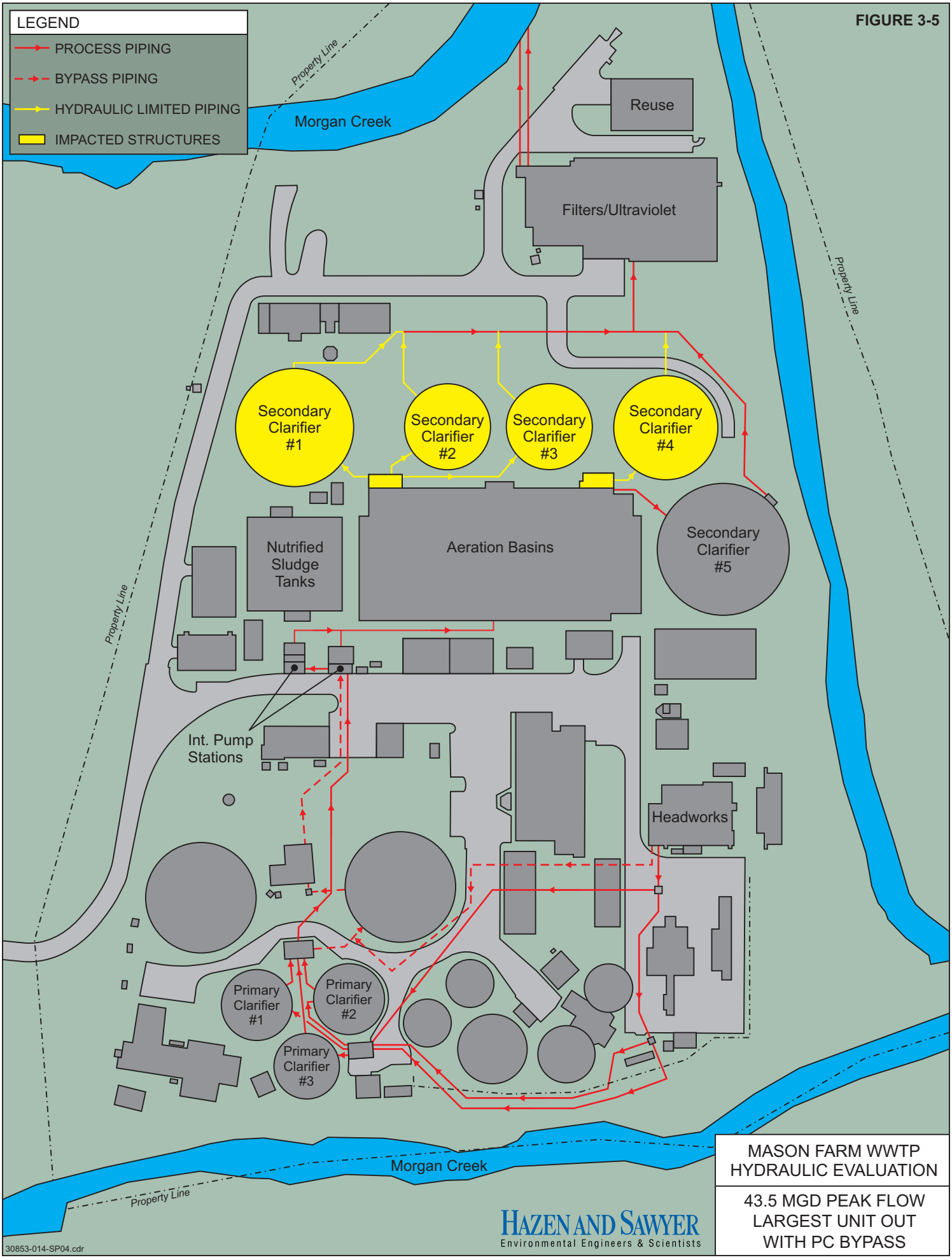
The hydraulic capacity analysis also determined that peak capacity through the plant would be limited based on the capacity of the secondary clarifier effluent piping. Depending upon which secondary clarifier is out of service, the peak hydraulic capacity would be limited to between 29 and 39 mgd. Investments in improvements to the secondary clarifier effluent piping are likely not warranted since, in the event of a 43.5 mgd peak flow event an out of service clarifier could be placed into service to eliminate this hydraulic constraint.

Since the existing facilities have the capacity to convey 43.5 mgd through the Mason Farm WWTP, none of the recommended hydraulic improvements are proposed to be completed at this time. If the need to convey peaks higher than 43.5 mgd develops in the future or if conveyance of all flows through the primary clarifiers becomes important, the proposed hydraulic improvements should be addressed at that time.

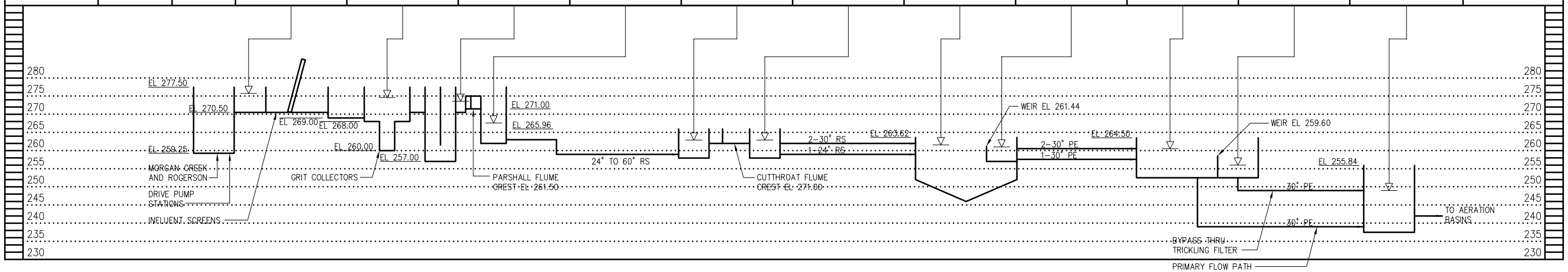

LEGEND
PROCESS PIPING
BYPASS PIPING
FIGURE 3-1
Morgan Creek
Property Line
Reuse
Filters/Ultraviolet
Secondary Clarifier #1
Secondary Clarifier #2
Secondary Clarifier #3
Secondary Clarifier #4
Secondary Clarifier #5
Nutrified Sludge Tanks
Aeration Basins
Int. Pump Stations
Headworks
Primary Clarifier #1
Primary Clarifier #2
Primary Clarifier #3
Morgan Creek
Property Line
MASON FARM WWTP
HYDRAULIC EVALUATION
SITE PLAN AND
PROCESS PIPING
HAZEN AND SAWYER
Environmental Engineers & Scientists
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LEGEND
PROCESS PIPING
BYPASS PIPING
HYDRAULIC LIMITED PIPING
IMPACTED STRUCTURES
FIGURE 3-3
Property Line
Morgan Creek
Reuse
Filters/Ultraviolet
Property Line
Secondary Clarifier #1
Secondary Clarifier #2
Secondary Clarifier #3
Secondary Clarifier #4
Secondary Clarifier #5
Nutrified Sludge Tanks
Aeration Basins
Property Line
Int. Pump Stations
Headworks
Primary Clarifier #1
Primary Clarifier #2
Primary Clarifier #3
Morgan Creek
Property Line
MASON FARM WWTP
HYDRAULIC EVALUATION
43.5 MGD PEAK FLOW
ALL UNITS IN SERVICE
NO PC BYPASS
HAZEN AND SAWYER
Environmental Engineers & Scientists
30853-014-SP02.cdr

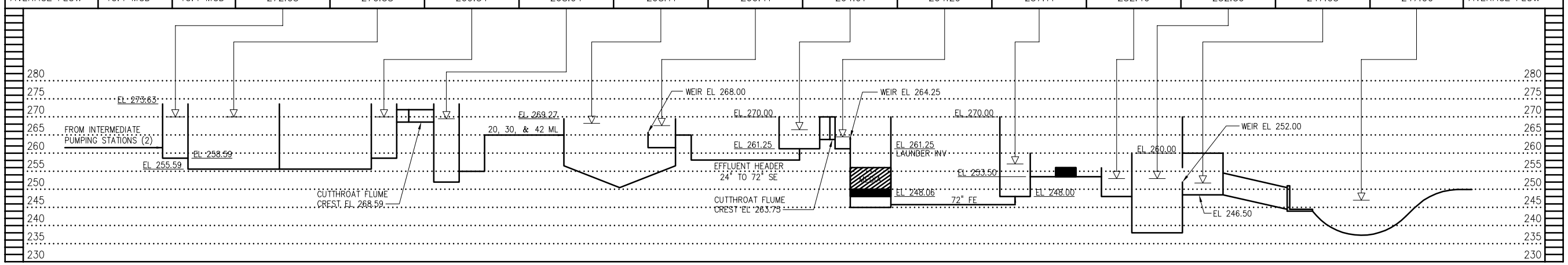

LEGEND
PROCESS PIPING
BYPASS PIPING
HYDRAULIC LIMITED PIPING
IMPACTED STRUCTURES
FIGURE 3-4
Property Line
Morgan Creek
Reuse
Filters/Ultraviolet
Secondary Clarifier #1
Secondary Clarifier #2
Secondary Clarifier #3
Secondary Clarifier #4
Secondary Clarifier #5
Nutrified Sludge Tanks
Aeration Basins
Int. Pump Stations
Headworks
Primary Clarifier #1
Primary Clarifier #2
Primary Clarifier #3
Morgan Creek
Property Line
MASON FARM WWTP
HYDRAULIC EVALUATION
43.5 MGD PEAK FLOW
ALL UNITS IN SERVICE
WITH PC BYPASS
HAZEN AND SAWYER
Environmental Engineers & Scientists
30853-014-SP08.cdr


LEGEND
PROCESS PIPING
BYPASS PIPING
HYDRAULIC LIMITED PIPING
IMPACTED STRUCTURES
FIGURE 3-5
Property Line
Morgan Creek
Reuse
Filters/Ultraviolet
Property Line
Secondary Clarifier #1
Secondary Clarifier #2
Secondary Clarifier #3
Secondary Clarifier #4
Secondary Clarifier #5
Nutrified Sludge Tanks
Aeration Basins
Property Line
Int. Pump Stations
Headworks
Primary Clarifier #1
Primary Clarifier #2
Primary Clarifier #3
Morgan Creek
Property Line
MASON FARM WWTP
HYDRAULIC EVALUATION
43.5 MGD PEAK FLOW
LARGEST UNIT OUT
WITH PC BYPASS
HAZEN AND SAWYER
Environmental Engineers & Scientists
30853-014-SP04.cdr

CONDITION	PLANT FLOW	RAS FLOW	HEADWORKS		INFLUENT FLUME		SPLITTER BOX 1		PRIMARY CLARIFIERS		SPLITTER BOX 2		INTERMEDIATE PUMP STATION	CONDITION
PEAK FLOW	43.5 MGD	14.5 MGD	275.60	274.41	273.49	268.12	264.77	263.67	261.64	261.24	260.56	259.90	249.00	PEAK FLOW
MAX MONTH FLOW	14.5 MGD	14.5 MGD	273.91	272.71	272.23	264.41	263.41	261.98	261.58	260.31	260.12	251.17	249.00	MAX MONTH FLOW
AVERAGE FLOW	10.4 MGD	10.4 MGD	273.45	272.36	271.99	263.56	263.03	261.77	261.57	260.12	260.02	250.13	249.00	AVERAGE FLOW



CONDITION	PLANT FLOW	RAS FLOW	AERATION BASINS				SECONDARY CLARIFIERS		EFFLUENT FILTERS		UV DISINFECTION		POST-AERATION		MORGAN CREEK	CONDITION
PEAK FLOW	43.5 MGD	14.5 MGD	272.97	271.79	270.17	269.99	268.21	267.60	266.61	264.47	257.12	253.16	253.12	N/A	262.00	PEAK FLOW
MAX MONTH FLOW	14.5 MGD	14.5 MGD	272.28	270.87	269.52	268.83	268.11	266.39	265.18	264.31	257.11	252.50	252.50	251.75	251.00	MAX MONTH FLOW
AVERAGE FLOW	10.4 MGD	10.4 MGD	272.05	270.58	269.54	268.94	268.11	266.41	264.91	264.29	257.11	252.40	252.39	247.03	247.00	AVERAGE FLOW



NOTE: PEAK FLOW PROFILE UPSTREAM OF INTERMEDIATE PUMP STATION ASSUMES 10 MGD PC BYPASS

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MASON FARM WWTP

EXISTING PLANT
HYDRAULIC PROFILE

PLOT DATE 09/03/09 9:53am FILE= O:\30853\30853-014\drawings\Figures\Hydraulic Profile Mason Farm by BGOODWIN XREF FILE = NONE

Section 4.0
Liquid Stream Facilities Evaluation

4.1 Current Plant Performance

Historical effluent data from September 2008 through August 2009 were evaluated to assess current plant performance (Table 4-1). Plant effluent data received reveals generally excellent plant performance, especially in regards to $cBOD_5$ and TSS. Full nitrification and ammonia removal were generally observed. There were a few periods of increased effluent ammonia; however the current permit limits were met through the period evaluated. Effluent nitrate/nitrite concentrations indicate some denitrification at the facility; however, additional denitrification will be required to meet the mass load allocations associated with the Jordan Lake nutrient management strategy. Effluent total phosphorus concentrations were variable and indicate potential inefficiencies of the biological phosphorus removal process.

Table 4-1
Effluent Quality – September 2008 – August 2009

	Flow (mgd)	$cBOD_5$ (mg/L)	TSS (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	NO_x-N (mg/L)	TN (mg/L)	TP (mg/L)
September 2008	9.02	<2.0	1	0.47	1.21	9.1	10.3	0.09
October 2008	7.29	<2.0	1	<0.10	1.41	12.3	13.7	0.21
November 2008	7.23	<2.0	1	<0.10	1.05	13.9	14.9	0.65
December 2008	7.75	<2.0	1	<0.10	1.61	11.9	13.5	0.48
January 2009	7.72	<2.0	<1	<0.10	1.64	14.9	16.5	1.12
February 2009	7.62	<2.0	<1	0.46	1.12	15.3	16.5	0.15
March 2009	8.75	<2.0	<1	0.17	1.29	9.3	10.6	0.25
April 2009	7.26	<2.0	<1	<0.10	1.35	13.9	14.9	0.27
May 2009	6.23	<2.0	<1	<0.10	0.92	14.6	15.5	0.15
June 2009	6.40	<2.0	<1	<0.10	0.82	12.8	13.6	0.10
July 2009	5.58	<2.0	<1	<0.10	0.90	15.1	16.0	0.13
August 2009	5.88	<2.0	<1	<0.10	1.23	13.2	14.4	0.72
Average	**7.23**	**< 2.0**	**1**	**0.13**[1]	**1.21**	**13.0**	**14.2**	**0.36**
Current Permit Limit	**14.5**	**4.0**	**30**	**1**	**Report**	**Report**	**Report**	**Report**

[1]Assumes values less than reported limit are 50% reportable limit

4.2 Liquid Stream Unit Process Capacity Evaluation

The capacity of the liquid stream facilities at the Mason Farm WWTP is evaluated in the following section. Major unit processes that were considered include:

- Preliminary Treatment
- Primary Treatment
- Intermediate Pumping
- Secondary Treatment
- Tertiary Filters
- UV Disinfection
- Post Aeration
- Effluent Disposal
- Chemical Storage and Feed Facilities

4.3 Preliminary Treatment

4.3.1 Influent Screening

The influent screens are located at the headworks facility, which receives raw sewage from Morgan Creek and Rogerson Drive Pump Stations. Three continuous, self-cleaning type screens are currently installed and a fourth channel is available for a future bar screen installation. Each screen has a dedicated screenings washer/press discharging to a common belt conveyor, which transports the compacted screenings to a dumpster for removal. Each screen is rated at a peak flow capacity of 23.3 mgd, providing a firm capacity of 46.6 mgd and a total capacity of 69.9 mgd. Influent screening design criteria is summarized in Table 4-2.

Table 4-2
Influent Screening Design Criteria

Mechanical Screens	
Number of units	3
Width (ft)	4
Bar Spacing (in)	¼
Peak Flow Per Screen (mgd)	23.3
Screenings Presses	
Number of Units	3

4.3.2 Grit Removal

Grit collection, pumping and dewatering equipment are also located at the headworks. Screened wastewater passes through two stirred-vortex grit collectors, each rated at 50 mgd, providing a firm capacity of 50 mgd and a total capacity of 100 mgd. Each grit collector has a dedicated grit pump to pump the grit slurry from the grit collector sump to a grit cyclone and classifier for dewatering. The equipment is configured such that each grit collector, pump, cyclone, and classifier form a single train of equipment, with no flexibility for using any piece of equipment with the other "train". Although this configuration minimizes potential for clogging at fittings and valves by simplifying piping, it also increases the potential for both trains being unavailable at the same time. Table 4-3 presents the grit removal system design criteria.

Table 4-3
Grit Removal Design Criteria

Grit Collectors	
Number of units	2
Capacity per Unit (mgd)	50
Grit Pumps	
Number of Units	2
Grit Cyclones	
Number of Units	2
Grit Classifiers	
Number of Units	2

4.3.3 Influent Flow Measurement

A Parshall flume is provided for influent flow measurement downstream of the grit collectors. The flume has a throat width of 5 feet, which provides for measurement of flows up to 55 mgd. The existing flume is "nested" in a larger 7-foot flume. At some point in the future, if peak flows are expected to exceed 55 mgd, the 5-foot flume can be permanently removed. The 7-foot flume would provide for measurement of flows up to 78 mgd.

4.4 Primary Treatment

The Mason Farm WWTP has three 70-foot diameter primary clarifiers that receive flow from the headworks facility. The primary influent is evenly distributed to the clarifiers by cutthroat flumes in Splitter Box 1. Primary clarifiers are typically designed for surface overflow rates of 1,000 gallons per day per square foot (gpd/sf) at average flow conditions and 3,000 gpd/sf at peak hour flow conditions. At these overflow rates, the three primary clarifiers have a capacity of 11.5 mgd at average flow and 34.5 mgd at peak flow. As described in Section 3, it is possible to pass more than 34.5 mgd through these units, but solids and BOD removal efficiencies may be reduced, increasing the loading to the secondary treatment process. Since influent concentrations will be dilute during peak flows, and the occurrence of flows over 34.5 mgd is expected to be infrequent, this limitation is not expected to significantly impact the performance of downstream process units or plant effluent quality, and no improvements are recommended.

The plant can additionally utilize the primary clarifier bypass piping which routes flow directly from the headworks to the intermediate pump station. Capacity of the bypass piping is approximately 17 mgd. Under peak flow conditions, the plant can pass more than 50 mgd of flow through or around the primary clarifiers. Table 4-4 summarizes the existing primary clarifiers.

Table 4-4
Primary Clarifier Design Criteria

Primary Clarifiers	
Number of Units	3
Inside Tank Diameter (ft)	70
Side Water Depth (ft)	13.25 (Clarifiers 1 & 2) 9.25 (Clarifier 3)
Tank Bottom Slope (inches/ft)	1

4.5 Intermediate Pump Stations 1 and 2

The two intermediate pump stations (IPS1 and IPS2) receive flow from the primary clarifiers, primary clarifier bypass and from other plant drains. All flow enters the wetwell of IPS2 and a connecting pipe carries flow to IPS1. IPS1 is equipped with three electrically powered pumps and one diesel engine driven pump installed adjacent to the wetwell. IPS2 has three electrically powered pumps and two diesel engine driven pumps. The firm capacity of the two stations is 43.5 mgd and the total capacity is 50 mgd. Tables 4-5 and 4-6 present the IPS1 and IPS2 design criteria, respectively.

Table 4-5
Intermediate Pump Station No. 1 Design Criteria

Electric Motor-Driven Pumps	
Number	3
Horse Power (hp)	50
Flow (gpm)	3,850
TDH (feet)	36
Diesel Engine-Driven Pumps	
Number	1
Flow (gpm)	4,500
TDH (feet)	46

Table 4-6
Intermediate Pump Station No. 2 Design Criteria

Electric Motor-Driven Pumps	
Number	3
Horse Power (hp)	60
Flow (gpm)	3,600
TDH (feet)	46
Diesel Engine-Driven Pumps	
Number	2
Flow (gpm)	4,500
TDH (feet)	46

4.6 Secondary Treatment

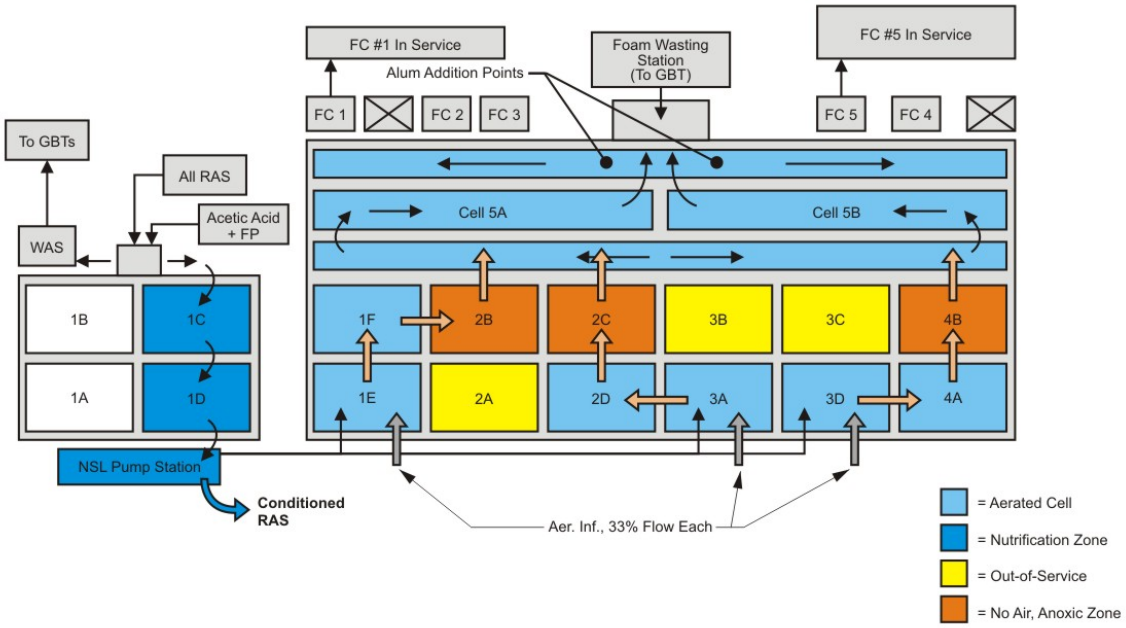
The secondary treatment facilities at the Mason Farm WWTP include the following: aeration basins, secondary clarifiers, nutrified sludge cells, return activated sludge (RAS) and waste activated sludge (WAS) pumping and aeration equipment for providing oxygen to the biological process. Primary effluent flows to the aeration basin influent channel, where it can be distributed to any of six different aeration cells. The aeration basins can be operated in numerous configurations, which are explained later in this section. The aeration basins provide for oxidation of organic matter and ammonia. The aeration basins can be operated to provide for a degree of denitrification by providing unaerated operation of one or more cells.

Effluent from the aeration basins is distributed to one of five secondary clarifiers. The secondary clarifiers settle the mixed liquor suspended solids (MLSS) produced in the activated sludge process, and clarified effluent is discharged over peripheral weirs to the downstream process units. Settled solids are returned to the nutrified sludge (NSL) cells by the RAS pumps. A portion of the settled solids (WAS) is wasted to the solids handling facility. The NSL cells provide for denitrification of the RAS and the required anaerobic volume to promote biological phosphorus removal (BPR). Effluent from the NSL cells is pumped directly to the aeration basins downstream of the influent channel.

Sodium hydroxide is fed to the primary effluent at the intermediate pump station (IPS) to provide alkalinity to the aeration basins to reduce pH inhibition of nitrification. Alum and polymer can be fed prior to the secondary clarifiers to precipitate/adsorb particulate phosphorus and aid settleability, respectively. Acetic acid and gravity belt thickener overflow (including primary sludge fermentate) are fed to the NSL cells to provide carbon for denitrification and phosphorus release.

Figure 3-3 presents a schematic of the aeration basin and NSL cells operation in August 2009. The secondary process has been operated in a similar configuration since October 2008. The individual activated sludge processes are discussed in the following sections.

Figure 4-1
Activated Sludge System Operation – August 2009



4.6.1 Nutrified Sludge (NSL) Cells

The NSL cells provide for RAS denitrification and biological phosphorus release. There are four cells, each with a capacity of approximately 275,000 gallons each. Two cells (Cell 1C and 1D) are currently in operation. Jet mixing is provided to mix the reactor contents.

RAS is pumped to the NSL influent channel. In addition gravity belt thickener (GBT) filtrate and acetic acid are fed to the NSL influent. The GBTs currently process a blend of fermented primary sludge and WAS. The filtrate contains volatile fatty acids (VFAs) and other readily biodegradable carbon (rbCOD) formed in the primary sludge fermenter. These VFAs are required for biological phosphorus removal (BPR), and the polyphosphate accumulating organisms (PAOs) uptake VFAs (in particular acetic and propionic acids) and release soluble phosphorus under anaerobic conditions. The released soluble phosphorus is assimilated by the poly-phosphate accumulating organisms under aerobic conditions (luxury phosphorus uptake). Fermented primary sludge is a particularly good source of VFAs for BPR since it provides a mixture of acetic and propionic acids. In contrast, feeding acetic acid in the absence of propionic acid

may promote the growth of glycogen accumulating organisms (GAOs), which compete with the PAOs for VFAs and may inhibit biological phosphorus removal.

Since the Mason Farm WWTP currently provides for limited denitrification, appreciable nitrate is returned to the NSL cells in the RAS. RAS nitrate concentrations observed during special sampling ranged from 10 mg/L to 19 mg/L. Heterotrophic denitrifying organisms will compete with the PAOs for available VFAs in the presence of nitrate. Nitrate is reduced in the NSL cells, and the remaining VFAs are utilized by the PAOs for biological phosphorus release. It is imperative that sufficient carbon and retention time be provided to fully denitrify within the upstream NSL cell to allow for the presence of VFAs and biological phosphorus release in the downstream NSL cell. In addition, it is necessary to limit the return of dissolved oxygen (DO) in the RAS to provide for anoxic/anaerobic conditions throughout the NSL cells and limit supplemental carbon feed requirements.

NSL cell effluent can be pumped to any of the western aeration basin cells (1E, 2A, 2D, 3A, 3D, 4A). The NSL pump station includes four pumps rated at 6 mgd each for a total firm capacity of 18 mgd. The NSL cells can provide for a total retention time of 1.5 hours based on the maximum firm RAS capacity of 18 mgd. Typical retention times will be much longer since this RAS capacity may not be fully utilized for many years. The NSL cells have sufficient volumetric capacity to provide for RAS denitrification and biological phosphorus release through a maximum month plant flow of 18 mgd provided sufficient supplemental carbon is available.

4.6.2 Aeration Basins

The aeration basin at the Mason Farm WWTP is divided into several sections as summarized in Table 4-7.

Table 4-7
Aeration Basin Configuration and Design Criteria

Name of Tank	Approximate Dimensions (L x W x SWD)	Volume (gallons)	Design Peak Hydraulic Capacity[1] (mgd)
Influent Channel	273 ft x 7 ft x 10.5 ft	150,000	80+
Aeration Cells (12 cells total)	50 ft x 50 ft x 14.1 ft per cell	275,000 each 3,300,000 total	42[2]
Effluent Channel 1	283 ft x 7 ft x 8.3 ft	125,000	100+
Cells 5A/5B	143 ft x 15 ft x 13.7 ft per cell	220,000 each 440,000 total	50+
Effluent Channel 2	264 ft x 7 ft x 8.3 ft	115,000	100+

[1]Per the Mason Farm WWTP Process Control Plan. Includes RAS flow.

[2]Primary effluent is fed directly to Cells 5A/5B at higher flows.

Primary effluent is fed to the influent channel, which can distribute flow to any of the six westernmost cells (1E, 2A, 2D, 3A, 3D, 4A). Nutrified sludge tank effluent is pumped into these cells. The mixed liquor suspended solids (MLSS) can be routed through a combination of additional cells before passing into Effluent Channel 1. Flow from Effluent Channel 1 flows to Cells 5A and 5B, which discharge to Effluent Channel 2. Effluent Channel 2 distributes flow to the five secondary clarifiers. Primary effluent can also be fed directly to Cells 5A and 5B.

Four basic modes of operation are detailed in the Process Control Plan (PCP) as follows:

1. 4:3 mode – Up to four (4) trains of three (3) cells each
2. 6:2 mode – Up to six (6) trains of two (2) cells each
3. 3:4 mode – Up to three (3) trains of four (4) cells each
4. High Flow mode – Provides step feed of primary effluent to Cells 5A and 5B when influent flows exceed 28 mgd

The Mason Farm WWTP is currently operated in the 4:3 mode, with only three (3) trains of the available four trains in service and configured with three (3) cells in each train. The third cell of each train is unaerated to provide anoxic volume for denitrification. Cells 5A and 5B are operated as a common cell under current operations. The capacity of the aeration basins is evaluated in the Secondary Process Evaluation subsection of this section.

4.6.3 Aeration System

Aeration and mixing of the 12 Aeration Cells and Cells 5A and 5B are provided through jet aeration. Table 4-8 summarizes the air flow and liquid pumping design capacities of the jet aeration equipment in each cell as provided in the Mason Farm WWTP Process Control Plan.

Table 4-8
Jet Aeration System Design Criteria

Aeration Cell	Air Flow Capacity (scfm) per cell	Liquid Flow Capacity (gpm) per cell
2A, 2D, 3A, 3D	4,200 (2,700 with control valves limiting flow)	5,300 per pump (10,600 total)
1E, 4A	2,700	5,300 per pump (10,600 total)
1F, 4B	750 per pod (1,500 total)	1,500 per pump (3,000 total)
2B, 2C, 3B, 3C	750	1,500
Cell 5A, Cell 5B	1,500	5,300

Aeration of the influent and effluent channels is provided by coarse bubble diffused aeration. Table 4-9 summarizes the design airflows for each channel.

Table 4-9
Channel Aeration Design Criteria

Channel	Air Flow Capacity (scfm)
Influent Channel	300 - 900
Effluent Channel 1 - North	150 - 450
Effluent Channel 1 - South	150 - 450
Effluent Channel 2 - North	150 - 450
Effluent Channel 2 - South	150 - 450

Five multistage centrifugal blowers provide air to the jet aeration headers located in the aeration cells. A separate blower provides for channel aeration. In addition a digester gas engine-driven blower can provide air to the aeration cells using methane generated during anaerobic digestion.

Existing blower capacity is summarized in Table 4-10. The capacity per blower as stated in the Process Control Plan was used for the preliminary evaluation of aeration capacity. Subsequent analysis of the existing blower system indicates that the existing aeration basin blowers have greater aeration capacity than indicated in the Process Control Plan. The revised blower capacity is included in Table 4-10 for comparison and is the basis for the capacity evaluation of the aeration system.

Table 4-10
Existing Blower Capacity

Blower Number	Horsepower per Blower	Process Control Plan Capacity per Blower (scfm)	Revised Capacity per Blower (scfm)
1, 2, 3	150	3,000	3,850
4, 5	250	5,000	5,800
6 (Channel Aeration)	250	5,000	-
Gas Engine Blower	-	7,000	-

Aeration of each cell can be controlled through an automated control valve. An airflow meter and dissolved oxygen probe is provided at each cell to allow aeration of each cell to be controlled based on DO concentration or airflow rate.

The capacity of the existing aeration system including the blowers and jet aeration equipment was evaluated based on the following criteria and assumptions:

- Maximum month primary effluent BOD_5 and TKN concentrations of 124 mg/L and 40 mg/L, respectively, at current wastewater strength based on the historical data analysis presented in Section 2 and simulated primary clarifier performance.

- Maximum month primary effluent BOD_5 and TKN concentrations of 185 mg/L and 59 mg/L, respectively, at increased wastewater strength based on the projected future worst case increase in influent concentrations presented in Section 2 and simulated primary clarifier performance.

- A peak day to maximum month load ratio of 1.8 was used for BOD_5 and TKN based on the 2008/2009 historical $cBOD_5$ load ratios.

- A jet aerator nozzle submergence of 9.9 feet was assumed based on a 14.1 foot sidewater depth, liquid header centerline depth of five feet from the basin floor (per the 14.5 mgd expansion drawings) and a 10-inch vertical distance between the liquid header centerline and nozzle tip, which assumes a 20° nozzle angle downward.

- An alpha value of 0.64 was assumed based on typical wastewater to clean water efficiency ratios for jet aeration.

- A standard oxygen transfer efficiency of 1.35% per foot submergence was assumed and is derived from field evaluation data reported in the literature[1,2], which ranged from 1.0% per foot to 1.7% per foot.

- A maximum temperature of 26°C was assumed based on the historical record.

- A dissolved oxygen (DO) concentration of 2 mg/L would be maintained under all conditions with the exception of peak day aeration demands, where DO would be allowed to drop as low as 0.5 mg/L.

[1] Mueller, J.A.; Boyle, W.C.; Popel, H.J. Aeration: Principles and Practice.CRC Press. 2002

[2]Water Pollution Control Federation Manual of Practice FD-13: Aeration. WPCP, ASCE. 1988.

The Mason Farm WWTP has a total installed aeration blower capacity of 23,150 scfm, excluding channel aeration and the backup gas engine blower, based on the revised blower capacities. Inclusion of the backup gas engine blower increases the capacity to 30,150 scfm. Based on the above capacities the Mason Farm WWTP aeration system is capable of delivering 36,300 lbs of oxygen per day to the process under maximum month conditions and 44,700 lbs of oxygen per day under peak day conditions. Operation of the methane gas engine blower increases the peak day capacity to 58,300 lbs of oxygen per day.

The existing aeration system at the Mason Farm WWTP has a capacity to treat 9.5 mgd of flow on a maximum month basis assuming current wastewater strength. This capacity would be reduced to 6.4 mgd under the increased wastewater strength condition discussed in Section 2. These capacities are based on meeting the peak day aeration requirement with all five aeration blowers in service and the gas engine blower out of service. The limited aeration capacity available at the Mason Farm WWTP is corroborated by the observation that four blowers (three small and one large, representing 74% of the total blower capacity excluding the gas engine-driven blower) were in operation when supplemental sampling was performed and influent wastewater flows were only averaging 7.6 mgd during that period.

Required aeration capacities for the various design conditions simulated during the process evaluation are summarized in Table 4-11 and assume that the existing jet aeration continues in service. The existing aeration system is not expected to meet the peak day air demands required by any of the design conditions, and improvements to the existing aeration system will be required in the near term, well in advance of the Year 2030.

Table 4-11
Estimated Aeration Requirements – Jet Aeration

Condition	Maximum Month Demand (scfm)	Peak Day Demand (scfm)
Year 2030 Expected Flows (13.0 mgd) Current WW Strength	21,600	31,600
Year 2030 Higher Flows (14.1 mgd) Current WW Strength,	23,500	34,200
Design Flow (14.5 mgd) Current WW Strength	24,100	35,200
Year 2030 Expected Flows (13.0 mgd) Increased WW Strength	32,100	46,700
Year 2030 Higher Flows (14.1 mgd) Increased WW Strength	34,800	50,700
Design Flow (14.5 mgd) Increased WW Strength	35,800	52,100

4.6.4 Secondary Clarifiers

MLSS is distributed from Effluent Channel 2 through Cutthroat flumes to the five secondary clarifiers. All five secondary clarifiers are circular center feed; however, only Clarifiers 2 and 3 are similar in size and internal configurations. The basic dimensions and RAS flow capacity for the five secondary clarifiers are summarized in Table 4-12.

Table 4-12
Secondary Clarifier Basic Dimensions and RAS Flow Capacity

Parameter	Clarifier 1	Clarifiers 2 and 3	Clarifier 4	Clarifier 5
Diameter (ft)	120	85	110	142.3
Side Water Depth (ft)	13.0	13.0	19.0	17.8
Bottom Slope (%)	2.1	2.1	2.1	2.1
EDI Diameter (ft)	15.0	None	18.0	16.4
Influent Feed Well Diameter (ft)	30.0	9.0	28.0	32.0
Influent Feed Well Depth (ft)	6.0	3.0	9.0	8.0
Effluent Launder	Outboard	Outboard	Inboard@ 13-ft from end wall	Inboard@ attached to end wall
Sludge Withdrawal	Suction/ TowBro	Suction/ TowBro	Suction/ Draft Tubes	Suction/ TowBro
Density Current Baffle	Stamford (Crosby) Baffle	Stamford (Crosby) Baffle	None	Stamford (Crosby) Baffle
RAS Capacity (mgd)	4	2	4	6

Hydraulic capacity to and from the secondary clarifiers is evaluated in Section 3. The Hazen Pro hydraulic model was used to determine the distribution of flow to the five clarifiers at the peak hydraulic flow of 43.5 mgd. The hydraulic modeling results indicate that the flumes serving all clarifiers except Secondary Clarifier 5 would be submerged under peak flow conditions, and the predicted flow distribution varies from the theoretical flow distribution based on flume throat width. The results of this evaluation and resulting surface overflow rates for the theoretical and predicted flow distributions are presented in Tables 4-13 and 4-14, respectively.

Table 4-13
Theoretical Secondary Clarifier Flows and Overflow Rates

Secondary Clarifier	% Flow @ 43.5 mgd	Flow @ 43.5 mgd (mgd)	Overflow Rate (gpd)
1	22.2%	9.66	854
2	11.1%	4.83	851
3	11.1%	4.83	851
4	22.2%	9.66	1016
5	33.3%	14.49	911

Table 4-14
Predicted Secondary Clarifier Flows and Overflow Rates

Secondary Clarifier	% Flow @ 43.5 mgd	Flow @ 43.5 mgd (mgd)	Overflow Rate (gpd)
1	22%	9.57	846
2	12%	5.22	920
3	9%	3.92	690
4	21%	9.14	961
5	36%	15.66	985

As indicated in Tables 4-13 and 4-14, Secondary Clarifiers 2 and 5 will experience higher loading conditions due to the hydraulics restrictions. Based on surface overflow rate (SOR), Secondary Clarifiers 2 and 5 will be loaded approximately 8% more heavily than the theoretical distribution. All other clarifiers would be underloaded compared to the theoretical distribution, with Secondary Clarifier 3 being underloaded by nearly 20%. Secondary Clarifier 5 is a deep, relatively new unit, provided with an energy dissipating inlet (EDI) and an appropriately-sized feedwell and is better equipped to handle the additional loading than Secondary Clarifier 2, which is a relatively shallow tank provided with an undersized feedwell and no EDI. The performance of Secondary Clarifier 2 could be significantly affected due to the higher loading condition, limiting the overall capacity of the secondary treatment facilities.

The capacity of the secondary clarifiers is determined by performance during peak flow conditions. A wet weather flow analysis was performed to evaluate the capacity of the secondary treatment system, including the secondary clarifiers, during peak flow events. The wet weather analysis is described in further detail in Appendix I. The capacity of the secondary treatment process is evaluated in the following section.

4.6.5 Secondary Process Evaluation

The capacity of the secondary process is determined by the combined performance of the biological treatment process and secondary clarifiers. Sufficient biomass must be provided under aerobic conditions to fully oxidize organic material and to nitrify ammonia. The performance of the secondary clarifiers is dependent on the clarifier design, hydraulic loading rate and solids loading rate to the clarifiers. Increases in MLSS concentrations may provide a greater level of biological treatment, but the increase in the solids load to the secondary clarifiers may result in biomass washout and effluent quality issues.

Nitrification requirements control the required biomass concentration in the aeration basins. A sufficient aerobic solids retention time (SRT) must be provided to fully oxidize ammonia to nitrate. The required MLSS concentration is set by the aerobic SRT for a given influent wastewater and aerobic volume. Since nitrifier growth is significantly slower in cold weather, additional aerobic SRT must be provided in winter, increasing the required MLSS concentration for a given volume.

A calibrated process model of the Mason Farm WWTP was developed using BioWin 3.01 (Envirosim, Ltd.) process simulation software. The model was calibrated based on historical data and the results of special sampling performed in August 2009. The results of the special sampling are summarized in Appendix E, and the process model calibration is detailed in Appendix F.

The process model was used to assess the capacity of the existing activated sludge process to treat the Year 2030 projected flows and loads developed in Section 2. In addition, simulations were performed to estimate plant performance at the current 14.5 mgd permitted design flow. The process modeling results are summarized in Appendix G. Annual average and maximum month design conditions are summarized in Tables 1 through 3 of Appendix G.

The annual average process simulations indicate that full nitrification would be achieved at an aerobic SRT of 5 days at a temperature of 20°C. Average MLSS concentrations assuming current wastewater strength are approximately 3,000 mg/L, which is typical of nutrient removal processes. The process simulations indicated that chemical phosphorus removal would be required to reduce effluent TP concentrations to 0.2 mg/L. Total nitrogen concentrations under this scenario would still be well above the required effluent criteria to meet the current and future TN mass allocations. Substantial denitrification would be required across the existing denitrification filters to meet the mass allocations.

The process simulations predicted that full nitrification would be achieved at an aerobic SRT of 7 days under maximum month conditions, which assumes a minimum temperature of 13°C. MLSS concentrations at current wastewater strength are in the 4,000 – 4,500 mg/L range, which is at the high end of typical activated sludge design and may lead to clarification issues during high flows. The MLSS concentrations at the increased wastewater strength are excessive, and the Mason Farm WWTP would have insufficient aeration basin capacity to treat maximum month load conditions during cold weather. Improvements and/or an expansion to the secondary treatment facilities prior to Year 2030 would be required if influent wastewater characteristics strengthen appreciably above current levels.

The need for significant alum feed was predicted to reduce effluent TP concentrations to meet current and future limits. Likewise the process simulations indicate that the Mason Farm WWTP will need to rely on the denitrification filters for the majority of nitrate and TN removal required if operated in the current secondary process configuration. Section 6 discusses the process evaluations of nutrient reduction alternatives and details the recommendations to optimize TP and TN removal.

The ability of the secondary treatment facilities at the Mason Farm WWTP to treat expected flows through Year 2030 is dependent on secondary clarifier performance at peak flows and MLSS concentrations. Predicted maximum month MLSS concentrations to provide full nitrification at current wastewater strength are relatively high, and clarifier performance could be compromised. A wet weather treatment analysis was performed to assess secondary clarifier performance and determine the capacity of the secondary treatment process. The wet weather analysis is summarized herein, and additional details are provided in Appendix I.

The calibrated BioWin model was employed to model the design storm event developed in Section 2 through the biological treatment facilities. The Clarifier 2Dc computation fluid dynamics (CFD) clarifier modeling program was implemented to simulate the effects of the increased hydraulic flows and solids loading rate on the existing secondary clarifiers. Dynamic analyses were performed using these two models to determine effluent quality changes, particularly in regards to ammonia and effluent suspended solids concentrations through the design storm. The two models were coupled to one another by applying the BioWin predicted MLSS concentrations during the design storm to the Clarifier 2Dc model to accurately represent the solids loading rates on the secondary clarifiers.

The hydraulic modeling predicted that Secondary Clarifiers 2 and 5 will experience flows greater than the theoretical distribution due to submergence of the flow distribution flumes at the 43.5 mgd peak flow. The predicted flow distribution used to model these two secondary clarifiers was previously summarized Table 4-14. Since Secondary Clarifier 2 is relatively shallow, has an undersized feed well and no EDI, the CFD clarifier model predicts that this unit will be the first secondary clarifier to fail under high flow and load conditions. Secondary Clarifier 2 is the limiting unit process in the secondary treatment system with the exception of the aeration system, which was previously addressed.

The predicted maximum month MLSS concentration at the Year 2030 assuming current wastewater strength and expected flows is approximately 4,000 mg/L. The CFD model predicted that Secondary Clarifier 2 would be close to failure during the design storm at this condition even under optimal settling conditions. Secondary Clarifier 2 is expected

to fail at MLSS concentrations approaching 3,500 mg/L under average settling conditions and would fail at MLSS concentrations exceeding 2,600 mg/L under poor settling conditions. Excessive blankets were predicted in the remaining clarifiers at a MLSS concentration of 3,500 mg/L under average settling conditions, and these clarifiers would be expected to fail if sustained peak flows were prolonged beyond the 24 hour period modeled. Application of polymer during periods of less than optimal settleability will be required to prevent clarifier thickening failure during sustained peak wet weather events.

The impacts of step feeding primary effluent flow were evaluating using the developed models. The BioWin simulations predicted that MLSS concentrations would be reduced to approximately 3,500 mg/L if primary effluent flows above 28 mgd were step fed to Cell 5 as recommended in the Mason Farm WWTP Process Control Plan. Step feed of primary effluent downstream limits the ability of the aeration basins to fully nitrify since a portion of the influent is under aeration for a short time. The process simulations predicted that effluent ammonia concentrations would be elevated between 1 mg/L to 2 mg/L for 36 hours following initiation of step feed. Ammonia concentrations returned to initial levels (less than 0.5 mg/L) after this period, and step feed of flows above 28 mgd is not expected to impact compliance with the current monthly effluent ammonia limits.

Excessively high loading rates inevitably result in accumulation of solids in the secondary clarifiers. This shifting in the biomass inventory results in a decrease in the MLSS concentration in the aeration basins with the consequent decrease in the solids loading rate to the secondary clarifiers. This phenomenon was observed during clarifier stress testing. The BioWin model was used to simulate the impact of clarifier storage and predicted that MLSS concentrations would be reduced below 3,000 mg/L by maintaining a five foot blanket in the clarifiers during the storm event. However the model predicted significantly increased ammonia concentrations since the aerobic SRT was effectively reduced. The storage of sludge in the secondary clarifiers is not a recommended wet-weather strategy, and solids blankets should be kept to a minimum during storm events in order to maximize the secondary clarifier capacity.

Additional reduction of MLSS concentrations to the clarifiers during peak flow events may be realized by storing solids in the NSL basins. Preliminary process simulations indicated that MLSS concentrations could be reduced to approximately 3,000 mg/L without step feed by increasing the RAS rate to the maximum capacity prior to and throughout the storm event. Combining this operation with step feed provides a greater reduction in MLSS concentrations during peak flows. The evaluation of alternatives considers wet weather treatment strategies to reduce the risk of clarifier failure. Modifications to the existing clarifiers including improving flow distribution will also be addressed.

Additional process and clarifier modeling was performed based on limiting peak sustained influent flows to 40 mgd. The modeling indicates that the existing clarifiers have sufficient capacity to treat a MLSS concentration up to 4,000 mg/L at a peak sustained flow of 40 mgd assuming average settling conditions. Step feed of primary effluent flows above 28 mgd under the 14.5 mgd maximum month design conditions will limit MLSS concentrations to 4,000 mg/L, and the existing secondary clarifiers have sufficient capacity up to the current permitted 14.5 mgd design flow provided that peak sustained flows to secondary treatment are limited to 40 mgd.

The maximum month capacity of the secondary treatment process is estimated at 14.5 mgd assuming current wastewater strengths and a maximum sustained peak flow of 40 mgd. Polymer feed and step feed operation will be necessary during peak flows and/or poor sludge settleability conditions in order to process this flow given the limitations of the secondary clarifiers. It is important that OWASA continue to be proactive in reducing inflow and infiltration (I/I) in the collection system to reduce peak flows to the Mason Farm WWTP. Flow equalization upstream of secondary treatment could also be provided to limit peak secondary effluent flows. Long-term collection system improvements and/or equalization facilities should be designed to limit peak sustained hydraulic flows to a maximum of 2.5 times the permitted design flow to provide reliable treatment during wet weather.

4.6.6 Summary of Secondary Treatment Capacity

Table 4-15 summarizes the maximum month flow capacity of the various secondary process elements at current wastewater strengths. In addition the year that the capacity of the existing facility is exceeded is presented for both expected flow and higher demand flow scenarios. The capacity of the existing secondary treatment process is limited by the aeration system, specifically the installed blower capacity to provide sufficient aeration to meet peak day aeration requirements. The capacity of the existing aeration system is expected to be exceeded within the next three years. The existing aeration basins and secondary clarifiers should provide sufficient capacity through the 20-year planning period provided that polymer is fed to the clarifiers during peak flow / poor settleability conditions and step feed of flow to Cells 5A and 5B is utilized during peak flow events.

Table 4-15
Secondary Treatment Capacity Summary – Current Wastewater Strength

	Maximum Month Flow Capacity	Year Capacity Exceeded based on Expected Flow	Year Capacity Exceeded based on Higher Flow
Secondary Treatment Process	14.5 mgd[1]	2040	2031
Aeration System	9.5 mgd	2013	2012

[1] Based on limiting peak sustained hydraulic flows to 40 mgd.

The impacts of increasing wastewater strength on the secondary treatment capacity were also assessed. A maximum month BOD_5 load capacity was determined for the existing secondary process and aeration system. The maximum month BOD_5 load generally sets the operating MLSS concentration in the existing aeration basins and solids loading rate to the secondary clarifiers. The maximum month BOD_5 load was also used to determine the maximum aeration system capacity, although changes in the TKN to BOD_5 ratios can also impact the load the existing aeration system can handle.

Once the maximum month load capacity was established, the year the capacity is expected to be exceeded was determined by assuming an annual 1.7% linear increase in expected maximum month BOD_5 loads from 2008 loads per meter equivalent. This is similar to the development of the Year 2030 increased wastewater strength concentrations addressed in Section 2. As in the case of the current wastewater strength evaluations the aeration system limits capacity of the secondary treatment facilities. Improvements to the existing aeration system will be required by 2020 if influent wastewater loads linearly increase 1.7% per year from 2008 levels. Additional aeration basin and/or secondary clarifier capacity will be required within the next 15 years if influent loads strengthen at this pace.

Table 4-16
Secondary Treatment Capacity Summary – Increasing Wastewater Strength

	Maximum Month BOD_5 Load Capacity (lb/d)	Year Capacity Exceeded
Secondary Treatment Process	23,600	2022
Aeration System	13,000	2012

4.7 Effluent Filters

Secondary effluent flows to six deep bed filters. The filters currently operate primarily in a conventional filtration mode but can operate in a denitrification mode with the addition of acetic acid. In order to meet the anticipated TN limits it is expected that the filters will need to operate in a denitrification mode. Recommended hydraulic loading rate criteria for full denitrification service to reduce effluent TN concentrations below 3 mg/L (15-25 mg/L nitrate removed across the filter) are 2 gpm/sf at average flow and 2.5 gpm/sf under maximum month conditions. Both of these criteria should be met with one filter out of service. Based on these criteria, five filters can provide treatment for an annual average flow of 11.4 mgd and a maximum month flow of 14.3 mgd. If the filters can be used in a “trim” mode, removing only a few parts of nitrate, more aggressive loading rates (3 gpm/sf) can be considered and the filters could be rated at a higher maximum month capacity of 17.1 mgd. Operation of the denitrification filters in a “trim” mode will require capital and/or operational improvements to the secondary treatment process.

Table 4-17
Deep Bed Filter Design Criteria

Effluent Filters	
Number of Filters	6
Type	Deep Bed
Media Type	Coarse Monomedia
Filter Width (feet)	11.67
Filter Length (feet)	68
Filter Area (sf)	793
Average Hydraulic Loading Rate (gpm/sf)	2

4.8 UV Disinfection

The UV disinfection facility is located in the Effluent Filter Complex. Three channels are equipped with UV lamps to disinfect the filtered effluent. Each channel has a treatment capacity of 16 mgd, for a firm capacity of 32 mgd and a total capacity of 48 mgd. Current North Carolina Division of Water Quality (NCDWQ) criteria require that UV disinfection facilities be designed to treat peak flows with one channel out of service; therefore the peak hour capacity of the UV disinfection system is 32 mgd. Since the UV disinfection system at the Mason Farm WWTP was installed prior to this guidance coming into effect, the existing facility would be "grandfathered" under the previous guidance until such time as a plant expansion, re-rate or UV disinfection facility upgrade is undertaken. The UV channels are sized to allow installation of additional lamps in the future to increase total capacity of the facility. Adding lamps in the existing channels would be a relatively low cost approach to increasing the overall UV system capacity. Should the UV system fail, the Mason Farm WWTP has the capability to feed sodium hypochlorite and sodium bisulfate to provide disinfection/dechlorination. This backup disinfection system would also serve the purpose of a standby or redundant UV channel. It is therefore recommended that the UV disinfection system be rated at the full 48 mgd with all channels in service.

4.9 Post-Aeration

Two post aeration tanks operate in parallel to re-aerate the treated effluent prior to discharge to Morgan Creek. Each has a fine bubble diffuser grid with 112 9-inch diameter flexible membrane disc diffusers. A single positive displacement blower provides up to 675 ACFM of air and backup capacity is provided by the Backwash Backup Blower. According to the Process Control Plan, each tank has adequate volume and aeration capacity to raise the effluent dissolved oxygen concentration to 6 mg/l at a flow rate of 40 mgd.

4.10 Effluent Disposal

Under normal conditions, Morgan Creek has a water surface elevation of 247 ft and the gravity outfall piping has a capacity of 101 mgd. Under high flood conditions, the level in Morgan Creek exceeds the level of the filter effluent weirs and effluent pumping is required. Four effluent pumps are currently installed, each rated for 17.5 mgd, providing a firm capacity of 52.5 mgd and a total capacity of 70 mgd. Space is provided for two additional pumps to be installed in the future.

4.11 Chemical Storage and Feed

4.11.1 Acetic Acid

Acetic acid solution (20% by weight) is added at the NSL tanks as a carbon source for denitrification of RAS and biological phosphorus removal. Acetic acid solution can also be added as a carbon source for denitrification in the filters when in denitrification mode. Table 4-18 summarizes the design of the existing acetic acid storage and feed facility.

Table 4-18
Acetic Acid Storage and Feed System Design Criteria

Storage Tanks	
Number	4
Volume per Tank (gallons)	20,000
Total Volume (gallons)	80,000
Available Feed Points	
Nutrified Sludge Tanks	RAS Denitrification VFA Supplementation
Filters	Denitrification

The 20% acetic acid feed rate to the denitrification filters to reduce TN to 3 mg/L at the 14.5 mgd design flow is estimated at 4,650 gpd (assuming current wastewater strengths). The existing storage tanks can provide for 17 days of storage at this rate. Additional demand for RAS denitrification and biological phosphorus removal was simulated at 1,500 gpd. This would increase the overall feed rate to 6,150 gpd, reducing the storage capacity to 13 days. Typical chemical storage designs provide between 15 and 30 days of on-site chemical storage. The evaluation of alternatives in Section 6 includes process optimization to limit chemical usage and alternative supplemental carbon source addition.

4.11.2 Alum

Alum (48% solution by weight) is added to the rotary press filtrate and upstream of the secondary clarifiers to remove soluble phosphorus through precipitation and adsorption. Alum can also be fed to the primary clarifiers and filters. Table 4-19 provides design criteria for the existing alum storage and feed equipment.

The estimated maximum month alum feed rate to reduce soluble ortho-phosphate concentrations below 0.05 mg/L at 14.5 mgd is 1,200 gpd assuming current wastewater strengths. The existing alum storage tanks provide almost 17 days of storage and have adequate capacity through the current design flow assuming wastewater strengths do not significantly increase.

Table 4-19
Alum Storage and Feed System Design Criteria

Storage Tanks	
Number	1
Volume per Tank (gallons)	20,000
Total Volume (gallons)	20,000
Available Feed Points	
Primary Clarifiers	Phosphorus removal
Aeration Basin Effluent	Phosphorus removal
Filters	Phosphorus removal
Rotary Press Filtrate	Phosphorus removal

4.11.3 Sodium Hydroxide

Sodium hydroxide (25% solution by weight) is added to Intermediate Pump Station 1 & 2 influent to provide supplemental alkalinity for nitrification and to the odor control scrubbers for pH adjustment. In addition, sodium hydroxide can be fed to the filters to adjust the effluent pH if necessary.

Table 4-20
Sodium Hydroxide Storage and Feed System Design Criteria

Storage Tanks	
Number	4
Volume per Tank	15,000 gal
Total Volume	60,000 gal
Available Feed Points	
IPS 1&2	Supplemental Alkalinity
Odor Scrubber	Odor Control
Filters	pH Adjustment

Approximately 2,200 gpd of 25% sodium hydroxide are required to maintain a pH of 6.6 under maximum month conditions at the current 14.5 mgd design flow assuming current wastewater strengths. The existing storage facilities have the capacity to store 27 days of sodium hydroxide required for the process, and the exiting sodium hydroxide storage tanks have sufficient capacity through the 14.5 mgd design flow.

4.11.4 Polymer

Polymer can be added upstream of the primary and secondary clarifiers to improve clarification during periods of poor settleability.

Table 4-21
Polymer Storage and Feed System Design Criteria

Storage Tanks	
Number	Totes
Volume per Tote	270 gal
Available Feed Points	
Primary Clarifiers	Enhanced Settling
Secondary Clarifiers	Enhanced Settling

4.11.5 Sodium Hypochlorite

Sodium hypochlorite can be added in an emergency along with sodium bisulfite for disinfection/dechlorination if the UV system fails. In addition it can be added to the aeration basins for foam and bulking organism control. Sodium hypochlorite is also used for reclaimed water disinfection and as an oxidant for odor control.

Table 4-22
Sodium Hypochlorite Storage and Feed System Design Criteria

Storage Tanks	
Number	2
Volume per Tank	3,000 gal
Total Volume	6,000 gal
Available Feed Points	
Aeration Basins	Foam Control
Effluent Reuse Facility	Reuse Disinfection
Odor Scrubber	Odor Control
UV Facility	Emergency Effluent Disinfection

4.11.6 Sodium Bisulfite

Sodium bisulfite is not routinely used at the Mason Farm WWTP. Feed lines to the filters have been installed that are capable of being connected to two 120 gpd pumps, but no pumps are currently present. If the UV system were to fail sodium bisulfite can be added along with sodium hypochlorite to provide an emergency disinfection/dechlorination backup.

Table 4-23
Sodium Bisulfite Storage and Feed System Design Criteria

Storage Tanks	
Number	1
Volume Each Tote	800 gal
Available Feed Points	
Plant Effluent	Emergency Effluent Dechlorination

4.12 Summary of Liquid Stream Capacity

The liquid stream unit process treatment capacity is summarized in Table 4-24. Maximum month and peak hour capacities are provided as applicable to the particular unit process. Figure 4-2 graphically presents the rated capacity of each process unit and compares these capacities to the current maximum month and peak hour design flows. Figure 4-3 presents a timeline of when expansion to the various unit processes will be required based on the expected demand and higher demand wastewater flow projections. Note that these summaries assume current wastewater strength is maintained. Increases in influent wastewater strength will require secondary treatment improvements to occur earlier than shown here.

Table 4-24
Liquid Stream Unit Process Capacity Summary

	Maximum Month Capacity	Peak Hour Capacity	Year Capacity Exceeded based on Expected Flow	Year Capacity Exceeded based on Higher Flow
Preliminary Treatment				
Influent Screening	-	46.6 mgd	2046	2037
Grit Removal	-	50 mgd	2054	2043
Influent Flow Metering	-	55 mgd	2060 +	2051
Primary Clarifiers	-	34.5 mgd	2023	2020
Intermediate Pumping	-	43.5 mgd	2040	2032
Secondary Treatment				
Secondary Process	14.5 mgd	40 mgd	2040	2031
Aeration System	9.5 mgd	-	2013	2012
Tertiary Filters	14.3 mgd	-	2039	2031
UV Disinfection	-	48 mgd	2049	2040
Post-Aeration	-	40 mgd	2032	2027
Effluent Pump Station	-	52.5 mgd	2058	2047

Figure 4-2
Liquid Stream Unit Process Rated Capacity

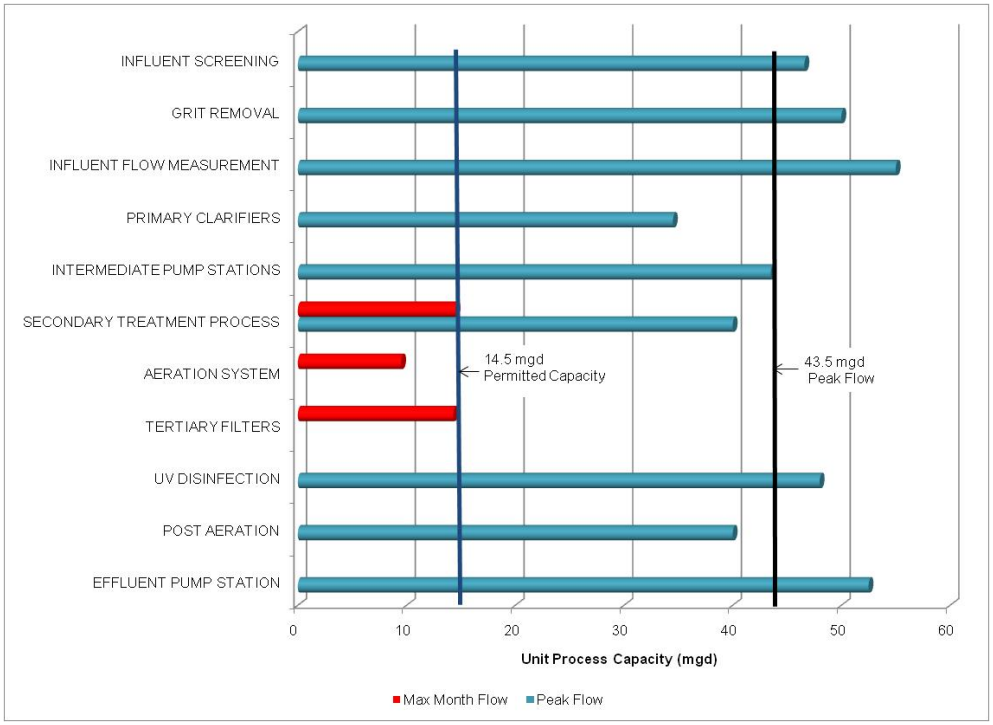
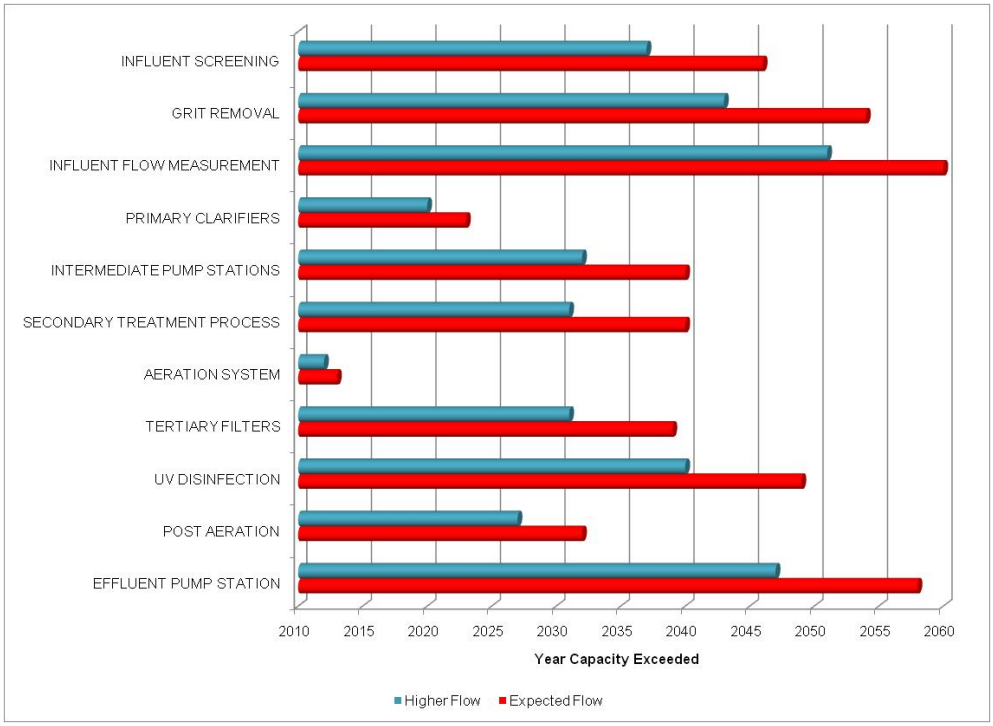


Figure 4-3
Liquid Stream Unit Process Capacity Timeline



4.13 Conclusions

The treatment capacity of the Mason Farm WWTP is limited by several unit processes, most notably the aeration system (blower) capacity in the liquid treatment train. The capacity of the existing aeration system is expected to be reached within the next three years. Capacity improvements are required in the immediate future to allow continued successful operation of the Mason Farm WWTP.

The existing secondary treatment process is expected to be at capacity by the Year 2030, and is limited by the ability of the secondary clarifiers to process the design peak sustained influent flow of 43.5 mgd. The existing secondary clarifiers do have sufficient capacity to process peak sustained flows up to 40 mgd at the current 14.5 mgd design flow. Limiting the peak sustained flow to the secondary process to 40 mgd will allow reliable treatment under maximum month conditions at the current design flow. Increases in projected wastewater flows and/or strength will require expansion to the secondary treatment facilities prior to 2030 to reduce the risk of non-compliance with current and future permit limits. Potential to further optimize the existing facilities and/or to operate existing facilities in alternate process configurations to improve treatment capacity and performance is discussed in later sections.

Section 5.0
Solids Stream Facilities Evaluation

5.1 Solids Stream Unit Process Capacity Evaluation

The capacity of the Mason Farm WWTP solids handling process is evaluated in this section. This evaluation addresses the major unit processes of the solids treatment train. Major unit processes that were considered include:

- Primary Sludge Fermentation
- Blended Sludge Gravity Belt Thickening
- Anaerobic Digestion
- Biosolids Storage
- Biosolids Dewatering

Estimated maximum month solids productions from the process evaluation were used to determine the capacity of each unit process. Solids production rates in terms of pounds of total solids produced per million gallons of flow treated (lb TS/MG) at current wastewater strength are summarized in Table 5-1.

Table 5-1
Estimated Raw Solids Production Rates – Current Wastewater Strength

	Annual Average (lb/MG Treated)	Maximum Month (lb/MG Treated)
Primary Solids	1,500	1,700
Secondary Solids	1,300	1,500
Blended Solids Production	**2,800**	**3,200**

The blended solids production rates estimated in the calibrated BioWin process model developed for this study vary significantly from the estimated blended solids production rates previously estimated for the facility (*Final Design Memorandum #4, 15 JAN 2002, Brown and Caldwell*) as shown in Table 5-2. These are further extended to estimated mass and volumetric loadings in Table 5-3 for comparative purposes at average annual flow (11.6 MGD) and maximum month design flow (14.5 MGD) conditions at typical gravity belt thickener solids discharge concentrations (5.5% total solids).

Table 5-2
Blended Raw Solids Production Rates – Current Wastewater Strength

	Annual Average (lb/MG Treated)	Maximum Month (lb/MG Treated)
Brown & Caldwell Estimates	1,520	2,000
Hazen and Sawyer Estimates	2,800	3,200
Percentage Change	84.2%	60.0%

Table 5-3
Blended Raw Solids Production Mass and Volume Comparison

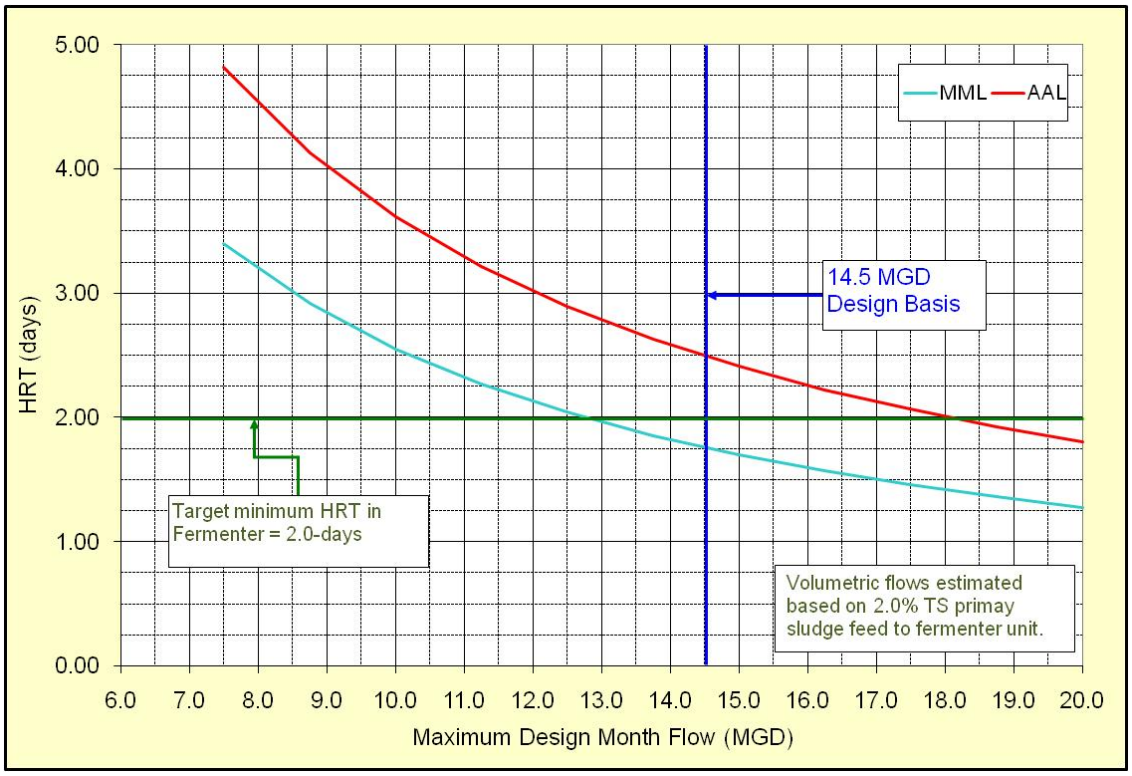
	B&C Estimates	H&S Estimates
Annual Average, lbs/day	17,630	32,480
Annual Average, gallons/day	38,430	70,800
Maximum Month, lbs/day	29,000	46,400
Maximum Month, gallons/day	63,200	101,200

Plant operating staff have indicated that current blended residual volumetric loadings to the anaerobic digestion process average approximately 40,000 to 45,000 gallons per day at an annual average flow rate of approximately 7.25 MGD – or a volumetric residuals production rate of about 5,500 to 6,200 gallons blended sludge per million gallons wastewater treated. The current observed residuals production rate would be consistent with the design condition annual average residuals production rate of 63,800 to 71,920 gallons per day at an annual average flow rate of 11.6 MGD. Therefore, the estimated residuals production rates from the H&S calibrated BioWin model appear to be more consistent with current residuals production rates and will be utilized to evaluate the residuals and solids handling unit treatment processes.

5.2 Primary Sludge Fermentation

Primary sludge is pumped through a grinder and into a fermenter. Under anaerobic conditions, the primary sludge is fermented in order to form volatile fatty acids (VFAs). Fermented sludge is sent to the gravity belt thickeners where the fermentate is separated from the solids and pumped to the nutrified sludge cells as a carbon source for RAS denitrification and biological phosphorus removal. The addition of these VFAs into the secondary process reduces the amount of supplemental carbon food source that must be purchased and applied. Typical fermentation design criteria provides for a two to three day retention time, and the Process Control Plan prepared for the plant recommends an optimum residence time in the fermenter of 2 days. The primary sludge fermenter has a volume of 260,000 gallons; therefore, at a 2-day retention time the estimated volumetric capacity of the fermenter is approximately 130,000 gallons per day. Estimated fermenter residence time as a function of the maximum month design flow is shown in Figure 5-1.

Figure 5-1
Primary Sludge Fermenter Capacity



The maximum month flow capacity of the fermenter is approximately 12.8-mgd based on maintaining a two day hydraulic retention time at maximum month loading conditions. As plant flows increase, the fermenter residence time will drop below 2 days resulting in a decrease in VFA production resulting in increased demands for supplemental alternative carbon food source for the biological nutrient removal process. A primary sludge feed concentration to the fermenter of 2% was assumed for the evaluation. Note that this is greater than the reported average primary sludge concentration of 1.3%. The fermenter hydraulic retention time will fall below the 2-day threshold before plant flows reach the 12.8-mgd threshold if primary sludge feed concentrations cannot be increased.

5.3 Blended Sludge Gravity Belt Thickening

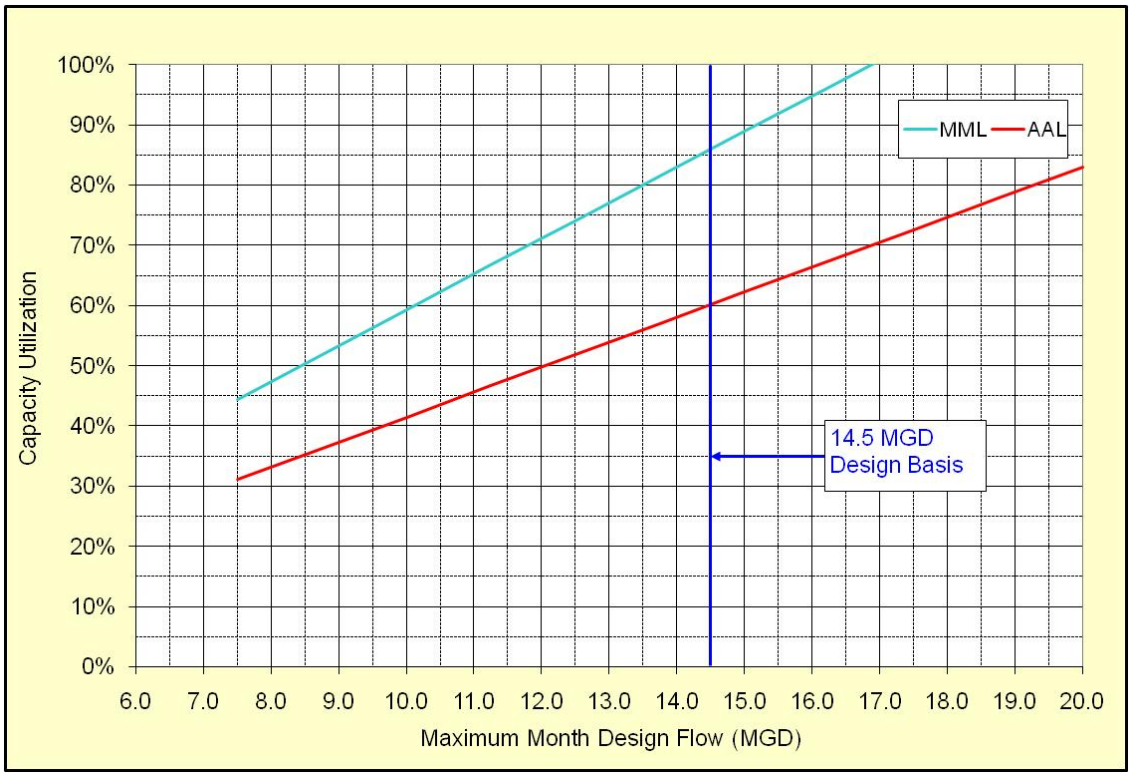
Three gravity belt thickeners are provided for thickening of fermented primary sludge (FPS) and waste activated sludge (WAS). Thickening is provided to reduce the amount of water conveyed with the sludge to the anaerobic digesters. Thickening reduces the amount of water that needs to be heated, effectively increases the detention time of sludge in the digesters and reduces the volume of sludge that must ultimately be land applied or dewatered for composting. FPS and WAS can either be thickened separately or together on the gravity belt thickeners. The current practice is to thicken FPS and WAS together. Table 5-4 summarizes the design criteria for the gravity belt thickeners. The "firm" operating condition is based on having one unit out of service.

Table 5-4
Gravity Belt Thickener Design Criteria

Gravity Belt Thickeners	
Installed Number of Units	3
Firm Number of Units	2
Operating Belt Width (meters)	1.5
Solids Loading Rate (lb TS/hour/meter)	750
Firm Solids Loading Rate (lb TS/hour)	2,250

The gravity belt thickening unit process firm treatment capacity was estimated for a range of loading conditions as shown in Figure 5-2 based on continuous (24 hour per day) operation of the blended sludge thickening unit process. As shown, the utilization rate (i.e., fraction of the firm solids processing utilized) at the 14.5-mgd maximum month flow design condition is approximately 60% under average loading conditions and approximately 85% under maximum month loading conditions. Firm gravity belt thickening capacity is in-place to meet anticipated maximum month loading conditions at design flow conditions up to approximately 16.9-mgd. Gravity belt thickening capacity expansions are not necessary to meet the current 14.5-mgd maximum month design condition.

Figure 5-2
Gravity Belt Thickening Process Rated Capacity (Firm)

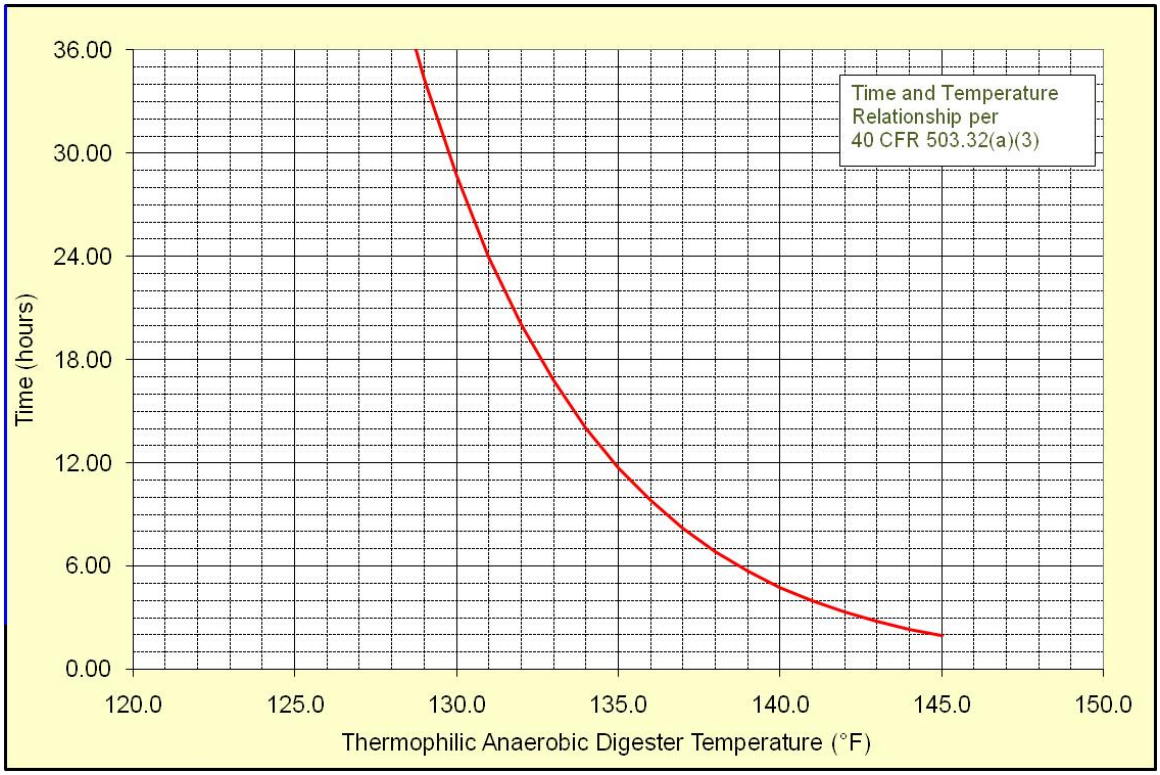


5.4 Anaerobic Digestion

Thickened fermented primary sludge (FPS) and waste activated sludge (WAS) are pumped through the anaerobic digesters in series. The anaerobic digesters are configured to operate in a temperature phased operating mode with the first stage operating at thermophilic temperatures (55-57 °C / 131.0-134.6 °F) followed by a second stage operating at mesophilic temperatures (35-40 °C / 95.0-104.0 °F). The anaerobic digestion process is configured to operate using the 40 CFR 503 “time and temperature” criteria to produce a Class A residuals product with regard to pathogen inactivation.

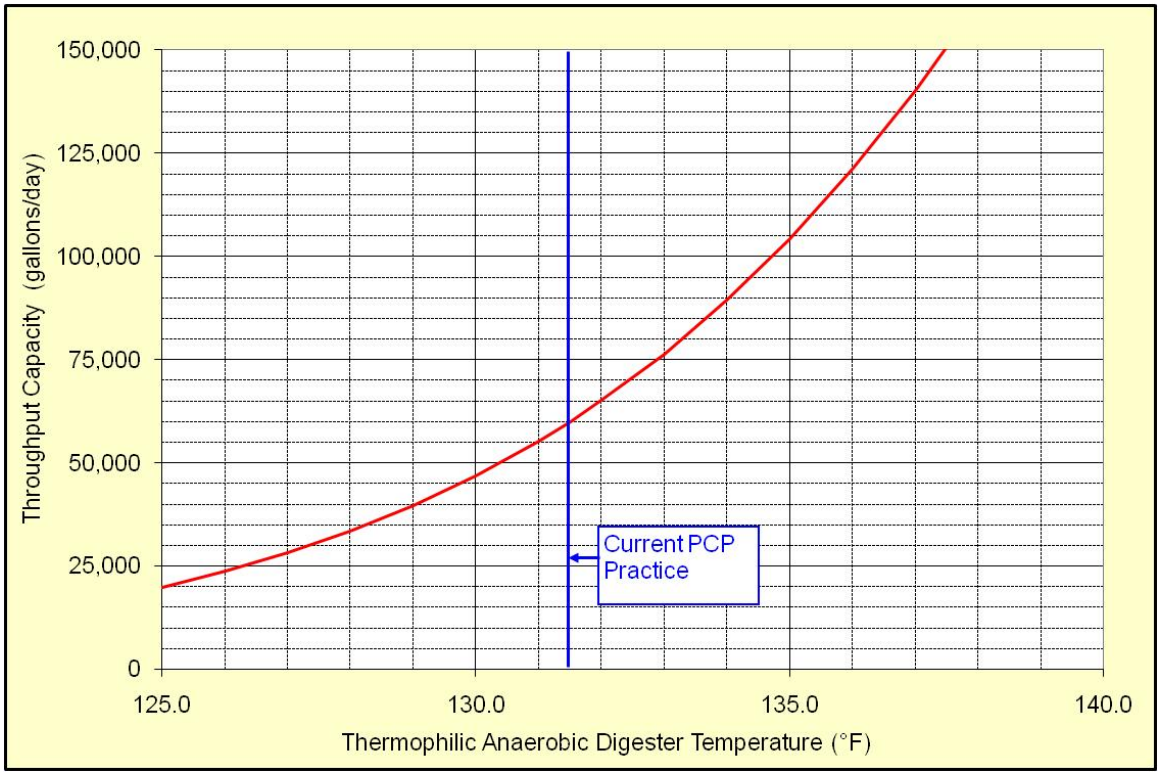
Thickened FPS and WAS are pumped into Digester No. 4 on a continuous basis and heated from ambient to thermophilic temperatures. Flow is then transferred from Digester No. 4 to Digester No. 3 where it is held unfed for a period of time to meet the 40 CFR 503 time and temperature requirements. The minimum duration of the un-fed holding time is dependent on the operating temperature of the thermophilic digester as shown in Figure 5-3. Once the batch time-temperature requirements have been achieved, biosolids are transferred to Digester No. 2. Digester No. 2 serves mainly as an equalization tank to accept batch transfers from Digester No. 3 and then transfer to Digester No. 1. Biosolids are further cooled in Digester No. 1 prior to transfer to the Biosolids Storage Tanks or biosolids dewatering.

Figure 5-3
Digester Time and Temperature Requirements



Based on continued operation in the present mode to achieve the 40 CFR 503 time and temperature requirements for Class A pathogen reduction the capacity of the anaerobic digestion facility is determined by the thermophilic batch process in Digester No. 3. The plant's Process Control Plan describes an operating scenario based on a 24-hour batch cycle time which includes the fill-hold-withdrawal operations. Further, the Process Control Plan indicates that the Digester No. 3 volume per foot of sidewater depth is approximately 13,500 gallons with a maximum water level difference of 4.5 feet. Based on these parameters, Digester No. 3 could process approximately 60,000 gallons per batch cycle. Fill and withdrawal sludge transfer pumping capacity is approximately 960 gallons per minute. The daily digester throughput capacity, as a function of digester operating temperature, is shown in Figure 5-4.

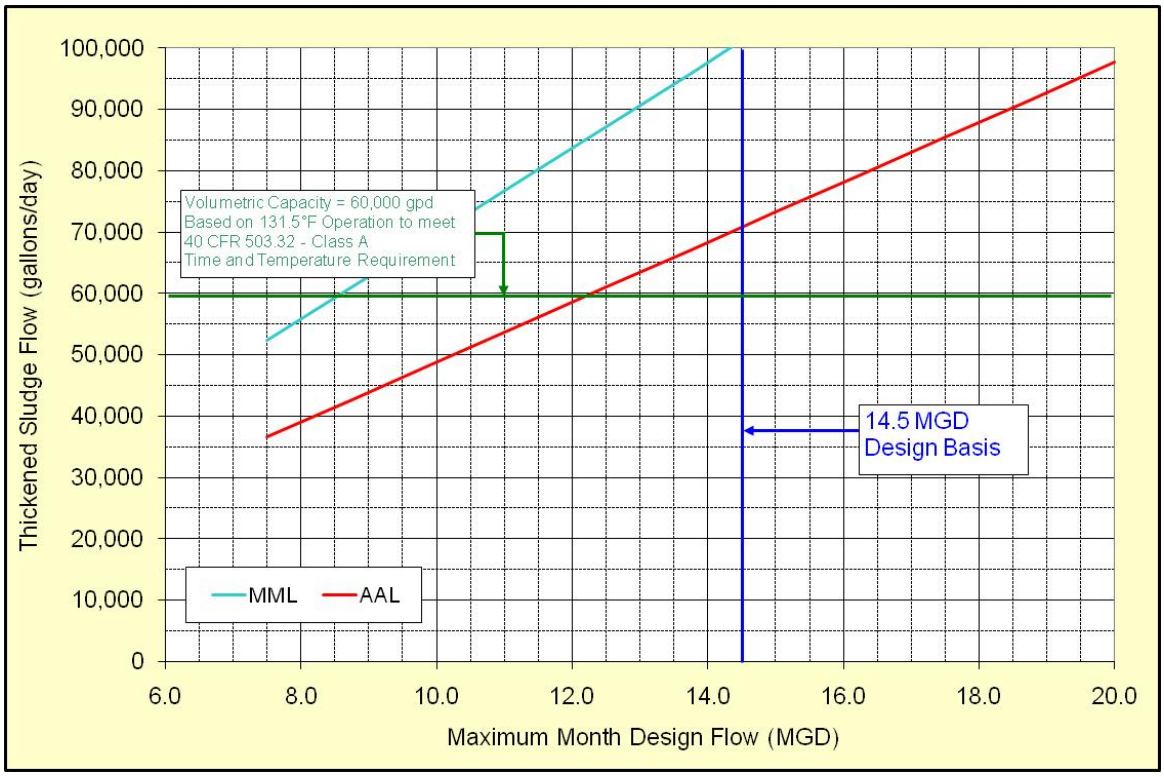
Figure 5-4
Digester Volumetric Throughput Capacity



As shown in Figure 5-4, the digester throughput capacity as described in the Process Control Plan is approximately 60,000 gallons per day based on operation at a minimum temperature in Digester No. 3 of 131.5°F. Based on these capacity constraints for meeting Class A pathogen reduction requirements the anaerobic digestion facilities would be adequate for the anticipated maximum month solids loadings associated with a design flow condition of approximately 8.6-mgd as shown in Figure 5-5.

OWASA operating personnel have indicated that minimum digester operating temperatures have been increased above the 131.5°F minimum temperature described in the Process Control Plan to a range of 135°F t0 138°F which would reduce the minimum unfed hold time in Digester No. 3 and allow throughput capacity to be increased. The impact of these changes to the anaerobic digestion operating temperature and associated ancillary equipment (e.g., heat exchangers, boilers, etc.) is evaluated in Section 7 to address overall capacity needs in the anaerobic digestion unit process.

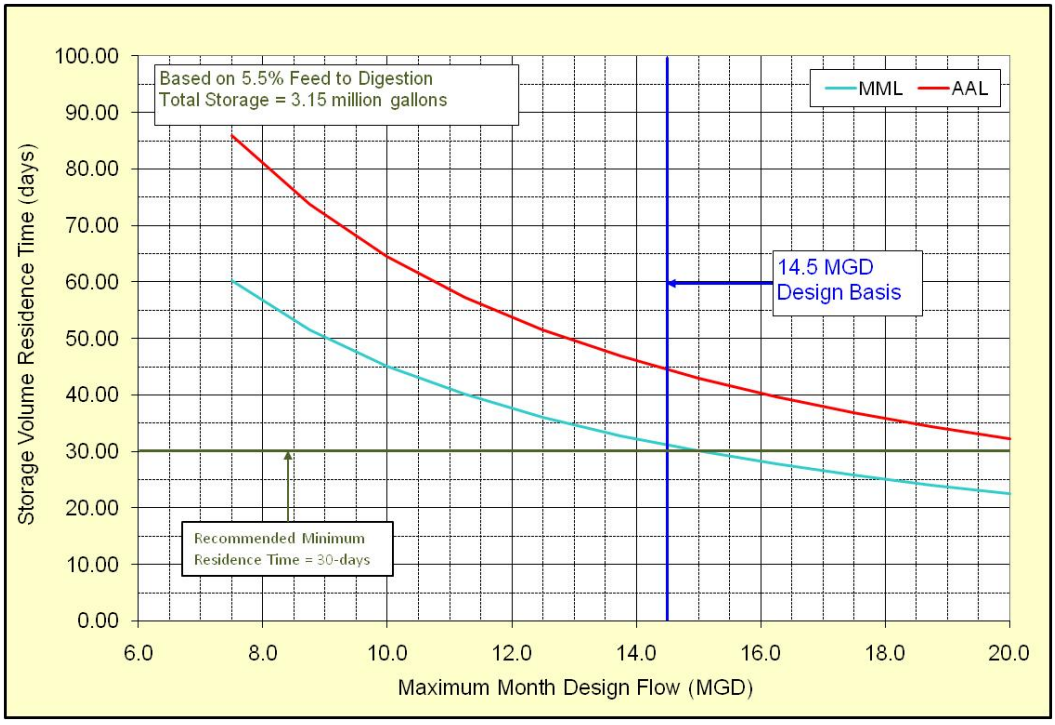
Figure 5-5
Digester Capacity Assessment



5.5 Biosolids Storage

Both on-site and off-site liquid biosolids storage is provided for anaerobically digested Class A biosolids. A total of 3.15 million gallons of storage is provided. Liquid storage volume sufficient to meet 30-days storage under maximum month loading conditions is recommended. Estimated storage volume residence time, as a function of design flow, is shown in Figure 5-6. As shown the existing storage volume is sufficient to meet the 30-day residence time at the maximum month load associated with the 14.5-mgd design loading condition. Additional liquid biosolids storage volume is not recommended to meet the 14.5-mgd design loading condition.

Figure 5-6
Liquid Stabilized Storage Capacity Assessment

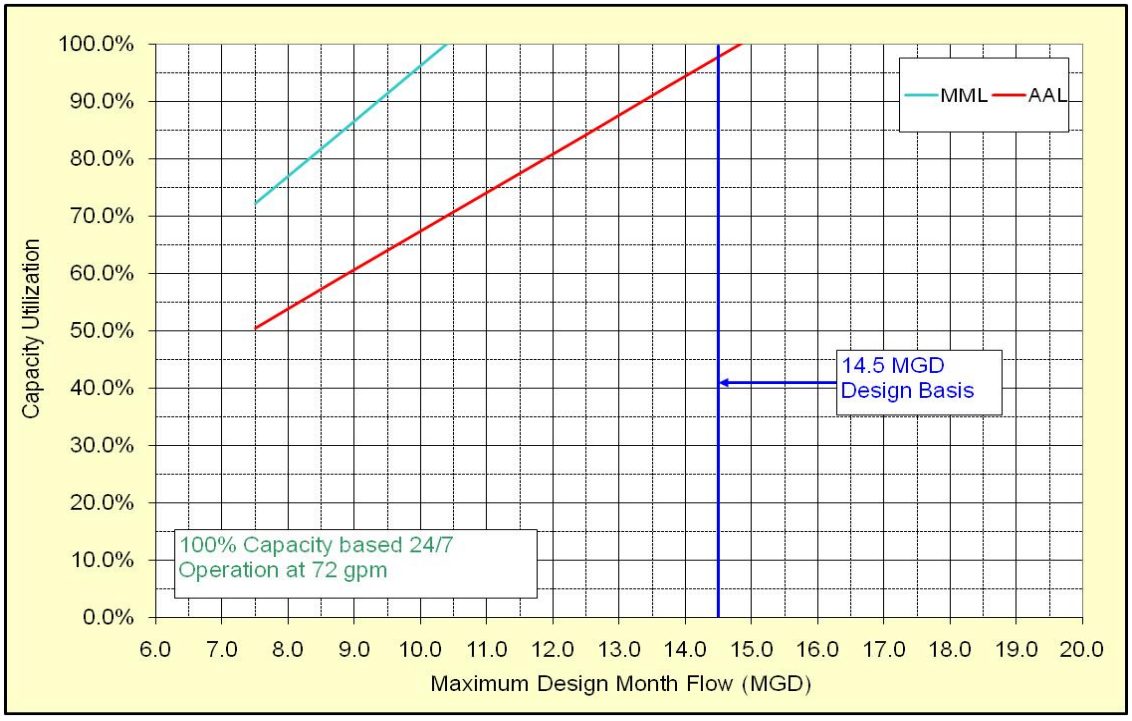


5.6 Biosolids Dewatering

Digested sludge is pumped from the Biosolids Storage Tanks or directly from Digester No. 1 to the Biosolids Dewatering Facility. A rotary press is provided to allow flexibility for the liquid biosolids land application program by allowing a portion of the biosolids to be dewatered and sent off-site for composting. The biosolids dewatering process does not limit the capacity of the solids treatment train since it is used to provide flexibility for disposal of biosolids.

The rotary press dewatering facility was specified with a design capacity of up to 72 gallons per minute at a feed solids concentration of 2.5% for a solids loading rate capacity of approximately 900 dry pounds per hour. Rotary press dewatering capacity was estimated based on operation at the specified design hydraulic capacity for a range of operating scenarios as shown in Figure 5-7. The rotary press dewatering unit process when operating continuously can accommodate annual average loadings associated with design flows of approximately 14.8-mgd and under maximum month loadings associated with design flows of approximately 10.5-mgd.

Figure 5-7
Rotary Press Dewatering Capacity Assessment



Biosolids dewatering capacity is not necessarily capacity limiting since this process is utilized to supplement liquid land application which is the primary means of ultimate residuals management for the Mason Farm WWTP.

5.7 Summary of Solids Stream Capacity

The capacity limiting unit process within the solids treatment train is the Class A time-and-temperature thermophilic anaerobic digestion unit process. Under current operating practice as outlined in the Process Control Plan, the anaerobic digestion process has an equivalent maximum month design flow rate capacity of approximately 8.6-mgd at current wastewater strengths when operating with a 24-hour batch cycle time at a throughput flow of 60,000 gallons per day. The anaerobic digestion process can be operated at a higher temperature and throughput in order to continue producing a Class A biosolids product while treating plant flows up to the permitted capacity of the Mason Farm WWTP. Section 7.5 presents an evaluation of potential modifications to the current thermophilic anaerobic digestion batch process that would increase system capacity.

The solids stream unit process treatment capacity is summarized in Table 5-5. Figure 5-8 graphically presents the rated capacity of each process unit and compares these capacities to the current maximum month and peak hour design flows. Figure 5-9 presents a timeline of when expansion to the various unit processes will be required based on the expected demand and higher demand wastewater flow projections. Note that these summaries assume current wastewater strength is maintained. Increases in influent wastewater strength will require solids handling improvements to occur earlier than shown here.

Table 5-5
Solids Stream Unit Process Capacity Summary at Current WW Strength

	Maximum Month Capacity	Year Capacity Exceeded based on Expected Flow	Year Capacity Exceeded based on Higher Flow
Primary Sludge Fermenter	12.8	2029	2025
Gravity Belt Thickener	16.9	2055	2054
Anaerobic Digestion (Class A Mode)	8.6*	Currently Exceeded	Currently Exceeded
Liquid Biosolids Storage	16.8	2054	2044
Rotary Press	10.5	2017	2015

***Based on operational, not facility, constraints**

Figure 5-8
Solids Stream Unit Process Rated Capacity

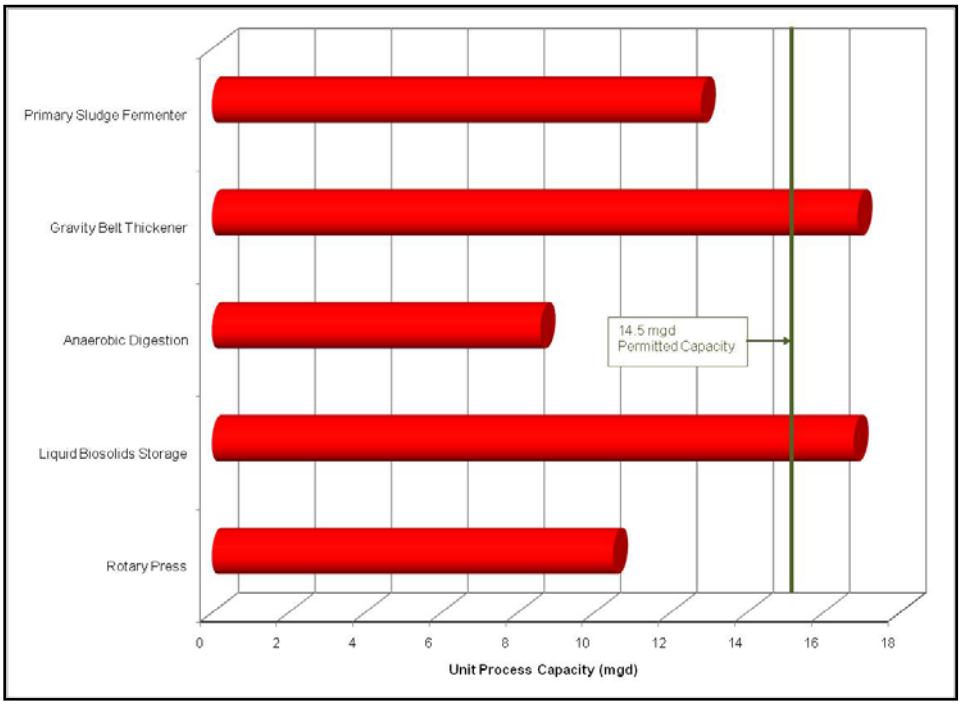
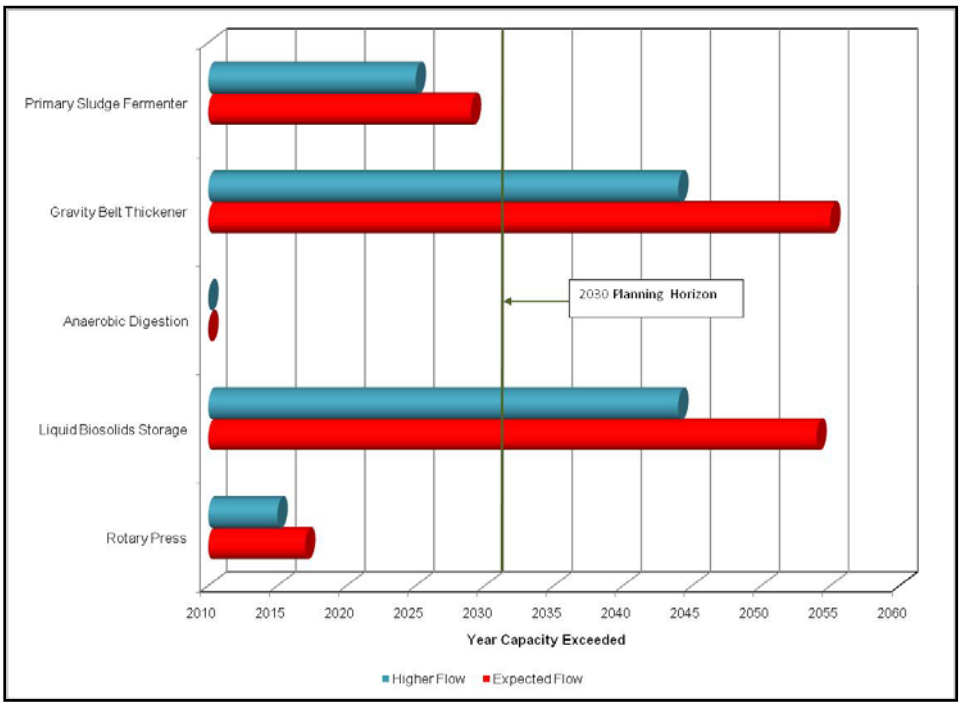


Figure 5-9
Solids Unit Process Capacity Timeline



5.8 Conclusions

The solids treatment capacity of the Mason Farm WWTP is limited by the anaerobic digestion process. The anaerobic digestion facilities are of particular concern since these facilities are currently at capacity for operation in a Class A stabilization mode. Capacity improvements will be required for this system in the near future to allow continued successful operation of the Mason Farm WWTP.

Section 6.0
Liquid Treatment Alternatives

6.1 Goals of Process Optimization

The goals for optimization of the treatment process at the Mason Farm WWTP are multi-faceted. Of primary importance to OWASA is to reliably produce a high quality effluent discharge that is protective of the environment and in compliance with permit requirements. It is also desirable to minimize operating costs for the Plant and to minimize the carbon footprint of the Plant as well. Since some capacity deficiencies have been identified, it is also a goal of this study to determine ways to maximize the treatment capacity of the Plant. Evaluation alternatives will consider the constraints of current and expected future permit limits, requirements of OWASA's reclaimed water system contract with the University of North Carolina, OWASA's odor elimination goals, and requirements for producing Class A biosolids. The evaluations shall accommodate the fact that permit limits for TN and TP are annual mass limits.

6.2 Liquid Treatment Deficiencies

The majority of the unit processes in the Mason Farm WWTP liquid treatment train have adequate capacity to treat a maximum month flow of 14.5 mgd and a peak hourly flow of 43.5 mgd based on current influent wastewater loadings to the Plant. However, the Mason Farm WWTP liquid treatment train has several deficiencies in its current process configuration that will make it difficult for the Plant to perform reliably at the currently permitted capacity of 14.5 mgd.

- The existing primary clarifiers have a rated peak capacity of 34.5 mgd. While this capacity is less than the rated peak capacity of the Mason Farm WWTP, there is bypass piping available that can divert a portion of influent flows around the primary clarification process. Loadings to the secondary treatment system may be marginally increased by wet weather partial flow diversion around the primary clarification process; however, limitations in this process are not anticipated to reduce the overall capacity rating for the Mason Farm WWTP and no improvements are recommended as necessary for this unit process.

- The capacity of the existing secondary treatment process is limited by the aeration system, specifically the installed blower capacity is not adequate to provide sufficient aeration to meet peak day aeration requirements. The existing blower capacity is sufficient for the peak day air demands associated with a maximum month flow of 9.5 mgd assuming current wastewater strengths.

- The aeration basins and secondary clarifiers together form the secondary treatment process at the Mason Farm WWTP. These existing facilities should provide sufficient capacity through the 14.5 mgd design flow provided that peak sustained flows are limited to 40 mgd. In addition, polymer feed to the clarifiers during peak flow / poor settleability conditions and step feed of flow to Cells 5A and 5B will be required during peak flow events.

6.3 Aeration Capacity Alternatives

The capacity of the existing secondary treatment process is limited by the aeration system; specifically, the installed blower capacity is inadequate to provide sufficient aeration to meet peak day aeration requirements that will occur within the next 20 years. The Mason Farm WWTP currently has a total installed electric motor-driven aeration basin blower capacity of 23,150 scfm. A 7,000 scfm methane gas engine-driven blower provides for backup/supplementary aeration for a combined total capacity 30,150 scfm. It is Hazen and Sawyer's experience that digester gas engine-driven equipment requires regular maintenance requiring significant down time; therefore the methane gas engine-driven blower is not counted towards the total peak day capacity.

Expected Year 2030 peak day aeration requirements are 31,000 scfm, and the estimated peak day air demand at the current 14.5 mgd design flow assuming current wastewater strengths are maintained is 35,200 scfm. The existing aeration system limits the maximum month treatment capacity of the Mason Farm WWTP to 9.5 mgd at current wastewater strengths and will need to be improved within the next three years to adequately treat expected future flow and loading conditions. Any increase in influent wastewater BOD_5 or nitrogen concentrations will result in capacity being reached earlier.

Insufficient aeration will increase effluent ammonia concentrations. It is crucial for adequate dissolved oxygen concentrations to be maintained to fully nitrify year-round and meet the effluent TN limit of 3.0 mg/L. In addition sufficient dissolved oxygen (between 2 and 3 mg/L) is required to promote enhanced biological phosphorus removal. Providing for additional denitrification in the secondary process will reduce aeration requirements; however sufficient blower capacity should be provided to meet peak day air demands to protect nitrification assuming no denitrification credit.

Two alternatives were evaluated to provide additional aeration capacity at the Mason Farm WWTP: 1) installing additional blower capacity and 2) replacing the existing jet aeration system with fine bubble diffusers and new blowers. Capital and operational cost estimates were developed to compare these two options and determine whether the increase in aeration efficiency associated with fine bubble aeration justifies the increased capital cost of this option.

6.3.1 Additional Blower Capacity

This alternative includes increasing the total installed capacity, exclusive of the methane gas engine-driven blower, to 35,200 scfm to meet expected peak day aeration requirements at the 14.5 mgd design flow. Installation of two (2) high speed direct-drive centrifugal blowers to supplement the existing blowers was assumed. High speed direct-drive centrifugal blower installations have rapidly expanded over the past two years and provide greater efficiencies throughout their operating range than comparable multistage centrifugal blowers. High speed direct-drive blowers typically have reduced footprint requirements and require no lubrication. Each 300-horsepower (hp) blower would be rated at approximately 6,025 scfm.

The existing blower building has insufficient space for installation of any new blowers, and the existing building would need to be expanded. Estimated capital costs for expanding the Mason Farm WWTP aeration capacity through installing additional blowers is summarized in Table 6-1. The estimated capital cost of providing additional blower capacity to meet peak day aeration requirements at 14.5 mgd is $1.26 million.

Table 6-1
Additional Blower Capacity Capital Cost Estimate

	Estimated Cost
Two (2) 300-HP High Speed Blowers (Installed)	$650,000
Piping and Miscellaneous Mechanical	$100,000
Expansion to Existing Blower Building	$200,000
Electrical Improvements	$100,000
Subtotal	$1,050,000
Engineering & Administration (20%)	$210,000
Total Cost (2010 Dollars)	**$1,260,000**

The estimated average aeration horsepower requirement under this alternative is 50 hp per mgd treated (hp/mgd). Note that this assumes that the dissolved oxygen (D.O.) concentrations in the aeration basins are controlled to 2 mg/L under average conditions. The estimated annual aeration energy cost at 9.5 mgd (the average predicted flow from 2016 through 2035) assuming $0.06/kW-hr is approximately $190,000.

The operation of the jet mixing pumps must be included in the overall cost of maintaining the existing aeration system. A total of (14) 5,300 gpm and (8) 1,500 gpm pumps are installed in the aeration basins. Assuming all pumps are operating (86,200 gpm total flow) at 15 feet of total dynamic head (TDH) and a wire-to-water efficiency of 70% results in a total jet mixing pump power demand of 470 hp. The energy cost associated with this demand assuming $0.06/kW-hr is approximately $180,000/year. The overall estimated energy cost per year at 9.5 mgd annual average flow is $370,000.

6.3.2 Replacement Blowers and Fine Bubble Diffusers

This alternative includes replacing the existing jet aeration system with fine bubble diffusers mounted close to the basin floor to increase oxygen transfer efficiency and reduce operational expenses. Since oxygen transfer efficiencies improve under this alternative air requirements will decrease. The estimated peak day air flows at the current 14.5 mgd design flow would be reduced from approximately 35,000 scfm to 20,000 scfm.

The existing blowers will not be able to meet the increased discharge pressure associated with greater diffuser submergence and pressure loss across the diffuser orifice and membrane. Modification of the existing blower equipment was considered. The three existing 150-horsepower Hoffman blowers cannot be modified and would need to be replaced; however the existing motors and motor starters could be reused. The proposed replacement blowers would have a capacity of 2,450 scfm per blower. The two existing 250-horsepower Lamson blowers could be modified to meet the additional pressure requirements by changing the impellers and adding one additional stage. The modified Lamson blowers would be de-rated to a capacity of 4,450 scfm. The total aeration system capacity would be de-rated to 16,250 scfm if these modifications were made.

Modifying the existing blowers and installing fine bubble diffusers will increase the maximum month aeration system capacity to 11.8 mgd (9.4 mgd annual average flow) compared to the 9.5 mgd rating of the existing system. This would provide for capacity through the Year 2023 assuming that flows and loads do not increase at a rate higher than currently projected. Installing fine bubble diffusers and modifying the existing blowers could be considered to meet short-term capacity needs, but will not adequately address OWASA's 20-year capacity needs. A similar capital investment could be made in new blower systems that would meet the aeration capacity requirements through the 20-year planning period.

Installation of new blowers is recommended if the existing aeration system is replaced with fine bubble diffused aeration. This evaluation assumes installation of two 6,000 scfm and two 4,000 scfm high speed single stage blowers to meet aeration requirements over a range of design conditions.

Table 6-2 presents the capital costs to convert the existing aeration system to fine bubble diffused aeration. The existing blower building has adequate space to allow installation of four high speed single stage blowers, and no new structure would be required. The estimated capital cost to replace the existing jet aeration system with fine bubble diffusers including new blowers is $2,880,000.

Table 6-2
Convert to Fine Bubble Aeration Capital Cost Estimate

	Estimated Cost
Four (4) High Speed Blowers (Installed)	$1,200,000
Fine Bubble Diffusers (Installed)	$800,000
Piping and Miscellaneous Mechanical	$200,000
Modifications/Demolition to Existing Blower Building	$100,000
Electrical Improvements	$100,000
Subtotal	$2,400,000
Engineering & Administration (20%)	$480,000
Total Cost (2010 Dollars)	**$2,880,000**

One potential downside to the installation of fine bubble diffused aeration is the incompatibility of this system with current IFAS technology. IFAS manufacturers currently require that media be installed with a coarse bubble diffused aeration system. It is uncertain at this point in time if the existing jet aeration system could be maintained if IFAS were to be installed in the future. There are few IFAS installations in which existing aeration equipment, jet aeration or fine bubble aeration, has been maintained. However, use of coarse bubble diffused aeration for IFAS systems significantly increases the operating costs as compared to conventional treatment alternatives. It is likely that IFAS manufacturers will continue to make product improvements that may allow use of fine bubble diffused aeration equipment in the future to result in more favorable operating costs.

The potential of keeping the existing jet aeration system for mixing was compared to installing vertical mixers to retain the ability to operate each aerobic cell as an anoxic zone. Since the existing jet aeration system is raised above the basin floor it could likely be retained for jet mixing of anoxic zones without significantly interfering with diffuser placement or performance. Replacing the existing jet mixing equipment in the aeration basins with vertical mechanical mixers would require installation of 16 new vertical mixers (assuming 2 each in cells 5A and 5B) and associated support structures. These improvements are estimated to cost approximately $1.0 million. Installation of submersible mixers would eliminate the need for structural supports for vertical mixers, but they have significant maintenance requirements and have failed at several local

WWTPs and therefore are not recommended. A subset of the aeration basin cells could be fitted with mechanical mixers rather than providing mixing in every cell as a cost savings measure; however, this would decrease the existing process flexibility.

The annual average mixing energy costs assuming operation in the recommended step feed configuration with all jet mix pumps in the anoxic zones operating is $84,000 annually based on $0.06/kW-hr. Replacing the jet mixing equipment with vertical mechanical mixers reduces annual energy costs to $26,000 per year. The 20-year present worth cost of the energy savings assuming a 5% return rate is $720,000.

OWASA reports that the jet mixing pumps need to be repaired every 2-3 years at a cost of $5,000 to $6,000 per pump. Assuming that ten pumps are currently repaired each year at a cost of $5,000 each results in an annual repair cost of $50,000 and a 20-year present worth cost of $620,000. Vertical mixers require much less maintenance, and many installations have operated for over twenty years without requiring major repair.

Converting to fine bubble diffused aeration and installing new high speed single stage blowers results in an average energy requirement of 30 hp/mgd versus 50 hp/mgd for the current system. Annual average energy costs for aeration based on a power cost of $0.06/kW-hr and 9.5 mgd average flow are estimated to be $115,000. The overall estimated energy costs including jet mixing of anoxic zones is $199,000 annually. The estimated energy costs including vertical mixing of anoxic zones is $141,000 annually.

6.4 Recommended Aeration Capacity Alternative

Table 6-3 provides a present worth cost comparison of the two blower replacement options. Fine bubble diffuser replacement costs assume a seven-year diffuser membrane. Annual jet mixing pump repair costs for the fine bubble option were assumed to be half of the current estimated cost, which assumes all mixing pumps installed and in operating. Present worth costs assume a twenty-year period and 5% return rate. All costs given are in 2010 dollars. The analysis assumes energy costs will increase at the same rate of inflation and does not account for potential energy savings credits.

Table 6-3
Aeration Expansion Alternatives Cost Comparison

	Additional Blowers	Convert to Fine Bubble & Jet Mixing	Convert to Fine Bubble & Vertical Mixing
Capital Cost	$1,260,000	$2,880,000	$3,880,000
Annual Energy Costs	$370,000	$199,000	$141,000
Annual Jet Mix Pump Repair Cost	$50,000	$25,000	$0
Present Worth Operating Costs	**$5,234,000**	**$2,792,000**	**$1,757,000**
PW Diffuser Replacement Costs	$0	$103,000	$103,000
Net Present Worth Cost	**$6,494,000**	**$5,775,000**	**$5,740,000**

Converting the existing aeration system to fine bubble diffused aeration and installing new blowers results in a net present worth savings of $720,000. The approximate payback period of these improvements is 11 years. Furthermore, OWASA has already designated capital improvements to repair or replace two of the existing blowers by 2020, and capital can be redirected to fund the conversion to fine bubble diffused aeration. It is recommended that OWASA replace the jet aeration system with fine bubble diffused aeration within the next 3 years to ensure sufficient aeration is maintained to meet the Jordan Lake nitrogen reduction criteria. It is also recommended that the existing jet mixing equipment be removed and vertical mixers be installed during the aeration improvements project to reduce maintenance requirements of submerged equipment and save energy.

Fine bubble diffused aeration is more efficient than jet aeration and air flows to the aeration cells will decrease. The reduction in airflow will impact the required capacity of the proposed carbon adsorption treatment systems for odor control of the basins. In addition, vertical mixer supports could be used to support the odor containment covers, eliminating the concrete pier supports currently designed and further reducing project costs. It was recommended that OWASA revisit the current design of the Phase 3 odor control improvements if fine bubble diffused aeration is incorporated at the Mason Farm WWTP. Detailed discussions of recommended modifications to the Phase 3 odor control improvements are presented in Section 6.7 and Appendix O.

6.5 Nutrient Removal Optimization Alternatives

Current Mason Farm WWTP effluent average TN and TP concentrations are 14 mg/L and 0.4 mg/L, respectively. The plant is not currently optimized for nitrogen removal, with limited denitrification occurring in the secondary process. In addition, significant chemical (alum and acetic acid) is fed to remove phosphorus, and substantial sodium hydroxide is fed to provide alkalinity to maintain robust nitrification.

OWASA has the ability to further reduce nitrogen in the plant effluent by operating the existing filters in denitrification mode through supplemental carbon addition. Effluent nitrate concentrations down to 1 mg/L were observed during full-scale denitrification performance testing of the filters. Operating the filters in the denitrification mode is expected to sufficiently reduce nitrate levels to comply with a TN concentration goal of 3.0 mg/L and comply with the Jordan Lake Rules. Relying on the filters for the majority of denitrification will result in substantial additional chemical costs and does not optimize the utilization of the carbon in the influent wastewater.

Modifications to the secondary treatment facility to improve nutrient, specifically nitrogen, removal were evaluated and compared to the operations under the current configuration. Alternatives were evaluated to maximize the ability of the Mason Farm WWTP to reduce nutrient concentrations in the plant effluent at current conditions and the current 14.5 mgd permitted design flow. The following alternatives were evaluated and are discussed in the following sections:

- Alternative 1 – Current Process Configuration
- Alternative 2 – MLE Process – Convert NSL Cells to Anoxic Zone
- Alternative 3 – 3-Stage BNR Process – Convert NSL Cells to Anoxic/Anaerobic Zones
- Alternative 4 – MLE Process with Internal Anoxic Zone
- Alternative 5 – MLE Process with External Anoxic Zone
- Alternative 6 – Operate Existing Process in Step Feed Mode

The BioWin process model developed in Technical Memorandum No. 3 was used to evaluate each of the nutrient removal alternatives. The model was primarily used to assess effluent TN concentrations, MLSS concentrations and chemical feed requirements for each option. Acetic acid feed requirements are based on the current feed rate of approximately 100 gpd per mgd treated with the exceptions of Alternatives 2 and 3, where acetic acid feed rates were adjusted to provide sufficient carbon for denitrification in the limited available NSL basin volume. Alum feed requirements are based on reducing effluent soluble phosphate below 0.1 mg/L and were calculated with a separate spreadsheet. The inert solids impacts from alum addition were modeled to evaluate solids production. Sodium hydroxide feed requirements were based on maintaining an effluent alkalinity of 75 mg/L (as $CaCO_3$) in Effluent Channel No. 2 to match historical operations.

6.5.1 Alternative 1 – Current Process Configuration

The existing process includes operation of the aeration basins in the 4:3 configuration detailed in Technical Memorandum No. 3. Currently three (3) trains of three (3) cells each are in operation, followed by Effluent Channel 1, Cells 5A and 5B and Effluent Channel 2, which distributes flow to the secondary clarifiers. The third cell in each train is currently operated anoxically for denitrification. Return activated sludge (RAS) is pumped to the Nutrified Sludge (NSL) basin influent channel. Acetic acid and filtrate from the gravity belt thickeners (fermentate) is also fed to the NSL basin influent channel. Currently two cells of the NSL basins are operated to provide for RAS denitrification and biological phosphorus removal.

The current process configuration was evaluated for both the 4:3 and 3:4 modes of operation. Each aeration basin cell and all four NSL cells were assumed to be in service. The third or fourth cell of each train was operated anoxically under annual average conditions for the 4:3 and 3:4 configurations, respectively. Maximum month simulations were performed assuming all aeration basin cells are operated aerobically under maximum month/minimum temperature conditions. Simulations were undertaken assuming the current RAS flow rate of 50% of the influent flow and increasing the RAS flow to 100% of the influent flow rate. Figure 6-1 presents the existing process configuration assuming the 3:4 configuration. Table 6-4 summarizes the predicted operation and effluent quality criteria for the current process configuration.

Figure 6-1
Alternative 1 – Current Process Configuration

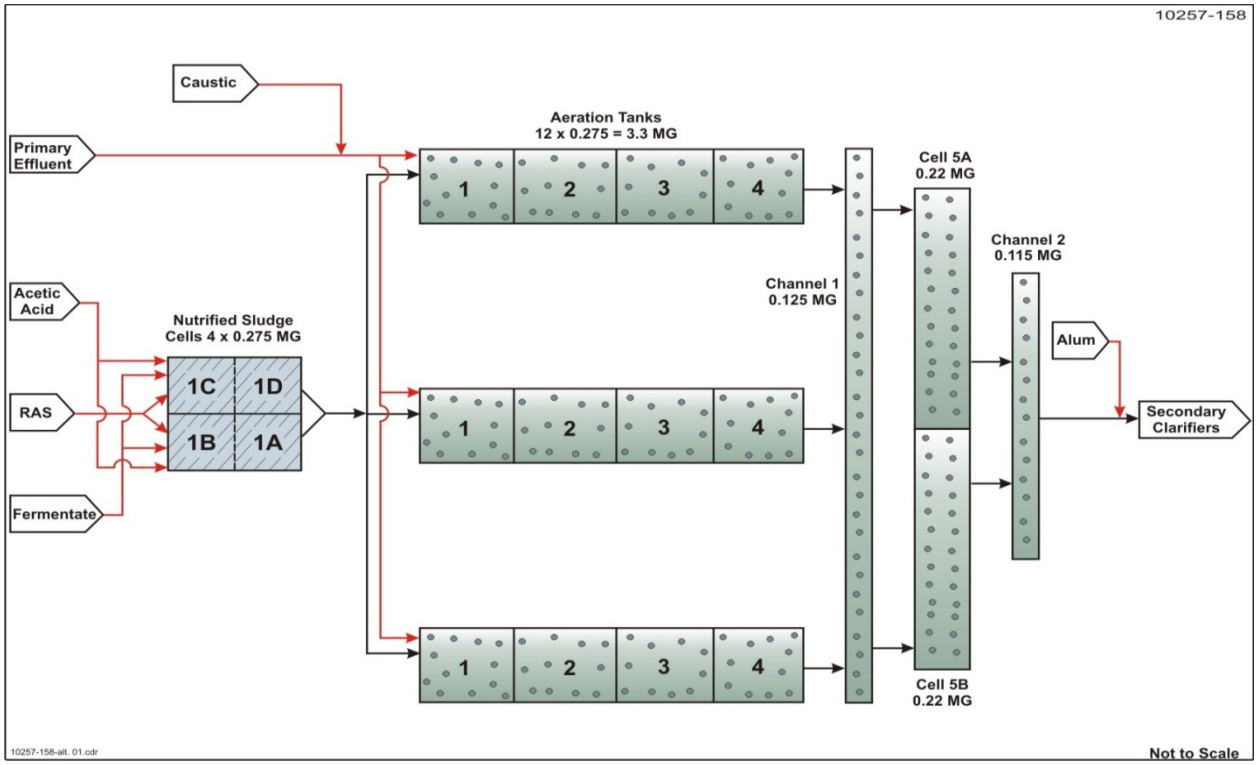


Table 6-4
Predicted Operational Criteria and Effluent Quality –
Existing Process Configuration

Parameter	Annual Avg 4:3 Config. 50% RAS	Annual Avg 4:3 Config. 100% RAS	Annual Avg 3:4 Config. 50% RAS	Max Month 4:3 or 3:4 50% RAS	Max Month 4:3 or 3:4 100% RAS
Flow, mgd	11.6	11.6	11.6	14.5	14.5
Temperature, C	20	20	20	13	13
Aerobic SRT, days	5	5	5	7	7
MLSS Conc., mg/L	3,200	3,000	2,900	4,600	4,300
RAS Flow	50%	100%	50%	50%	100%
Effluent NH3-N, mg/L	0.1	0.1	0.1	0.1	0.1
Effluent NOx-N, mg/L	16.4	12.3	16.7	20.7	15.6
Effluent sTKN, mg/L	1.4	1.5	1.5	1.6	1.7
Effluent TN, mg/L	17.9	13.8	18.3	22.5	17.4
Eff. TP w/o alum, mg/L	2.5	3.6	2.3	2.8	4.7
48% Alum Feed, gpd	900	1,060	830	1,130	1,560
25% NaOH Feed, gpd	2,200	2,000	2,200	4,000	3,300
20% HAc Feed, gpd	1,200	1,200	1,200	1,500	1,500

Operation in the 3:4 configuration results in a slight increase in annual average effluent nitrate and TN since less anoxic volume is available than in the 4:3 configuration. Operation in the 3:4 configuration decreased MLSS concentrations by approximately 10% because a greater proportion of the aeration basin volume was aerobic and the aerobic SRT was maintained at 5 days for both configurations. Changes in operational criteria and effluent quality under maximum month conditions between the 4:3 and 3:4 configurations are insignificant since all aeration basin volume is operated aerobically. Significant denitrification will need to occur in the filters to meet the Jordan Lake nutrient reduction rules if the existing process configuration is maintained.

Increasing the RAS rate to 100% will increase the nitrate load to and removal in the NSL basins; however, biological phosphorus removal is reduced and more alum is required. Higher RAS flows also decrease the MLSS concentration due to additional solids storage in the NSL basins.

6.5.2 Alternative 2 - MLE Process - Convert NSL Cells to Anoxic Zone

Alternative 2 includes conversion of the existing process to a modified Ludzack-Ettinger (MLE)-type process. Nitrified recycle (NRCY) would be pumped from Effluent Channel 2 to NSL Cells 1B and 1C. All aeration basin cells are aerated under this alternative. Primary effluent would also be redirected to NSL cells 1B and 1C to provide carbon for denitrification.

The hydraulic profile in the NSL cells would need to be increased to allow for gravity flow of NSL effluent to the aeration basin influent channel to eliminate the need to repump the flow to the aeration basins. Improvements to the flow distribution gates between the various aeration basin cells and channels would be required to accommodate the increased flows through the aeration basins. The hydraulics through the NSL basins would also need to be improved to accommodate the increased flow through the basins.

This alternative does not provide for biological phosphorus removal, and chemical phosphorus removal would be relied on for phosphorus removal. A schematic of this alternative is provided in Figure 6-2. Predicted effluent quality and operational parameters under annual average and maximum month conditions are presented in Table 6-5.

Figure 6-2
Alternative 2 – MLE Process – Convert NSL Cells to Anoxic Zone

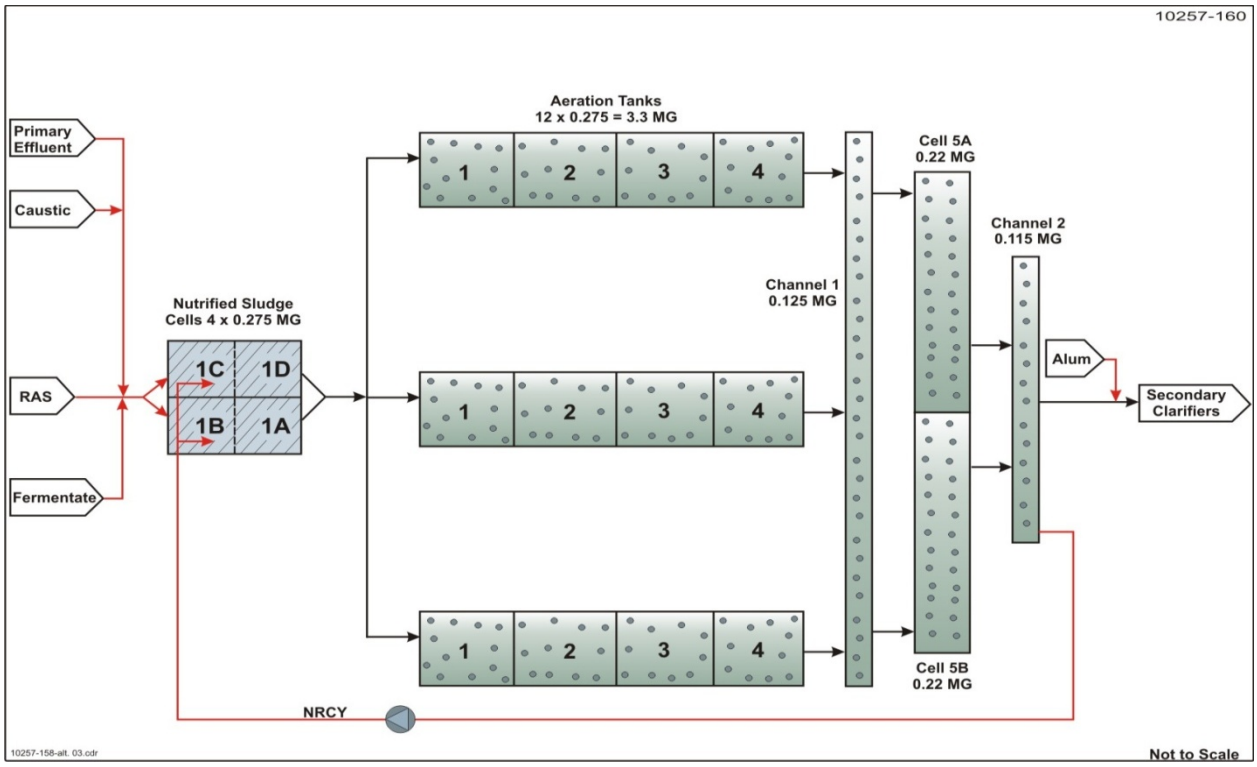


Table 6-5
Predicted Operational Criteria and Effluent Quality –
Convert NSL Cells to Anoxic Zone

Parameter	Annual Avg Design Flow	Max Month Design Flow
Flow, mgd	11.6	14.5
Temperature, C	20	13
Aerobic SRT, days	5	7
MLSS Conc., mg/L	2,200	4,400
NRCY Flow	300%	300%
RAS Flow	100%	100%
Effluent NH3-N, mg/L	0.1	0.2
Effluent NOx-N, mg/L	6.4	6.9
Effluent sTKN, mg/L	1.6	1.8
Effluent TN, mg/L	8.1	8.9
Eff. TP w/o alum, mg/L	4.9	5.2
48% Alum Feed, gpd	1,280	1,660
25% NaOH Feed, gpd	1,500	2,200
20% HAc Feed, gpd	2,000	3,000

Substantial acetic acid and alum feed were predicted under this configuration. Sodium hydroxide feed requirements are significantly reduced due to alkalinity recovery through denitrification. The NSL cells have a total volume of approximately 1.1 MG, resulting in an anoxic retention time of 1.8 hours at 14.5 mgd. Typical pre-anoxic zone retention times are between 3 and 4 hours at design flow. The low available retention time requires substantial supplemental carbon feed to fully denitrify in the limited volume. It is not necessary to feed acetic acid under this alternative since no biological phosphorus removal is occurring; another carbon source can be fed instead. Although effluent TN concentrations between 8 to 9 mg/L are achievable, they come at significant chemical costs.

6.5.3 Alternative 3 – 3-Stage BNR Process - Convert NSL Cells to Anoxic/ Anaerobic Zone

Alternative 3 includes conversion of the existing process to a three-stage biological nutrient removal (BNR) process for phosphorus and TN removal. NRCY would be pumped from Effluent Channel 2 to NSL Cells 1A and 1D. All aeration basin cells are aerated under this alternative. Primary effluent would be pumped directly to NSL cells 1B and 1C to provide additional carbon for denitrification and biological phosphorus removal.

The hydraulic profile in the NSL cells would need to be increased to allow for gravity flow of NSL effluent to the aeration basin influent channel to eliminate the need to repump the flow to the aeration basins. Improvements to the flow distribution gates between the various aeration basin cells and channels would be required to accommodate the increased flows through the aeration basins. The hydraulics through the NSL basins would also need to be improved to accommodate the increased flow through the basins.

A schematic of this alternative is provided in Figure 6-3. Table 6-6 summarizes the predicted annual average and maximum month effluent quality and operational conditions at the current design flow.

Figure 6-3
Alternative 3 – 3-Stage BNR Process –
Convert NSL Cells to Anaerobic/Anoxic Zones

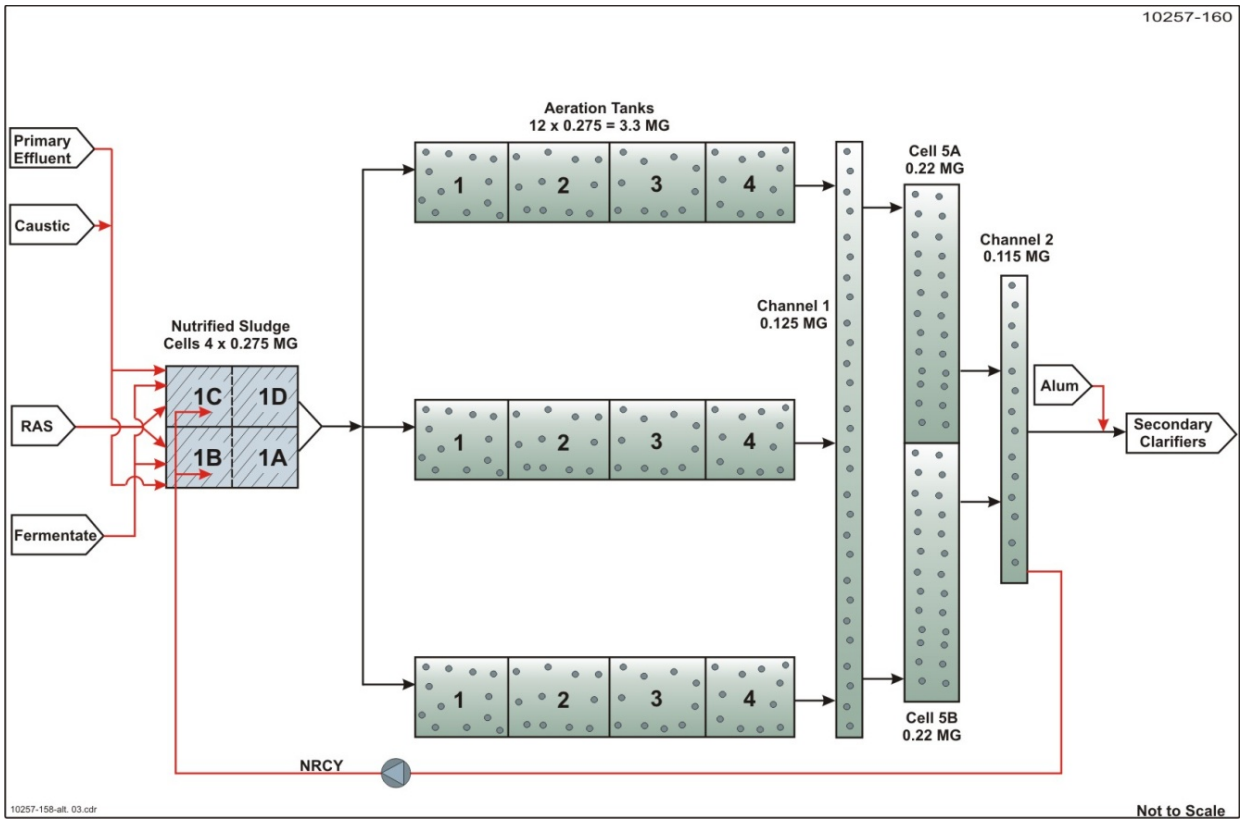


Table 6-6
Predicted Operational Criteria and Effluent Quality – 3-Stage BNR Process

Parameter	Annual Avg Design Flow	Max Month Design Flow	Max Month 2030 Flow
Flow, mgd	11.6	14.5	13.0
Temperature, C	20	13	13
Aerobic SRT, days	5	7	7
MLSS Conc., mg/L	2,600	4,900	4,400
NRCY Flow	200%	200%	200%
RAS Flow	100%	100%	100%
Effluent NH3-N, mg/L	0.1	0.1	0.1
Effluent NOx-N, mg/L	7.9	8.5	8.5
Effluent sTKN, mg/L	1.5	1.7	1.7
Effluent TN, mg/L	9.5	10.4	10.3
Eff. TP w/o alum, mg/L	1.2	2.2	2.2
48% Alum Feed, gpd	620	1,000	900
25% NaOH Feed, gpd	1,500	2,300	2,000
20% HAc Feed, gpd	4,000	5,000	4,500

Substantial acetic acid addition is predicted under this configuration. Sodium hydroxide feed requirements are significantly reduced due to alkalinity recovery through denitrification. The NSL cells have a total volume of approximately 0.55 MG, resulting in an anoxic retention time of 0.9 hours at 14.5 mgd. Typical pre-anoxic zone retention times are between 3 and 4 hours at design flow. The low available retention time requires substantial acetic acid feed to fully denitrify in the limited volume in addition to the acetic acid required for biological phosphorus release in the anaerobic zone. Although effluent TN concentrations between 9 to 12 mg/L are achievable, they come at significant chemical costs. In addition, maximum month MLSS concentrations are excessive under maximum month design flow conditions due to excessive acetic acid addition and associated solids production. The maximum month flow that this alternative could treat using existing volume is 13.0 mgd, which is the projected 2030 maximum month design flow.

6.5.4 Alternative 4 –MLE Process with Internal Anoxic Zone

Alternative 4 includes pumping NRCY from Effluent Channel No. 2 to the first cell in each aeration train. This alternative assumes operation in the 3:4 configuration. The first cell in each aeration train would be operated anoxically to provide denitrification. The first two cells of each train could be operated anoxically under warm weather and/or lower flow and load conditions to provide for a greater degree of denitrification. Improvements to the flow distribution gates between the various aeration basin cells and channels would be required to accommodate the increased flows through the aeration basins. NSL Cells 1B and 1C would continue to receive RAS and fermentate, and acetic acid would still be fed to the NSL cells to supplement the fermentate for biological phosphorus removal.

A schematic of this alternative is provided in Figure 6-4. Predicted effluent quality and operational criteria are summarized in Table 6-7. Annual average conditions assume anoxic operation of the first two cells in each train under annual average conditions and anoxic operation of the first cell in each train under maximum month conditions.

Figure 6-4
Alternative 4 – MLE Process with Internal Anoxic Zone

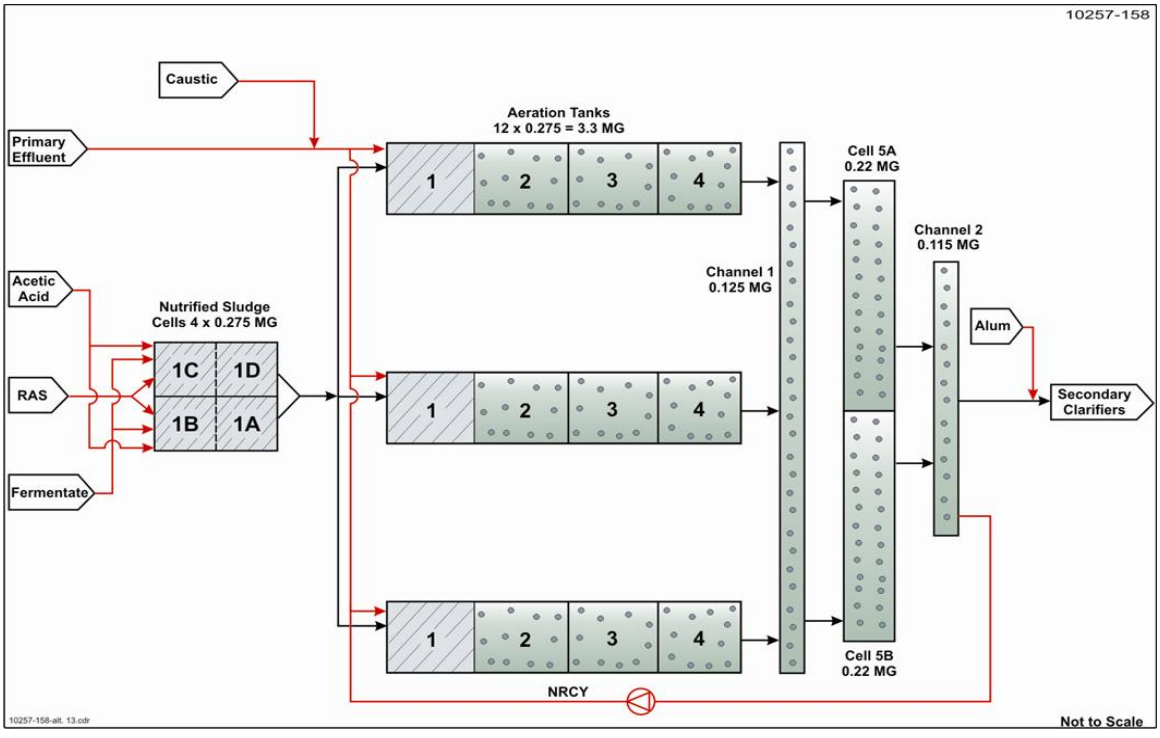


Table 6-7
Predicted Operational Criteria and Effluent Quality –
MLE Process w/ Internal Anoxic Zone

Parameter	Annual Avg Design Flow	Max Month Design Flow	Max Month 2030 Flow
Flow, mgd	11.6	14.5	14.5
Temperature, C	20	13	13
Aerobic SRT, days	5	7	6
MLSS Conc., mg/L	3,700	5,100	4,600
NRCY Flow	300%	150%	150%
RAS Flow	100%	100%	100%
Effluent NH3-N, mg/L	0.3	0.5	0.9
Effluent NOx-N, mg/L	6.7	10.4	9.9
Effluent sTKN, mg/L	1.8	2.0	2.5
Effluent TN, mg/L	8.5	12.5	12.5
Eff. TP w/o alum, mg/L	3.3	3.7	3.3
48% Alum Feed, gpd	1,000	1,330	1,170
25% NaOH Feed, gpd	1,200	2,500	2,300
20% HAc Feed, gpd	1,200	1,500	1,300

The process model predicted annual average TN requirements below 9 mg/L under this alternative with a small increase in alum requirements. Predicted annual average ammonia concentrations are greater than the other alternatives due to reduced aerobic retention times. Sodium hydroxide feed requirements are significantly reduced due to alkalinity recovery through denitrification. Anoxic operation of the first two cells in each train results in a retention time of 3.4 hours at the 11.6 mgd annual average flow, which is appropriate for pre-anoxic zones.

Maximum month MLSS concentrations are excessive under maximum month/minimum temperature conditions assuming operation at a 7-day aerobic SRT since a portion of the available aerobic volume is operated anoxically. Decreasing the aerobic SRT to 6 days under this condition decreases the MLSS concentration sufficiently to allow treatment of up to the 14.5 mgd design flow. The process model predicted that effluent ammonia concentrations would remain below 1 mg/L at the decreased retention time. It

should be noted that operating at the decreased retention time will reduce the degree of conservatism in the nitrification design; however, OWASA still has the ability to provide additional aerobic volume by operating the anoxic zone as an aerobic zone during infrequent cold weather/high load events, while providing a greater portion of the required denitrification in the denitrification filters.

6.5.5 Alternative 5 – MLE Process with External Anoxic Zone

Alternative 5 includes construction of an external anoxic basin upstream of the existing aeration basins. NRCY from Effluent Channel 2 would be pumped to the new anoxic basin, and primary effluent would be redirected to the new basin to provide carbon for denitrification. All aeration basin cells are aerated under this alternative. NSL Cells 1B and 1C would continue to receive RAS and fermentate, and acetic acid would still be fed to the NSL cells to supplement the fermentate for biological phosphorus removal.

Improvements to the flow distribution gates between the various aeration basin cells and channels would be required to accommodate the increased flows through the aeration basins. In addition, improvements to the intermediate pump station may be required to allow gravity flow from the external anoxic zone back to the aeration basins.

A schematic of this alternative is provided in Figure 6-5. Table 6-8 presents the predicted effluent quality and operational criteria for Alternative 5.

Figure 6-5
Alternative 5 – MLE Process with External Anoxic Zone

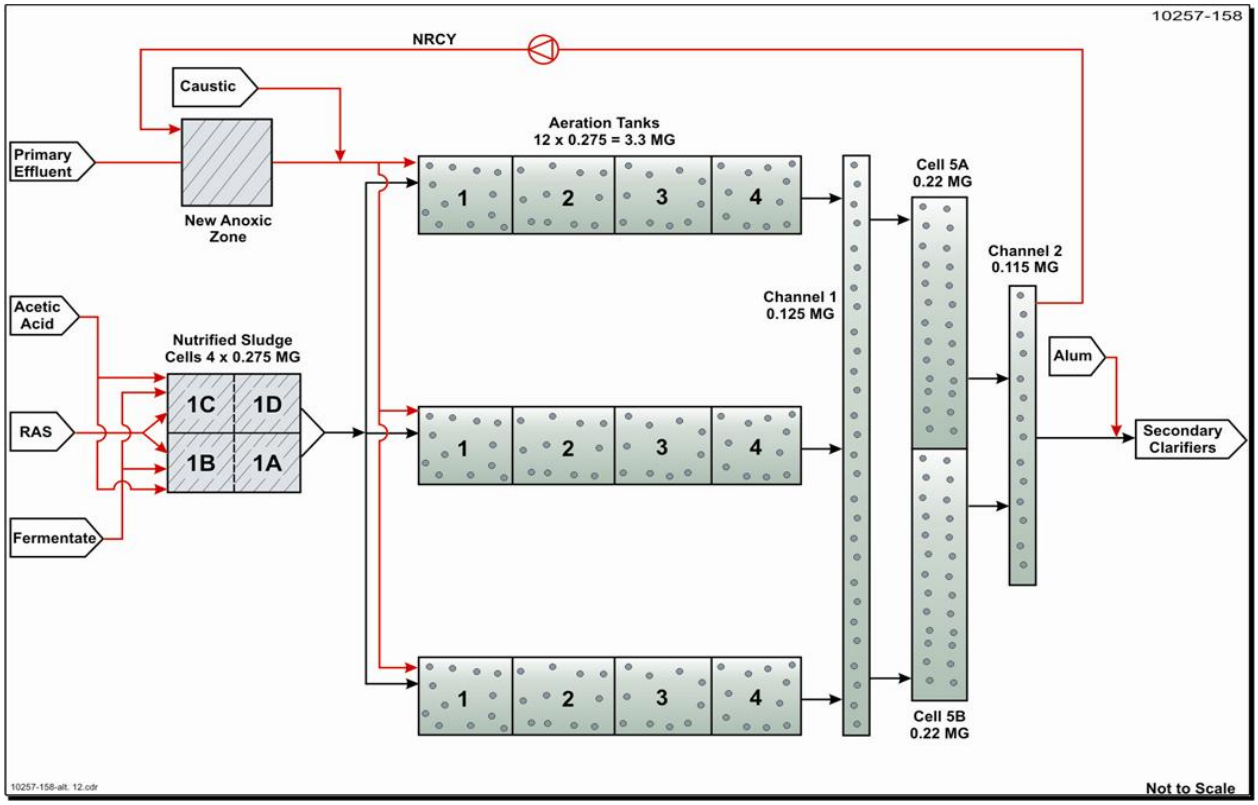


Table 6-8
Predicted Operational Criteria and Effluent Quality –
MLE Process w/ External Anoxic Zone

Parameter	Annual Avg Design Flow	Max Month Design Flow
Flow, mgd	11.6	14.5
Temperature, C	20	13
Aerobic SRT, days	5	7
MLSS Conc., mg/L	2,200	4,300
NRCY Flow	250%	250%
RAS Flow	100%	100%
Effluent NH3-N, mg/L	0.1	0.5
Effluent NOx-N, mg/L	8.0	8.3
Effluent sTKN, mg/L	1.6	2.1
Effluent TN, mg/L	9.6	10.5
Eff. TP w/o alum, mg/L	3.0	3.8
48% Alum Feed, gpd	950	1,350
25% NaOH Feed, gpd	1,300	2,200
20% HAc Feed, gpd	1,200	1,500

The process model predicted annual average TN requirements below 10 mg/L under this alternative with a small increase in alum requirements. Sodium hydroxide feed requirements are significantly reduced due to alkalinity recovery through denitrification. The modeled external anoxic zone had a volume of 2.4 MG, resulting in an anoxic retention time of 4.0 hours at 14.5 mgd. Annual average TN concentrations are greater than in the internal NRCY option because of lower operational MLSS concentrations since NSL effluent (RAS) is still being returned directly to the aeration basins under this option. Maximum month TN concentrations are reduced since existing volume is not providing additional denitrification capacity. Increasing the NRCY rate above 250% did not significantly impact effluent TN concentrations

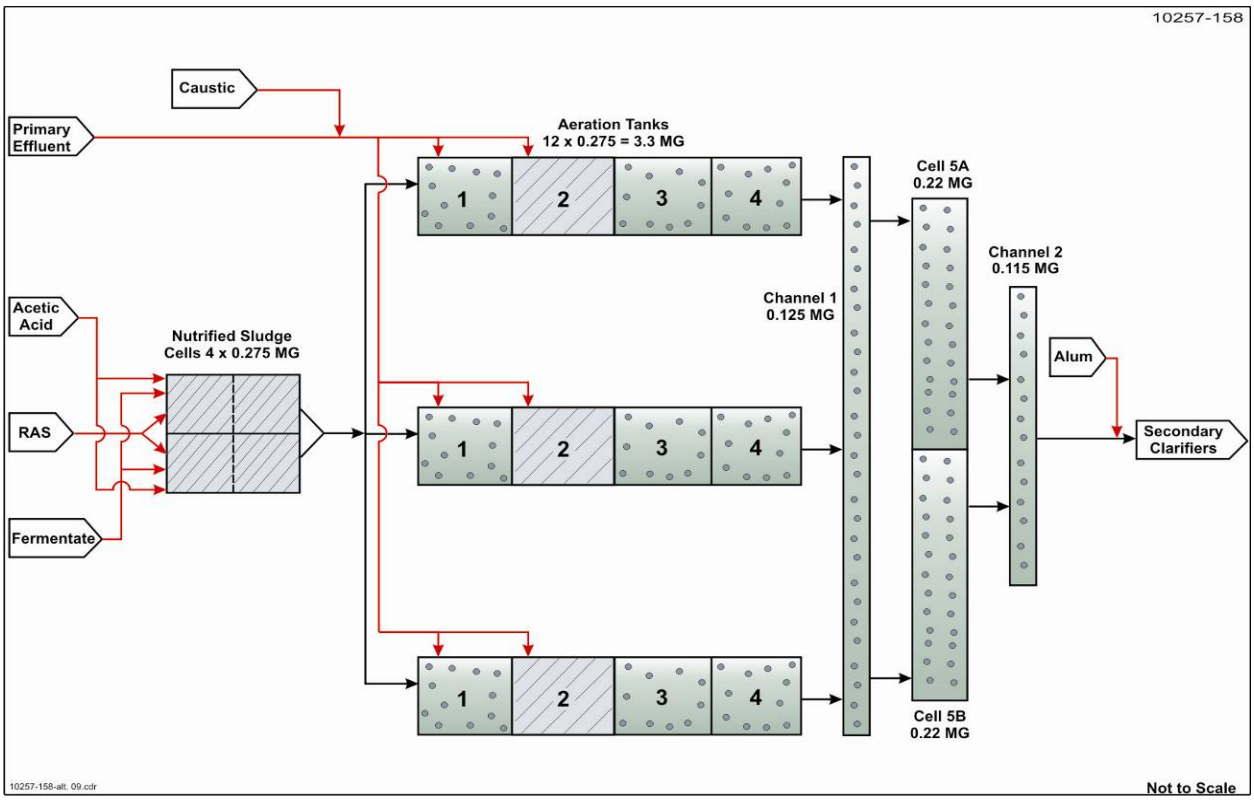
6.5.6 Alternative 6 – Operate Existing Process in Step Feed Mode

Alternative 6 includes operating the second cell of each aeration train anoxically and splitting the primary effluent flow to the first two cells. Feeding primary effluent directly to the second (anoxic) cell provides carbon for denitrification. When operated in the 3:4 configuration, the second and third cells of each train could be operated anoxically under warm weather and/or lower flow and load conditions to provide for a greater degree of denitrification. NSL Cells 1B and 1C would continue to receive RAS and fermentate, and acetic acid would still be fed to the NSL cells to supplement the fermentate for biological phosphorus removal. No capital improvements to the existing facilities related to nutrient removal are anticipated under this alternative.

Figure 6-6 presents a flow schematic of this alternative. Table 6-9 summarizes the predicted annual average and maximum month effluent quality and operational conditions for Alternative 6. This alternative assumes operation in the 3:4 configuration and step feed of 50% of the primary effluent flow under annual average and maximum month conditions. Annual average conditions assume anoxic operation of the second and third cell of each train under annual average conditions and anoxic operation of the second cell in each train under maximum month conditions.

Figure 6-6

Alternative 6 – Operate Existing Process in Step Feed Mode



THIS FIGURE REPRESENTS OPERATION UNDER MAXIMUM MONTH CONDITIONS.

Table 6-9
Predicted Operational Criteria and Effluent Quality – Step Feed Operation

Parameter	Annual Avg Design Flow	Max Month Design flow	Max Month Design Flow
Flow, mgd	11.6	14.5	14.5
Temperature, C	20	13	13
Aerobic SRT, days	5	7	6
MLSS Conc., mg/L	3,300	5,000	4,500
NRCY Flow	0%	0%	0%
RAS Flow	100%	100%	100%
Effluent NH3-N, mg/L	0.2	0.3	0.5
Effluent NOx-N, mg/L	8.3	11.6	11.4
Effluent sTKN, mg/L	1.6	1.8	2.1
Effluent TN, mg/L	10.0	13.5	13.6
Eff. TP w/o alum, mg/L	3.2	4.0	3.8
48% Alum Feed, gpd	970	1,400	1,200
25% NaOH Feed, gpd	1,400	2,700	2,400
20% HAc Feed, gpd	1,200	1,500	1,500

The process model predicted an annual average TN concentration of 10 mg/L under this alternative with a small increase in alum requirements. Sodium hydroxide feed requirements are significantly reduced due to alkalinity recovery through denitrification.

Maximum month MLSS concentrations are excessive under maximum month/minimum temperature conditions assuming operation at a 7-day aerobic SRT since a portion of the available aerobic volume is operated anoxically. Decreasing the aerobic SRT to 6 days under this condition decreases the MLSS concentration sufficiently to allow treatment of up to the 14.5 mgd design flow. The process model predicted that effluent ammonia concentrations would be approximately 0.5 mg/L at the decreased retention time, indicating sustained nitrification performance. It should be noted that operating at the decreased retention time will reduce the degree of conservatism in the nitrification design; however, OWASA still has the ability to provide additional aerobic volume by operating the anoxic zone as an aerobic zone during infrequent cold weather/high load events, while providing a greater portion of the required denitrification in the denitrification filters.

Aeration basin cells adjacent to the influent channel have two hydraulic gates that, when open, allow flow to enter the cell. Since there are multiple gates, the plant has the ability to step feed 50%, 33% or 67% of primary effluent depending upon the configuration of open gates. The BioWin process model for the following 3 scenarios was run to determine the best way to optimize the step feed alternative:

- 50/50 Flow Split – 50% of primary effluent to first cell, 50% of primary effluent to following cell
- 67/33 Flow Split – 67% of primary effluent to first cell, 33% of primary effluent to following cell
- 33/67 Flow Split – 33% of primary effluent to first cell, 67% of primary effluent to following cell

Results from the process modeling indicate that operating under the 33/67 Flow Split is not recommended due to insufficient hydraulic residence time for aerobic treatment and resulting elevated ammonia concentrations. Operation in the 67/33 Flow Split arrangement may be a viable option for the plant pending negative hydraulic implications. However, operation in the 50/50 flow split configuration is still considered the preferred configuration. Table 6-10 presents a comparison of the flow split options under annual average conditions.

Table 6-10
Predicted Operational Criteria and Effluent Quality – Step Feed Flow Split Scenarios

Parameter	50/50 Flow Split	67/33 Flow Split	33/67 Flow Split
Flow, mgd	11.6	11.6	11.6
Temperature, C	20	20	20
Aerobic SRT, days	5	5	5
MLSS Conc., mg/L	3,300	3,400	3,300
NRCY Flow	0%	0%	0%
RAS Flow	100%	100%	100%
Effluent NH3-N, mg/L	0.2	0.1	0.7
Effluent NOx-N, mg/L	8.3	8.0	9.5
Effluent sTKN, mg/L	1.6	1.6	2.1
Effluent TN, mg/L	10.0	9.7	11.7
Eff. TP w/o alum, mg/L	3.2	3.4	2.4
48% Alum Feed, gpd	970	1,010	870
25% NaOH Feed, gpd	1,500	1,500	1,500
20% HAc Feed, gpd	1,200	1,200	1,200

6.6 Nutrient Removal Alternatives Discussion

Each of the alternatives evaluated has the potential to reduce annual average secondary effluent total nitrogen concentrations to the 8-10 mg/L range, which represents a significant improvement over the existing configuration (14-18 mg/L). The additional denitrification provided by these options also provides for significant alkalinity recovery, and estimated sodium hydroxide feed rates would be approximately 30% lower than the baseline condition.

The impacts of operating Cell 5 anoxically were also evaluated using the process models. Since limited biodegradable carbon is available for denitrification in Cell 5, a 20% acetic acid feed to Cell 5 was assumed. Cell 5 has very limited volume (0.44 MG) for post-anoxic denitrification. The retention time (based on forward flow) would be under 1 hour at the 11.6 mgd annual average flow. Typical post-anoxic zone retention times are from 2 to 4 hours. Since insufficient volume exists in Cell 5, operating Cell 5 as a post-anoxic zone will not be considered further.

Alternatives 2 and 3 include utilizing the NSL basins for denitrification. The NSL cells have a total combined volume of approximately 1.1 million gallons. The resulting anoxic hydraulic retention times (based on forward flow) at the 14.5 mgd design flow under Alternatives 2 and 3 are 1.8 and 0.9 hours, respectively. Typical anoxic zone detention times are between three and four hours. The limited volume available for denitrification in the NSL cells results in significant supplemental carbon requirements in these zones (even when primary effluent is fed) to provide for effluent TN similar to the other alternatives. Alternatives 2 and 3 will likely also require the most extensive hydraulic modifications. These alternatives are expected to have little denitrification benefit unless excess supplemental carbon feed is applied, resulting in high operational costs. In light of the high operational costs and significant hydraulic modifications, these alternatives are removed from further consideration for nutrient removal improvements at the Mason Farm WWTP.

Alternatives 4, 5 and 6 were selected for further evaluation. Table 6-11 compares the predicted annual average effluent quality and operational criteria for each alternative to the current configuration (Alternative 1).

Table 6-11
Nutrient Removal Alternatives Comparison – Annual Average Design Flow

Parameter	Alternative 1 Exist. Process	Alternative 4 Int. Anoxic	Alternative 5 Ext. Anoxic	Alternative 6 Step Feed
Flow, mgd	11.6	11.6	11.6	11.6
Temperature, C	20	20	20	20
Aerobic SRT, days	5	5	5	5
MLSS Conc., mg/L	3,200	3,700	2,200	3,300
NRCY Flow	0%	300%	250%	0%
RAS Flow	50%	100%	100%	100%
Effluent NH3-N, mg/L	0.1	0.3	0.1	0.2
Effluent NOx-N, mg/L	16.4	6.7	8.0	8.3
Effluent sTKN, mg/L	1.4	1.8	1.6	1.6
Effluent TN, mg/L	17.9	8.5	9.6	10.0
Eff. TP w/o alum, mg/L	2.5	3.3	3.0	3.2

6.7 Odor Control Impacts

OWASA is implementing phased odor control improvements at the Mason Farm WWTP. Phase 1, completed in December 2007, included containing and treating emissions from the primary clarifier splitter box, aeration basin influent channel and influent pump station wet well. Phase 2, completed in September 2009, expanded odor containment and treatment to the primary clarifiers. The Phase 3 odor control improvements project has been designed to the 95% level and OWASA is committed to completing Phase 3 improvements by December 2014. The estimated construction cost of these improvements is $2,227,000 (December 2009). Phase 3 odor control improvements as currently designed include the following:

- Covering the NSL basins (Cells 1A through 1D), NSL influent channel, and aeration basin cells 1E, 2A, 3D and 4A
- Improvements to the existing aeration basin influent channel covers
- Construction of a 5,000 scfm carbon adsorption system to treat emissions from the NSL basins, NSL influent channel and NSL effluent pump station
- Construction of two (2) 9,000 scfm carbon adsorption systems to treat emissions from the four aeration basin cells and aeration basin influent channel

Optimizing nitrogen removal may require changing the design of the aeration basin odor control facilities in order to provide an appropriate level of odor emission reduction. In the current Phase 3 design, Cells 1E, 2A, 3D and 4A were selected for containment and treatment as these cells could potentially be operated as first aerobic cells. Optimizing nitrogen removal will increase the number of cells that may be operated either anoxically or as the first aerobic zone. The potential impacts to the Phase 3 odor control improvements design were evaluated and are summarized below:

- Alternative 4 (Internal Anoxic Zone) would require a total of nine cells to be covered and treated to provide for odor control of emissions from potential anoxic or first aerobic zones.
- Alternative 5 (External Anoxic Zone) would not require any additional aeration

basin cells to be covered or treated; however, the external anoxic zone would need to be covered and treated.

- Alternative 6 (Step Feed) would require that a total of six cells be covered and treated to provide odor control from both the first and second aerobic zones of the aeration basins.
- The NSL Basins would require covers and odor control equipment as previously planned.
- Two (2) 15,000 scfm carbon adsorption equipment and ancillary facilities should provide sufficient capacity to treat emissions associated with Alternatives 4 and 6.
- A detailed evaluation of the odor control improvements along with additional dispersion modeling is provided in Appendix O.
- The carbon adsorption unit for the NSL Basins should be sized for 6,700 cfm insteal of 5,000 cfm.
- The capacity of the units provided under Alternative 4 could be potentially reduced to 6,500 scfm each, and an additional 5,000 scfm treatment system would be required to treat the external anoxic zone.
- Table 6-12 presents a comparison of the maximum total airflow for Alternative 6 using jet aeration or fine bubble diffusers to the estimates developed by Brown and Caldwell during design. Values presented are representative of worst-case airflows and do not warrant reevaluating the design capacity of the proposed carbon adsorption equipment.

Table 6-12
Comparison of Maximum Total Airflow

Location	B&C Estimate (cfm)	Alternative 6 Jet Aeration (cfm)	Alternative 6 Fine Bubble (cfm)
Influent Channel	2,171	1,271	1,271
Cell 1E	3,525	5,119	3,216
Cell 2A	3,525	8225	825
Cell 3D	3,525	825	825
Cell 4A	3,525	5,119	3,216
Cell 2D		825	825
Cell 3A		5,119	3,216
Total	**16,271**	**19,104**	**13,395**

- It is recommended that flexibility be provided in the piping to allow either of the two proposed aeration basin carbon adsorption systems to treat any cell by providing a common suction header.

Potential capital cost impacts to the Phase 3 odor control project are summarized in Table 6-13. Alternative 1 costs are taken directly from the 95% design level estimate provided by OWASA dated December 21, 2009. The costs estimate for Alternative 5 assumes the external anoxic zone would be covered with an elevated concrete slab, which would be less expensive than providing aluminum covers similar to those proposed for the existing basins. The cost of the elevated slab is included in the concrete costs for Alternative 4.

Covering of additional two cells under Alternative 6 (step feed configuration) is expected to increase the cost of the Phase 3 odor control improvement project by approximately $500,000. Additional odor control improvements associated with Alternatives 4 and 5 are expected to be at least $500,000 more expensive than Alternative 6. The proposed step feed configuration will have the lowest cost impact of the nutrient optimization alternatives considered. Odor control impacts related to replacing the existing aeration system with fine bubble diffusers and vertical mixers is discussed in the aeration capacity alternatives in Section 6.4.

Table 6-13
Potential Phase 3 Odor Control Project Cost Impacts

Parameter	Alternative 1 Existing	Alternative 4 Internal Anoxic	Alternative 5 External Anoxic	Alternative 6 Step Feed
General Conditions	$192,000	$192,000	$192,000	$192,000
Demolition	$24,300	$24,300	$24,300	$24,300
Misc. Metals	$65,900	$65,900	$65,900	$65,900
Site Construction	$11,300	$11,300	$15,000	$11,300
Concrete	$52,100	$76,100	$465,000	$0
Aluminum Covers	$917,200	$1,477,200	$917,200	$1,141,200
Carbon Units/Fans/etc.	$348,600	$348,600	$400,000	$400,000
Ductwork/Supports	$268,400	$431,400	$560,400	$333,400
Electrical	$58,800	$58,800	$80,000	$58,800
Subtotal	**$1,939,000**	**$2,685,600**	**$2,719,800**	**$2,226,900**
Contingency (10%)	$194,500	$268,600	$272,000	$222,700
Subtotal	**$2,133,500**	**$2,954,200**	**$2,991,800**	**$2,449,600**
Startup	$7,000	$7,000	$9,000	**$7,000**
Insurance/Bonds (3.5%)	$86,500	$103,400	$104,700	**$85,700**
Total Capital Cost	**$2,227,000**	**$3,064,600**	**$3,105,500**	**$2,542,300**
Additional Engineering Costs	**$0**	**$226,000**	**$226,000**	**$185,000**
Additional Costs to Phase 3 Odor Project	**$0**	**$1,063,600**	**$1,104,500**	**$500,000**

6.8 Carbon Footprint Considerations

The relative carbon footprints of the treatment alternatives were compared to the existing configuration. Greenhouse gas (GHG) accounting protocols differentiate between the following three scopes of emissions:

- Scope 1 emissions – These are direct emissions from a facility and include emissions from fuels combusted on site, direct emissions from the treatment process, digester emissions and emissions from digester gas flaring.

- Scope 2 emissions – These are the emissions associated with production of electrical power used at the facility but generated off-site (i.e. at the power plant). Scope 2 emissions are heavily influenced by aeration and pumping requirements at wastewater treatment plants.
- Scope 3 emissions – These include indirect emissions from a facility's operations and can include transportation emissions from employees, delivery vehicles, emissions attributed to the production and transport of consumables such as chemicals, etc. These are very difficult to quantify and most GHG accounting protocols have focused on Scope 1 and Scope 2 emissions.

Scope 1 emissions from the secondary process include carbon dioxide and nitrous oxide emissions to the atmosphere from basin off-gas. The BioWin model was used to estimate carbon dioxide off-gas quantities and predicted similar loads for each alternative. Carbon dioxide emissions from the optimization alternatives are omitted from the evaluation as they are biogenic in origin and would be naturally released in the carbon cycle. Nitrous oxide emissions from various treatment configurations are a topic of much current research, and models (including work sponsored by the Water Environment Research Foundation) are currently being developed to predict these emissions; however these models are not yet available to the industry, and nitrous oxide emissions from the various alternatives were not quantified. It would be expected, however, that the alternatives would have similar nitrous oxide emissions. Reactor off-gas testing could be used to determine nitrous oxide emissions if desired.

The carbon footprint evaluation focuses on the Scope 2 emissions of the nutrient optimization alternatives. These are easily quantifiable as they are directly dependent on the energy usage for each process. The total major energy inputs for each alternative including aeration, RAS pumping, NRCY pumping (if applicable), NSL basin effluent pumping and mixing were determined, and the overall energy demand was calculated. The GHG emissions associated with power consumption are based on the USEPA eGrid2007 Version 1.1 emissions data for North Carolina. The emissions data is based on Year 2005 emissions and is the most recent data in the USEPA inventory.

Table 6-14 summarizes the total annual power demand at the design year (corresponding to an 11.6 mgd annual average flow) and resulting greenhouse gas

emissions. Greenhouse gas emissions evaluated included carbon dioxide (CO_2), methane (CH_4) and nitrous oxide (N_2O). It is assumed that the amount of the other known greenhouse gas compounds, fluorinated gases (CFCs, HCFCs and halons), are negligible, and they were not considered in this evaluation.

Table 6-14
Nutrient Removal Alternatives Comparison – Scope 2 GHG Emissions

Parameter	Alternative 1 Exist. Process	Alternative 4 Int. Anoxic	Alternative 5 Ext. Anoxic	Alternative 6 Step Feed
Power Demand (MW-hr/year)	7,750	8,540	9,290	7,750
CO_2 Emissions (ton/year)	4,750	5,230	5,690	4,745
CH_4 Emissions (ton/year)	0.08	0.09	0.09	0.08
CO_2 Equivalents	21	21	21	21
CH_4 Emissions (ton CO_2/year)	2	2	2	2
N_2O Emissions (ton/year)	0.08	0.09	0.10	0.08
CO_2 Equivalents	310	310	310	310
N_2O (ton CO_2/year)	26	28	31	26
Total Emissions (ton CO_2/year)	**4,780**	**5,260**	**5,720**	**4,770**

Carbon dioxide emissions from off-site electrical power generation account for over 99% of the Scope 2 emissions, and methane and nitrous oxide emissions from power generation are relatively insignificant. The current configuration has a similar Scope 2 carbon footprint to Alternative 6. Even though aeration requirements are greater for the current configuration, twice as much RAS flow (which also increases NSL effluent pumping requirements) is assumed for Alternative 6, and pumping energy requirements are higher. Alternatives 4 and 5 have a greater carbon footprint due to NRCY pumping requirements. Alternative 5 has the greatest emissions due to additional mixing requirements associated with the external anoxic zone.

Scope 3 emissions for Alternatives 4 through 6 are expected to be lower than the current configuration due to decreased impacts of GHGs from chemical production and transportation. Although these impacts are not readily quantifiable, it is expected that Alternatives 4 through 6 would have similar Scope 3 emissions since chemical feed requirements for these alternatives are similar. In summary, Alternative 6 is expected to have the lowest carbon footprint of the nutrient removal optimization alternatives since

this option does not require NRCY pumping for denitrification. Scope 2 emissions are similar to the current configuration, and Scope 3 emissions are expected to be similar to Alternatives 4 and 5.

6.9 Alkalinity Considerations and Impacts on Reuse

A portion of the Mason Farm WWTP effluent is reclaimed and pumped to the University of North Carolina for irrigation, toilet flushing and cooling tower make-up water. The latter application requires an alkalinity between 50 and 75 mg/L as $CaCO_3$ at the application site to reduce the potential for cooling tower fouling. A sulfuric acid feed system was recently installed to reduce the alkalinity of the reclaimed water as necessary.

The historical Mason Farm WWTP effluent alkalinity is 75 mg/L as $CaCO_3$. OWASA currently feeds 25% sodium hydroxide to maintain this alkalinity and optimize conditions for nitrification. The denitrification process results in a gain of 3.6 mg/L (as $CaCO_3$) of alkalinity per mg-N/L nitrate removed. Denitrification across the filters to meet the effluent TN standards will result in an increase in effluent alkalinity, which will require additional sulfuric acid feed. As flows increase, lower effluent nutrient concentrations are required to meet the mass load allocations, and there is further reliance on the denitrification filters to meet the limits and a greater potential for increased effluent alkalinity. Providing for denitrification in the secondary process will not only reduce alkalinity feed requirements, but it will limit the amount of alkalinity formed in the denitrification filters, reducing the amount of sulfuric acid necessary.

Alkalinity profiles through the treatment plant were performed for Alternatives 4, 5 and 6 and were compared to maintaining current operations. Sodium hydroxide feed is based on maintaining a secondary effluent alkalinity of 75 mg/L as $CaCO_3$. The alkalinity balance included influence of the filtrate and fermentate streams, alum feed, nitrification and denitrification. Table 6-15 presents the alkalinity profile in mg/L as $CaCO_3$.

Table 6-15
Alkalinity Profile Comparison (in mg/L as $CaCO_3$)

Location	Alternative 1 Exist. Process	Alternative 4 Int. Anoxic	Alternative 5 Ext. Anoxic	Alternative 6 Step Feed
Plant Influent	175	175	175	175
Primary Clarifier Influent	174	172	173	172
Primary Clarifier Effluent	174	172	173	172
Aeration Basin Influent	253	216	221	220
Secondary Effluent	76	75	75	75
Filtered Effluent	133	95	100	102

Providing greater denitrification capacity in the secondary process results in significantly lower effluent alkalinity. The effluent alkalinity is expected to be between 30 and 40 mg/L lower than if the current operations were continued and the denitrification filters were relied upon for the bulk of TN removal. Substantial sodium hydroxide and sulfuric acid savings can be realized by providing for denitrification in the secondary process. Additional reduction in supplemental alkalinity feed and effluent alkalinity may be possible by lowering the target secondary effluent alkalinity. Nitrification performance should be monitored to ensure lowering the secondary process alkalinity does not lead to low pH inhibition.

6.10 Nutrient Removal Optimization Cost Comparison

Table 6-16 compares the expected capital costs of each nutrient optimization alternative. Capital costs include structures, NRCY pumping facilities, piping and hydraulic improvements associated with each alternative. A contingency of 20% was assumed, and engineering, administration and construction services were assumed to be 20%. Additional Phase 3 odor control improvement costs developed previously were included to provide a total capital cost of required improvements.

Table 6-16
Nutrient Removal Alternatives Comparison – Capital Costs

Parameter	Alternative 4 Internal Anoxic	Alternative 5 External Anoxic	Alternative 6 Step Feed
External Anoxic Zone	$0	$2,000,000	$0
NRCY Pump Wetwell	$140,000	$140,000	$0
NRCY Pumps	$800,000	$1,000,000	$0
NRCY Piping/Mechanical	$100,000	$200,000	$0
Hydraulic Modifications	$100,000	$200,000	$0
Electrical Improvements	$100,000	$150,000	$0
Subtotal	**$1,240,000**	**$3,690,000**	**$0**
Contingency (20%)	$248,000	$738,000	$0
Subtotal	**$1,488,000**	**$4,428,000**	**$0**
Engineering and Administration (20%)	$298,000	$886,000	$0
Subtotal	**$1,786,000**	**$5,314,000**	**$0**
Additional Phase 3 Odor Control Work	$1,063,600	$1,104,500	$500,000
Total Capital Costs	**$2,849,600**	**$6,418,500**	**$500,000**

Annual chemical costs are compared in Table 6-17. This evaluation included acetic acid feed to the denitrification filters to reduce effluent nitrate concentrations below 1 mg/L to meet the 3 mg/L TN limit. As previously discussed significant additional savings could be realized through methanol addition to the denitrification filters. An average annual flow of 9.5 mgd was assumed and is based on the expected approximate average flow between 2016 and 2035. Sulfuric acid savings was estimated assuming an average cost of $4.00/gallon, a target effluent alkalinity of 50 mg/L and an average reclaimed water flow of 1.35 mgd between 2010 and 2030. Additional chemical costs are based on delivered costs quoted by Brenntag on 12/31/2009 and are as follows:

- 48% alum - $1.11/gallon
- 25% sodium hydroxide - $0.65/gallon
- 20% acetic acid - $0.83/gallon

Table 6-17
Nutrient Removal Alternatives Comparison – Chemical Feed Requirements

Parameter	Alternative 1 Existing	Alternative 4 Internal Anoxic	Alternative 5 External Anoxic	Alternative 6 Step Feed
Feed Rates (gpd)				
48% Alum	670	740	710	720
25% Sodium Hydroxide	1620	900	960	1040
20% Acetic Acid	3170	1840	1990	2070
93% Sulfuric Acid	60	30	30	30
Annual Chemical Costs				
48% Alum Feed	$267,000	$296,000	$282,000	$288,000
25% NaOH Feed	$384,000	$211,000	$228,000	$246,000
20% HAc	$967,000	$558,000	$612,000	$625,000
93% Sulfuric Acid	$85,000	$43,000	$49,000	$52,000
Total Chemical Costs	**$1,701,000**	**$1,106,000**	**$1,170,000**	**$1,208,000**

Alternatives 4, 5 and 6 have fairly comparable chemical feed requirements (within 10%). Each alternative is expected to require substantially less chemical than continuing current operations, particularly with respect to supplemental carbon addition. Potential chemical savings (at current chemical costs) are estimated to average more than $500,000 annually over the next 20 years.

Electrical power costs associated with aeration, jet mixing, RAS pumping and NRCY pumping were evaluated for each alternative and compared to the current operations. Table 6-18 summarizes the estimated power requirements and costs assuming an average annual flow of 9 mgd. A power cost of $0.06/kilowatt-hour (kWh) was assumed, and the analysis assumes energy costs will increase at the same rate of inflation.

Table 6-18
Nutrient Removal Alternatives Comparison – Estimated Energy Costs

Parameter	Alternative 1 Existing	Alternative 4 Internal Anoxic	Alternative 5 External Anoxic	Alternative 6 Step Feed
Power Demands (kW)				
Aeration	272	243	245	247
RAS Pumping	25	47	47	47
NRCY Pumping	0	72	60	0
Mixing	483	483	579	483
Total Power (kW)	780	845	931	778
Annual Power Cost	**$409,719**	**$444,077**	**$489,555**	**$408,723**

Table 6-19 provides a present worth cost comparison of the nutrient removal alternatives. Present worth costs assume a twenty-year period and 5% return rate. All costs given are in 2010 dollars. As shown below, providing step feed within the existing treatment volume (Alternative 6) is the most cost effective approach for nutrient removal optimization through the next 20 years.

Table 6-19
Nutrient Removal Alternatives Comparison – Present Worth Analysis

Parameter	Alternative 1 Existing	Alternative 4 Internal Anoxic	Alternative 5 External Anoxic	Alternative 6 Step Feed
Capital Costs	-	$2,849,600	$6,418,500	$500,000
Annual Chemical Costs	$1,701,000	$1,106,000	$1,170,000	$1,208,000
Annual Energy Costs	$409,719	$444,077	$489,555	$408,723
Total Operational Cost	$2,110,719	$1,550,077	$1,659,555	$1,616,723
PW Operational Cost	$26,304,224	$19,317,386	$20,681,723	$20,147,942
Net Present Worth Cost	**$26,304,224**	**$22,166,986**	**$27,100,223**	**$20,647,942**

6.11 Nutrient Removal Optimization Recommendations

It is recommended that the secondary treatment process be operated in the step feed configuration (Alternative 6) discussed herein to reduce effluent TN concentrations to the denitrification filters. Figure 6-7 presents the configuration of the proposed step feed process. The existing facilities have sufficient capacity to operate in the step feed configuration and treat up to the 14.5 mgd maximum month design flow. Operation in this configuration under current conditions will also reduce sodium hydroxide consumption and aeration energy. Acetic acid addition for biological phosphorus removal is expected to be reduced as well since less nitrate will be returned to the NSL basins with the RAS.

Figure 6-7
Proposed Step Feed Configuration

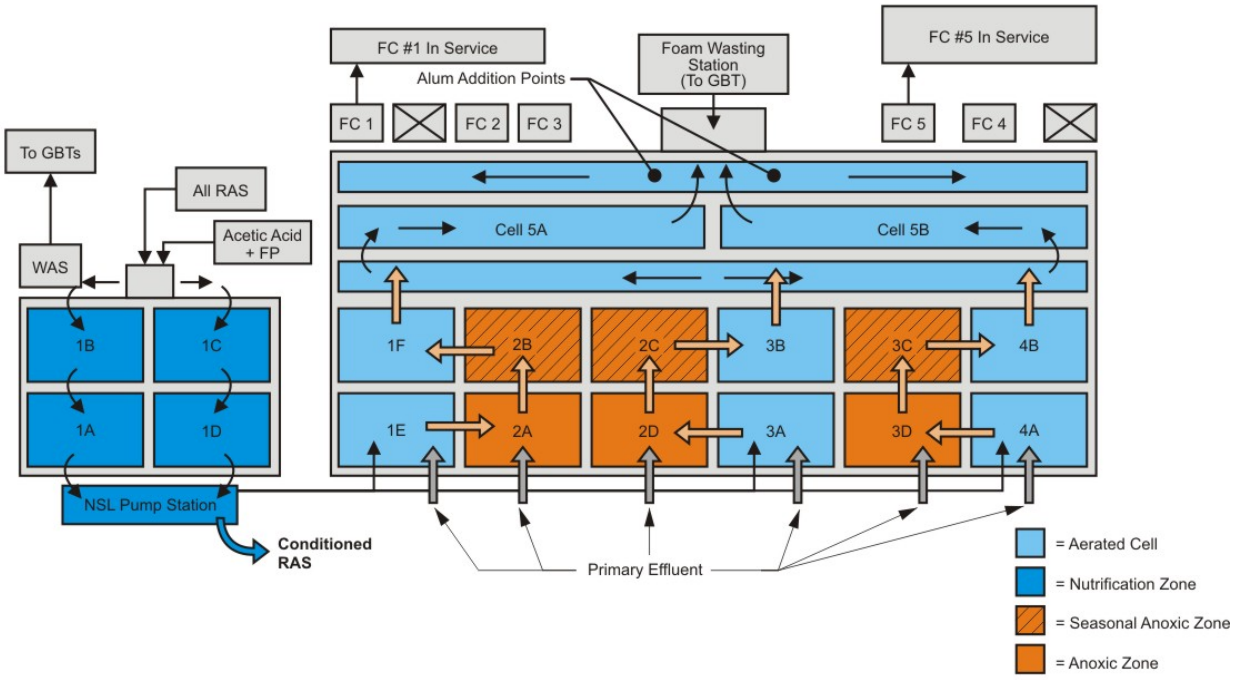


Table 6-20 compares the recommended alternative to continued operation in the current configuration. Acetic acid feed rates include denitrification filter requirements to meet a TN of 3 mg/L.

Table 6-20
Nutrient Removal Alternatives Comparison – Annual Average Design Flow

Parameter	Annual Average Conditions		Maximum Month Conditions	
	Alternative 1 Exist. Process	Alternative 6 Step Feed	Alternative 1 Exist. Process	Alternative 6 Step Feed
Flow, mgd	11.6	11.6	14.5	14.5
Temperature, C	20	20	13	13
Aerobic SRT, days	5	5	7	6
MLSS Conc., mg/L	3,200	3,300	4,600	4,500
RAS Flow	50%	100%	50%	100%
Effluent NH3-N, mg/L	0.1	0.2	0.1	0.5
Effluent NOx-N, mg/L	16.4	8.3	20.7	11.4
Effluent sTKN, mg/L	1.4	1.6	1.6	2.1
Effluent TN, mg/L	17.9	10.0	22.5	13.6
Eff. TP w/o alum, mg/L	2.5	3.2	2.8	3.8
48% Alum Feed, gpd	900	970	1,130	1,200
25% NaOH Feed, gpd	2,200	1,400	4,000	2,400
20% HAc Feed, gpd	4,300	2,800	6,300	4,100

In addition to the proposed operational changes to increase TN removal in the secondary process, several improvements to the chemical phosphorus removal system are recommended. An additional alum feed point should be provided at the effluent filter influent to allow for additional phosphorus trim prior to discharge. Multi-point chemical addition will optimize alum or other chemical feed requirements, and increase the reliability of phosphorus removal at the Mason Farm WWTP by providing another barrier.

OWASA currently uses the ChemScan nutrient analyzer system to monitor filter influent nitrate and filter effluent ammonia, nitrate and phosphorus. It is recommended that an additional phosphorus monitoring point be added at the secondary effluent/filter influent (prior to the proposed alum feed at the filters) to allow OWASA to optimize chemical feed between the secondary clarifiers and filters. Decreasing alum addition to the MLSS to adjust for the additional feed point at the filters will also decrease the amount of alkalinity

required to be fed to the secondary process. Furthermore, alum feed to the filters will reduce the alkalinity of the reclaimed water. Care must be taken to limit the alum feed to the filters to prevent filter blinding and decreased run time; however it is estimated a target secondary effluent TP concentration between 0.5 mg/L and 1 mg/L will allow for sufficient reduction through the filter to meet the 0.23 mg/L effluent TP limit without significant operational impacts on the filters.

Sodium aluminate feed should also be considered to reduce the amount of caustic soda addition required to the aeration basin influent channel. The existing alum storage and feed facilities were evaluated and it appears the existing facilities are compatible with use of sodium aluminate. Further discussion of the potential advantages and disadvantages of sodium aluminate addition over alum are discussed in Section 8.3.

6.12 Supplemental Carbon Evaluation Alternatives

OWASA currently feeds 20% acetic acid to the NSL basin to supplement the carbon available in the fermentate for RAS denitrification and biological phosphorus removal. In addition acetic acid can be added to the deep bed filters to provide for denitrification. Recent studies by Hazen and Sawyer indicate that acetic acid is a relatively expensive supplemental carbon source, and more economical options are available. Appendix J evaluates the various supplemental carbon options including acetic acid, methanol, and glycerin on both a process and cost basis.

The analysis provided in Appendix J indicates that feeding methanol to the denitrification filters would provide for substantial cost savings over acetic acid addition. Methanol feed is estimated to result in an annual chemical cost savings of approximately $300,000 per year at the design year and $150,000 in the first year of compliance with the Jordan Lake Rules for nitrogen removal (2016) assuming operation in the recommended step feed configuration. The total 20-year net present worth savings of chemical costs of methanol feed over 20% acetic acid are estimated to be $3.8 million. Methanol is well proven in its application to denitrification filters, and fouling problems due to aerobic biomass growth are eliminated.

Construction of new methanol storage and feed facilities would be necessary since the existing acetic acid facility is not compatible with methanol. There are safety concerns associated with the use of methanol; these concerns are discussed in detail in Appendix J. The estimated cost of a new methanol feed facility is $1.0 million, which assumes 304 stainless steel construction to provide the flexibility to feed glycerin or other carbon sources. The approximate payback period for construction of methanol feed facilities is 4 years. The construction cost of the methanol feed facility could be significantly less if carbon steel is used instead of 304 stainless steel. There is also potential that use of glycerin in lieu of acetic acid in the NSL cells would reduce chemical costs. Some of the existing acetic acid tanks could potentially be used to store glycerin or other products pending a structural evaluation of the tanks.

There is potential for OWASA to provide flexibility to select a carbon source based on current market conditions rather than be limited to a single carbon source. However, in the case of methanol, transitioning from another carbon source will require an extended period of time since methylotrophs, the microorganisms that utilize methanol as a substrate, require several weeks to establish themselves. Transitioning between other carbon sources would have minimal process implications since these other carbon sources do not require specialist populations to accomplish denitrification.

It is recommended that OWASA investigate the potential to utilize methanol or another supplemental carbon source in lieu of acetic acid for to reduce the chemical costs associated with carbon feed to the denitrification filters. Potentially, OWASA could modify a portion of the acetic acid feed facilities to allow storage and feed of glycerin or other carbon sources to provide flexibility to adjust to market conditions for supplemental carbon and reduce overall operations costs with only a modest capital investment.

6.13 Secondary Treatment Process Capacity Expansion Evaluation Alternatives

Section 4 discusses the capacity limitations of the secondary treatment process. The secondary treatment process at the Mason Farm WWTP can satisfactorily treat up to the current 14.5 mgd permitted capacity at current wastewater strength conditions provided that peak sustained 24-hour flows do not exceed 40 mgd. This rating excludes the

limitation of the existing aeration system discussed previously and assumes that conversion to a step feed biological process configuration to optimize nutrient removal has been implemented. Careful secondary process operation will be necessary in order to process peak wet weather flows associated with a 14.5 mgd maximum month flow. Polymer feed to aid settling and step feed of flow to Cells 5A and 5B to reduce solids loading rates to the secondary clarifiers will be required to reduce the risk of clarifier failure.

The existing secondary treatment process has inadequate capacity to treat peak flow and load conditions above the current 14.5 mgd design flow or at lower flows if wastewater strength increases. This section discusses the alternatives to improving the reliability and expanding the capacity of the existing secondary process at the Mason Farm WWTP with an emphasis on reusing existing tankage. An expansion to a permitted flow of 18.5 mgd is identified as the next step in expanding the facility, was considered in the evaluations.

6.13.1 Flow Equalization

The Mason Farm WWTP was designed to treat a peak flow of 3.0 times the design maximum month flow, for a total peak flow capacity of 43.5 mgd. Process and clarifier modeling performed and discussed in Technical Memorandum No. 3 indicate that peak flows at the Mason Farm WWTP will need to be limited to 40 mgd to prevent secondary clarifier failure. This results in a maximum allowable peaking factor of 2.75 times the design maximum month flow. This peak day factor is relatively high and greater than that required by the State of North Carolina.

Flow equalization was considered to improve the reliability of the secondary treatment process. Providing for flow equalization upstream of secondary treatment will reduce the hydraulic and solids loadings to the secondary clarifiers during wet weather flow events and allow operation of the secondary treatment process at a higher MLSS concentration. Locating flow equalization facilities upstream of the primary clarifiers will also provide for reduced peak primary clarifier loadings, which will improve solids removal performance. Flow equalization can also reduce hydraulic constraints and free capacity in the pump stations, tertiary filters, UV disinfection and flow measurement facilities.

Required flow equalization volumes were estimated using the design peak wet weather event hydrograph developed in Section 2 and are summarized in Table 6-21. Equalization requirements to limit the peak flow factor to 2.0 and 2.5 were calculated along with the required volume to limit the peak wet weather flow to 40 mgd (2.75 peak factor). Equalization volumes were also estimated for the future 18.5 mgd design flow.

Table 6-21
Required Equalization Volume

	2.0 Peak Factor	2.5 Peak Factor	2.75 Peak Factor
14.5 mgd Design Flow	19.5 MG	9.3 MG	4.2 MG
18.5 mgd Design Flow	24.9 MG	11.9 MG	5.5 MG

Limiting peak wet weather flows below a 2.5 peaking factor requires substantial flow equalization volume. Since area for additional facilities at the Mason Farm WWTP is limited, providing substantial flow equalization facilities on-site will be difficult. OWASA is currently preparing a collection system study to investigate peak flow events and inflow/infiltration (I/I) impacts. Reducing I/I in the collection system may be a better long term solution to reduce peak flows through or potentially increase the dry weather flow capacity of the Mason Farm WWTP.

As flows to the plant increase and investment in I/I reduction is continued, the peak flow factor to the plant will likely decrease. Construction of flow equalization facilities at this time is unnecessary to meet the current effluent limits but should be considered during the next expansion to the Mason Farm WWTP. The need for flow equalization will be dependent on future collection system conditions and technologies installed to meet future flows and/or effluent limits. Membrane technologies such as MBR will require flow equalization down to a peaking factor of 2.0 or less to be economical.

6.13.2 Secondary Clarifier Modifications

The hydraulic analysis presented in Section 3 indicates uneven flow distribution to the secondary clarifiers during peak flow events results in excessive flow to Secondary Clarifiers 2 and 5. In addition Secondary Clarifiers 2 and 3 are shallow and have relatively undersized feed wells. These issues would need to be addressed in a capacity expansion to 18.5 mgd.

6.13.3 Additional Secondary Clarification Capacity

The secondary process is limited by the existing secondary clarifier capacity. Construction of additional secondary clarifiers will allow for treatment of additional flow and increased MLSS concentrations. One new secondary clarifier of the same size and configuration as secondary clarifier 5 would be required to treat a design follow of 18.5 mgd. While there are site constraints on the Mason Farm site, there is space for additional secondary clarifiers to support the expansion.

6.13.4 Additional Aeration Basin Capacity

Construction of additional aeration basin capacity will allow for operation at a lower MLSS concentration to achieve full nitrification and will lower the solids loading to the secondary clarifiers. Additional aeration basin volume should be provided to provide for a maximum month MLSS concentrations to 3,500 mg/L at the 18.5 mgd design flow. One additional aeration train of four cells would be provided.

6.13.5 Conversion of Nutrified Sludge Tank to Aeration Basin Capacity

Conversion of the existing NSL basin to provide additional aerobic volume was considered. This option will require that primary effluent flow be rerouted to the NSL cells. Additional chemical phosphorus removal would be required under this option, and increased nitrate loads would need to be processed through the denitrification filters. The hydraulic profile in the NSL cells would need to be increased to allow for gravity flow of NSL effluent to the aeration basin influent channel to eliminate the need to repump the flow to the aeration basins. The hydraulics through the NSL basins would also need to be improved to accommodate the increased flow through the basins.

Process modeling indicates that this option can provide sufficient nitrification capacity at the 18.5 mgd maximum month conditions assuming current wastewater strength while maintaining an MLSS concentration of 4,100 mg/L. All tankage would be operated aerobically under maximum month conditions, resulting in effluent TN concentrations near 30 mg/L. This would have a significant impact on the ability of the denitrification filters to reduce TN concentrations under 2.3 mg/L to meet the nutrient mass load allocation at 18.5 mgd and likely result in difficulties in reliably meeting effluent nutrient limits. Elevated TN concentrations may also cause problems with floating sludge in the

secondary clarifiers. Providing for external, upstream anaerobic and anoxic basins would be required for biological phosphorus removal and TN reduction under this alternative. Fermentate could be redirected to the external anaerobic zone to provide additional carbon.

Converting the NSL cells to aeration volume may provide the Mason Farm WWTP with ample nitrification capacity through a design flow of 18.5 mgd. Converting the NSL cells to aeration volume eliminates RAS denitrification and biological phosphorus release in these cells, and upstream anaerobic and anoxic zones would need to be constructed to reduce chemical costs and reliably meet the current effluent mass load allocations. Separate anaerobic and anoxic zones would be constructed in the area where the abandoned trickling filters are currently located. It is recommended that this option be considered during preliminary design of the expansion to 18.5 mgd assuming no significant change in effluent limits.

6.13.6 Integrated Fixed Film Activated Sludge (IFAS)

Integrated Fixed Film Activated Sludge (IFAS) processes use either fixed or floating media to increase the nitrification capacity of a given basin volume. The media can be added to an existing aeration basin to provide for fixed film biomass (particularly nitrifier) growth. This increase in biomass effectively increases the solids retention time of the process and allows for greater nitrification capacity for a given volume and MLSS concentration. Addition of IFAS media to the existing aeration basins may allow for an uprating of the existing aeration basins by allowing more flow to be treated at a given MLSS and/or allowing operation at a reduced MLSS concentration, which would decrease the solids loading to the secondary clarifiers.

Hazen and Sawyer recently piloted an IFAS process utilizing floating media at the City of Greensboro T.Z. Osborne WRF. The full-scale pilot indicated an approximate 50% increase in treatment capacity in the IFAS reactor compared to a similarly sized control reactor without media at similar MLSS concentrations. Numerous operational issues including foaming, excessive headloss through media retention screens and debris accumulation were encountered, but overall the pilot test indicated that IFAS is a viable technology for increasing secondary treatment capacity.

Addition of IFAS media to the existing aeration basins may allow the facility to be

expanded to 18.5 mgd without construction of additional aeration basin volume. This is a particularly attractive solution as limited area exists on-site for new facilities. The configuration of the basins may be particularly well suited as individual cells can be isolated and used for media storage during maintenance activities. Particular attention would need to be focused on the available freeboard and hydraulic head available in the basins as significant headloss may occur through the media retention screens. In addition, current IFAS floating media technologies require coarse bubble diffusers to provide adequate media scouring. Investment providing fine bubble diffused aeration equipment may be lost if IFAS is selected in the future, and coarse bubble diffused aeration will likely result in greater aeration requirements and energy costs. Fixed media systems do allow for fine bubble diffused aeration but have lower potential capacity per unit volume. Installation of IFAS at the Mason Farm WWTP should be revisited during preliminary engineering of the future expansion to 18.5 mgd.

6.13.7 Membrane Bioreactors

Membrane bioreactors (MBR) essentially replace the secondary clarification and filtration step in an activated sludge process with a permeable membrane that separates the solids from the liquid. MBR processes are typically operated at much greater MLSS concentrations (8,000 mg/L to 10,000 mg/L) than conventional activated sludge processes, and increased particulate phosphorus and nitrogen removal will reduce overall effluent TN and TP concentrations. Drawbacks to the MBR process include increased operating costs, particularly in regards to membrane scour air requirements, and the need for equalization and ultrafine (1-2 mm spacing) screening to protect the membranes. Since the Mason Farm WWTP already has secondary clarifiers and tertiary filters, the investment in the facilities may be lost unless they can be reused as membrane tanks or other process volume.

It is unlikely that converting the existing facility to MBR will be an economical alternative from both a capital and operational cost perspective under the current Jordan Lake nutrient regulations. The MBR process can serve as an effective means for pretreatment of wastewater requiring treatment through nanofiltration (NF) or reverse osmosis (RO), which may be required to meet future stringent nutrient limits (below 2 mg/L TN and/or 0.1 mg/L TP) or remove emerging contaminants. In this case converting the existing process to MBR may be technically and economically viable.

A preliminary evaluation was performed to determine whether sufficient capacity was available in the existing aeration basins to provide for nutrient removal at 18.5 mgd using MBRs. Process simulations were based on converting the existing tankage to provide both pre- and post-anoxic zones and increasing the MLSS concentration to approximately 8,000 mg/L. The process simulations predicted effluent annual average TN concentrations below 2.3 mg/L with carbon feed to the post-anoxic zone, meeting the effluent TN mass load allocation at 18.5 mgd. It should be noted that MBR processes are typically not followed by filtration, as the membrane produces a much lower effluent TSS than filters. Elimination of the filters from the Mason Farm WWTP removes a secondary nitrate removal barrier, reducing process redundancy.

The NSL basins could continue to be used for RAS denitrification and biological phosphorus removal. Chemical phosphorus removal would likely still be required. Membrane tank sizing criteria was based on a flux rate of 12 gpd/sf at design flow. An estimated footprint of 11,000 sf would be required for the membrane tanks, and an additional 18,000 sf would be required for the membrane equipment room, which houses membrane blowers, permeate pumps, recycle pumps, cleaning equipment and ancillary MBR facilities.

Converting the existing secondary process to MBR should be considered during preliminary design of the next expansion to the Mason Farm WWTP if more stringent nutrient regulations, reclaimed water requirements or emerging contaminant regulations require incorporation of a membrane process. Conversion of the existing secondary tankage to an MBR process should be sufficient through a design flow of 18.5 mgd based on current wastewater strengths. New membrane tanks and a membrane building would need to be constructed unless the existing clarifiers or filters could be modified to house the membranes. Using the existing tankage for the membrane reactors may not be possible due to constructability and maintenance of operations concerns.

6.13.8 Secondary Treatment Capacity Expansion Summary

The Mason Farm WWTP can likely be expanded to 18.5 mgd using conventional treatment by adding aeration basins and secondary clarifiers. Alternately, the existing NSL basins could be converted to aeration volume and new anaerobic and anoxic zones could be constructed upstream. The Mason Farm WWTP could also be expanded to 18.5 mgd without construction of additional aeration basin volume by incorporating IFAS

or MBR technologies. Ancillary facilities would need to be constructed including screening, flow equalization, membrane tanks and a membrane building for the MBR option. Media retention facilities and possible hydraulic modifications would be necessary to incorporate IFAS. Additionally, current IFAS systems require coarse bubble diffused aeration and use of IFAS media with the existing jet aeration system or the recommended fine bubble diffused aeration system may not be feasible. Additional clarifier capacity would be required for all expansion options with the exception of the MBR option.

Conventional treatment similar to the existing process, or potentially IFAS, would have a lower capital and operating cost than MBR for a plant expansion to 18.5 mgd. Future regulatory drivers including stricter nutrient regulations, reclaimed water requirements and removal of emerging contaminants will dictate the technologies to be considered and ultimately selected at future flows. Potential future regulations and/or the need for more advanced treatment may require the use of membrane separation technology such as MBR. The MBR option will likely be most economical if future regulatory drivers require lower effluent nutrient limits than the current mass load allocations or if removal of emerging contaminants is required.

Section 7
Solids Treatment Alternatives Evaluation

7.1 Goals of Process Optimization

The goals for optimization of the treatment process at the Mason Farm WWTP are multi-faceted. Of primary importance to OWASA is to reliably produce a high quality effluent discharge that is protective of the environment and in compliance with permit requirements. It is also desirable to minimize operating costs for the Plant and to minimize the carbon footprint of the Plant as well. Since some capacity deficiencies have been identified, it is also a goal of this study to determine ways to maximize the treatment capacity of the Plant. Evaluation alternatives will consider the constraints of current and expected future permit limits, requirements of OWASA's reclaimed water system contract with the University of North Carolina, OWASA's odor elimination goals, and requirements for producing Class A biosolids. The evaluations shall accommodate the fact that permit limits for TN and TP are annual mass limits.

7.2 Solids Treatment Alternatives

The solids handling facilities for the Mason Farm WWTP have some deficiencies that will prevent the plant from achieving its maximum month capacity rating of 14.5-mgd. Mason Farm WWTP solids handling process improvement alternatives to remedy the deficiencies are evaluated in this section. Major unit processes evaluated in this section include:

- Primary Sludge Fermentation
- Digester Feed Sludge Thickening
- Anaerobic Digestion
- Biosolids Storage
- Biosolids Dewatering
- Digester Gas Utilization

Annual average and maximum month solids production rates from the process evaluation were used to evaluate unit process performance needs. Additional information on unit process performance needs are also described in Section 5. Solids production rates at current wastewater strength are summarized in Table 7-1. Additional discussion on solids production rates developed from the calibrated BioWin model, as compared to solids production rates developed by Brown and Caldwell in the 2002 facility design, is also provided in Section 5.

Table 7-1
Estimated Solids Production Rates – Current Wastewater Strength

	Annual Average (lb/MG Treated)	Maximum Month (lb/MG Treated)
Primary Solids	1,500	1,700
Secondary Solids	1,300	1,500
Blended Solids Production	2,800	3,200

7.3 Primary Sludge Fermentation Capacity Alternatives

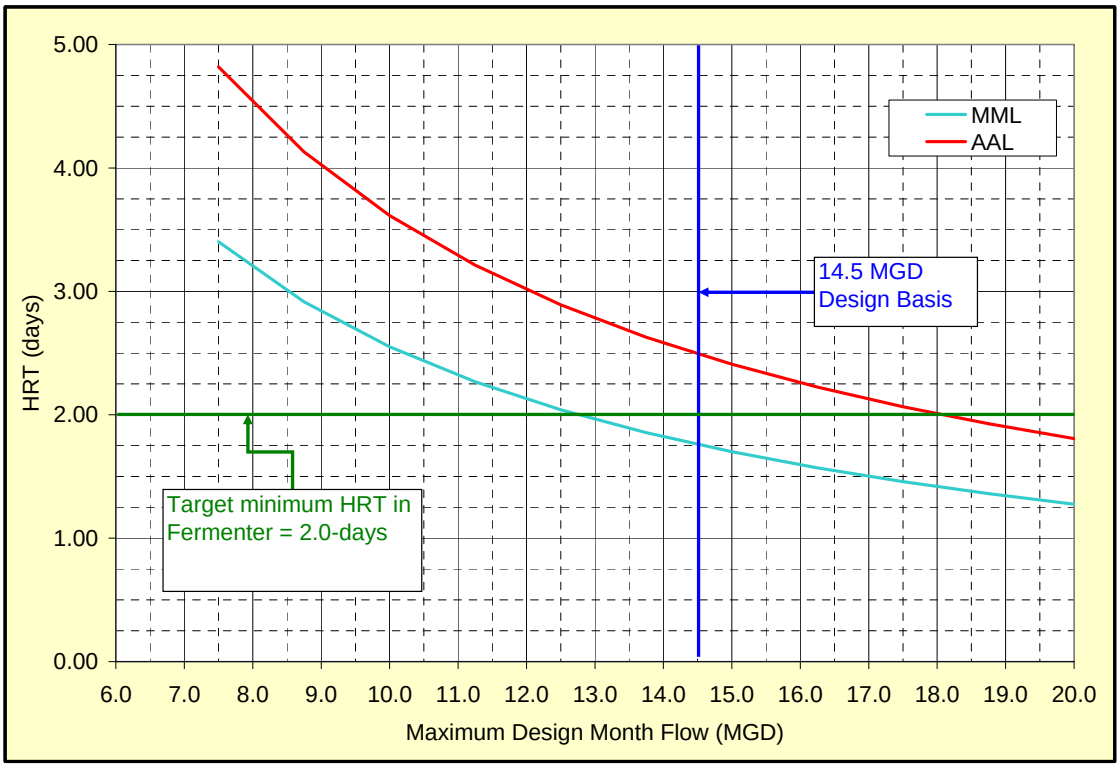
Primary sludge fermentation is utilized to generate soluble volatile fatty acids (SVFA) for enhanced biological nutrient removal in the activated sludge liquid treatment train. Fermented sludge is combined with waste activated sludge (WAS) and is sent to the gravity belt thickeners where the fermentate is separated from the solids and pumped to the nutrified sludge cells as a carbon source for RAS denitrification and biological phosphorus removal. The addition of these VFAs into the secondary process reduces the amount of supplemental carbon that must be purchased and applied. Typical fermentation design criteria provides for a two to three day retention time, and the Process Control Plan prepared for the plant recommends an optimum residence time in the fermenter of 2-days.

Primary sludge fermenter residence time is shown in Figure 7-1 for a range of maximum month design flow conditions for maximum month loading (MML) and annual average loading (AAL) conditions at primary sludge feed solids concentrations of 2.0% total solids. As shown in Figure 7-2, the residence time in the existing fermenter would be greater than 2-days under average annual loading conditions and would fall to

approximately 1.75-days under maximum month loading conditions at the 14.5-mgd design basis condition. The recommended 2-day residence time could be maintained at maximum month loading conditions with flow rates up to approximately 13 mgd.

Additional primary sludge fermentation tankage could be added at the Mason Farm WWTP to provide increased fermentation residence time. However, as shown in Figure 7-1 the existing fermenter could maintain residence time at greater than 2-days under average loading conditions at the design point and estimated residence time would fall to slightly below 2-days (1.75-days) at the maximum month loading condition. The resultant reduced residence time would likely result in lower SVFA production during these transient periods resulting in a need for intermittent additional supplemental carbon. Installing additional fermentation tankage during the near term represents a significant capital expenditure and on-going O&M expense for a facility which may not be needed until 2030 based on expected flow growth.

Figure 7-1
Primary Sludge Fermenter Residence Time



It is recommended that intermittent addition of supplemental carbon should be utilized to address transient reductions in primary sludge fermenter SVFA production based on the following:

- Estimated residence time in the existing fermenter is greater than 2-days at the 14.5-mgd design condition under average annual loading conditions.

- The existing fermenter can maintain residence times above 2-days at design loading rates up to approximately 13-mgd maximum month flow (10.4-mgd average annual flow) which is the expected flow in 2030.

- Supplemental carbon addition infrastructure is already in place and can be utilized to replace reduced SVFA production in the fermenter beyond 2030 when reduced residence time condition would be anticipated.

- Capital and O&M costs associated with additional fermenter facility construction and operation can be deferred for at least 20 years.

7.4 Digester Feed Sludge Thickening Alternatives

Gravity belt thickening process capacity is evaluated in Section 5 and shown to have sufficient firm capacity to meet anticipated solids loading rates for design maximum month flow conditions of in excess of the 14.5-mgd design flow condition. The gravity belt thickening unit process is anticipated to have sufficient capacity to meet loadings through the 20-year planning period horizon and capacity expansion is not needed to meet anticipated loading conditions associated with the14.5-mgd design condition.

7.5 Anaerobic Digestion Capacity Alternatives

Anaerobic digestion is utilized to achieve 40 CFR 503 requirements for vector attraction reduction and pathogen reduction. A thermophilic batch feed system is currently utilized to demonstrate conformance with the 40 CFR 503.32 “time and temperature” requirements to achieve “Class A” pathogen reduction and control. Anaerobic digestion capacity, when operated in the mode described in the Process Control Plan, was identified as being a capacity bottleneck for operation at the 14.5-mgd design flow loading conditions.

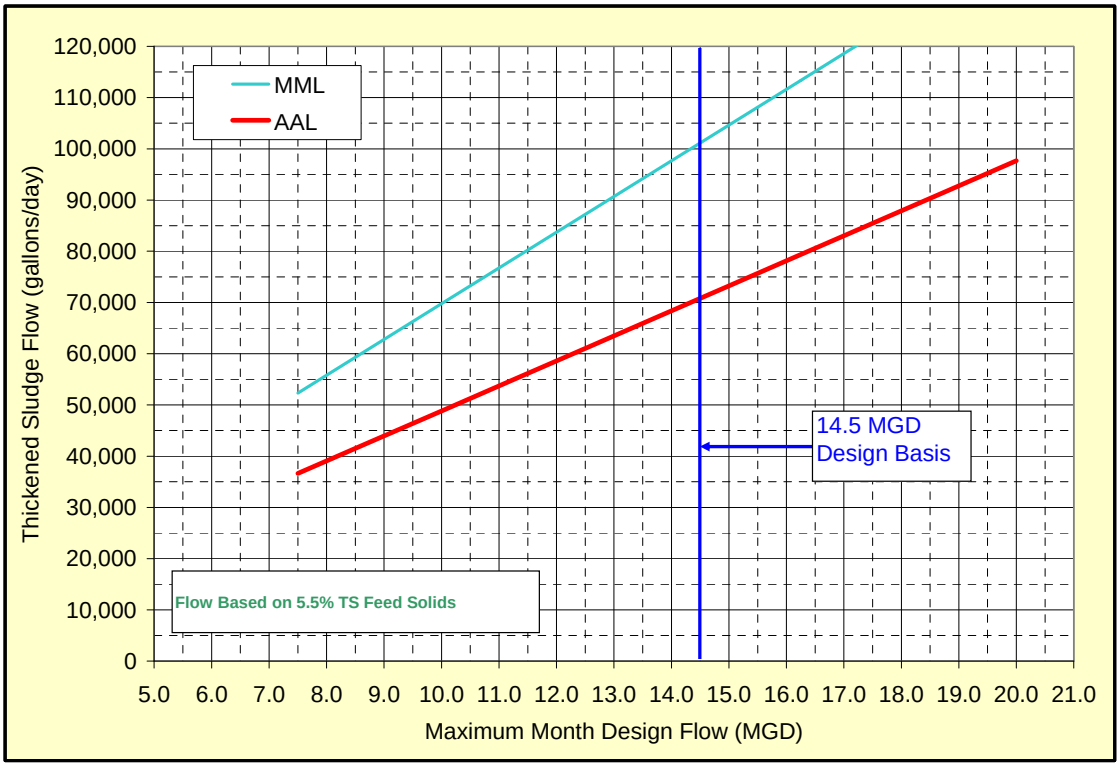
Two alternatives were evaluated for increasing the anaerobic digestion capacity to meet the 14.5-mgd design flow condition:

- Additional Thickening to Reduce Digester Feed Volumes
- Process Reconfiguration for Continued Class A Stabilization

7.5.1 Background Information

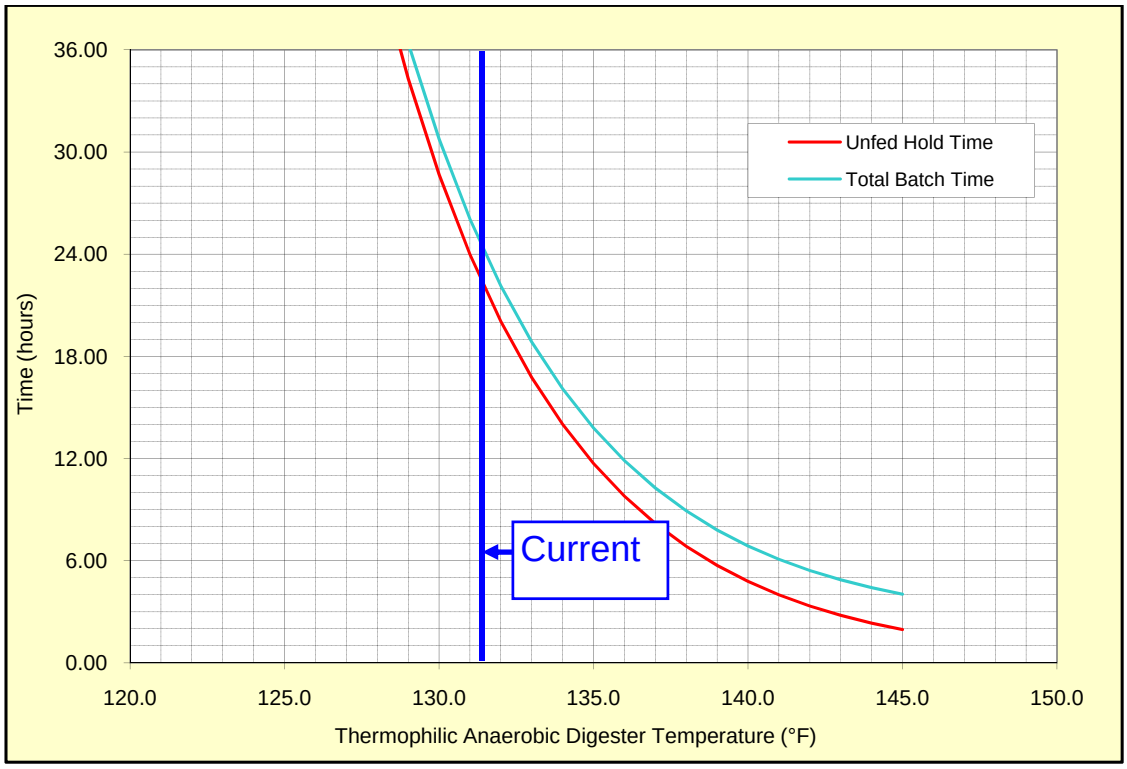
Fermented primary sludge (FPS) and WAS are currently co-thickened on gravity belt thickening equipment to recover soluble volatile fatty acids from the primary sludge and reduce the volume of the blended sludge prior to anaerobic digestion. Gravity belt thickened blended sludge is thickened to an average solids concentration of 5.5%. Anaerobic digester feed (ADF) volumes will vary depending on the plant influent flow rate, influent loading characteristics, and actual thickened sludge feed solids concentrations. Estimated anaerobic digester feed volumes, based on current gravity belt thickener performance, are shown in Figure 7-2 for the annual average loading and maximum month loading conditions for a range of design flow conditions. Volumetric flow rates for the 14.5-mgd design condition are estimated to range from approximately 70,800 gallons per day under annual average loading conditions to 101,200 gallons per day under maximum month loading conditions.

Figure 7-2
Anaerobic Digestion Feed Thickened Sludge Flow



The current process control plan (PCP) describes operation of the thermophilic anaerobic digestion process at a nominal operating temperature of 131.5°F in a cyclical batch feed operation with a maximum batch volume of 60,000 gallons. To achieve the 40 CFR 503 time and temperature requirements at this operating temperature the unfed batch time for Anaerobic Digester No. 3 must at least 22 hours and the total processing time (batch time plus fill and draw cycle time) must be in excess of 24 hours, as shown in Figure 7-3. The Class A thermophilic digestion capacity under the current PCP guidance is therefore 60,000 gallons per 24 hour period, or 60,000 gallons per day. As shown above in Figure 7-3 the expected thickened sludge feed flows under average annual and maximum month loadings at the 14.5-mgd design condition cannot be accommodated with the current tankage when operating at 131.5°F.

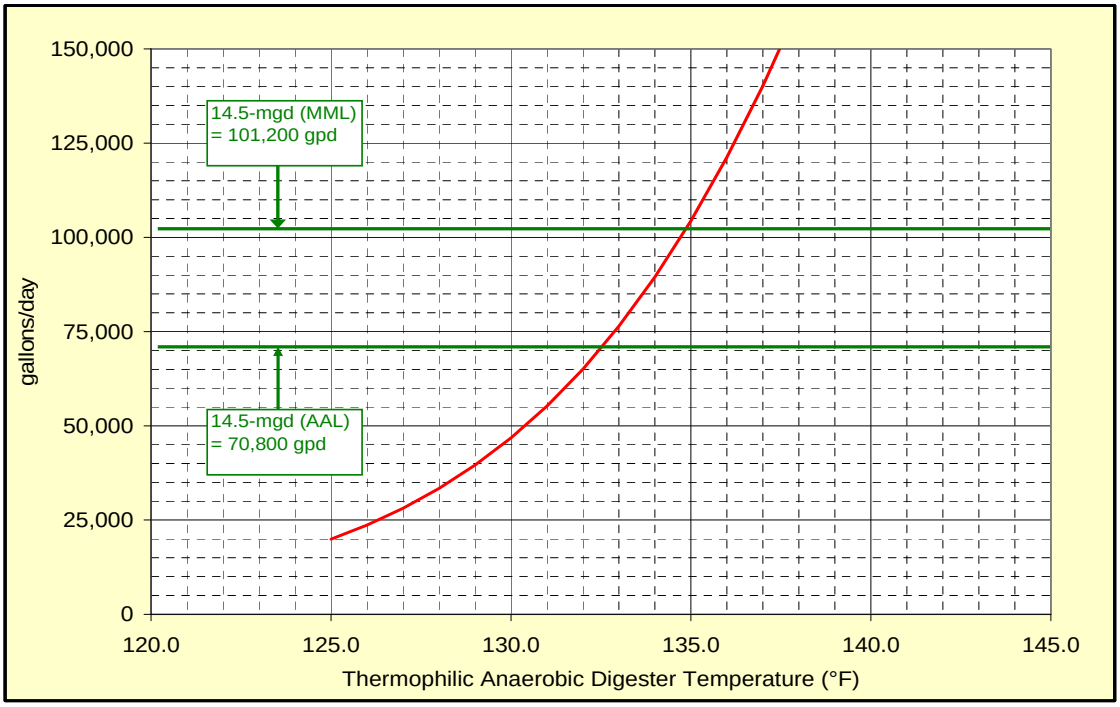
Figure 7-3
Anaerobic Digestion Batch Time for Class A Time & Temperature



As shown above in Figure 7-4, as anaerobic digester operating temperature increases the unfed batch and total batch time decreases. Therefore, for the same 60,000 gallon per batch volume total daily volumetric throughput can be increased while still meeting the 40 CFR 503 time and temperature requirements. Estimated maximum thermophilic anaerobic digestion unit process capacity (gallons per day) as a function of operating temperature is shown in Figure 7-5 based on a 60,000 gallon per batch volume and operation at the minimum unfed batch hold time. The thermophilic anaerobic digesters would need to be operated at a temperature of at least 135°F to achieve the 40 CFR 503 Class A time and temperature requirements based on the maximum month loading condition volumetric sludge production rates at the 14.5-mgd design condition.

Reconfiguration of the existing unit process and evaluation of alternatives for operating at an increased temperature will be evaluated in a later section of this technical memorandum.

Figure 7-4
Throughput Capacity vs. Anaerobic Digester Temperature



7.5.2 Additional Thickening to Reduce Feed Volume

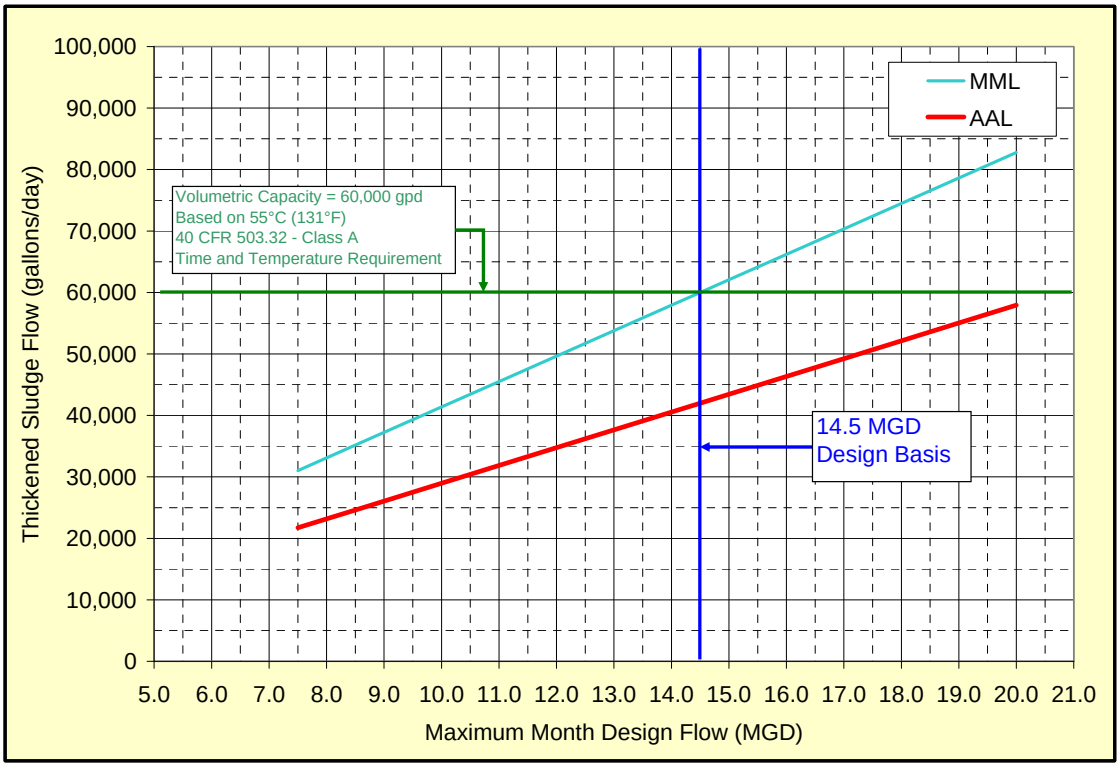
Additional thickening was evaluated as an alternative to reduce digester feed volumes to less than 60,000 gallons per day under maximum month loading conditions at the 14.5-mgd design flow. To achieve this functional objective anaerobic digester feed solids would need to be thickened to at least 9.3% total solids concentration as shown in Figure 7-5. Operation at this increased solids concentration is limited by the following constraints:

- Gravity belt thickening equipment is likely unable to achieve or maintain thickened blended sludge feed concentrations at these levels.

- The increased feed solids concentrations would result in a significant increase in anaerobic digester solids concentrations. Anaerobic digester operating concentrations, assuming 50% volatile solids reduction and 80% VS/TS feed fraction, would be approximately 5.6%. Operation at these high solids concentrations is not compatible with the existing anaerobic digester mixing, pumping or heat transfer equipment.

Based on these constraints, increased thickening to reduce feed volumes is not recommended as a viable alternative to removing the anaerobic digestion capacity constraint.

Figure 7-5
Anaerobic Digestion Feed Thickened Sludge – 9.3% TS

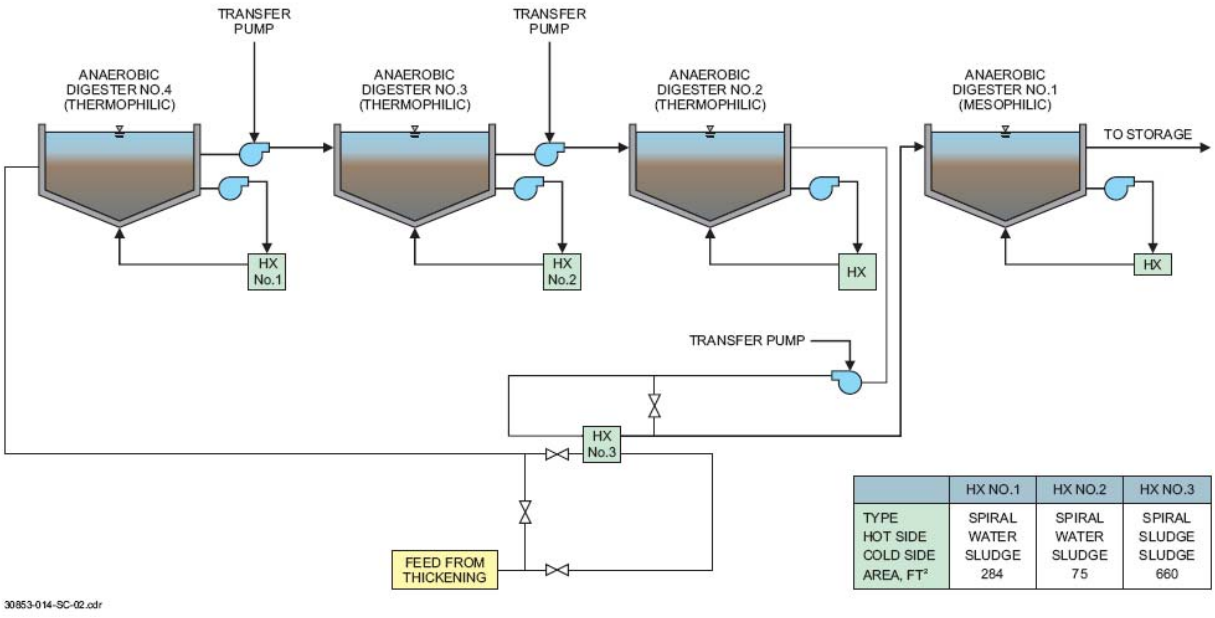


7.5.3 Process Reconfiguration for Class A Stabilization

The existing anaerobic digestion unit process configuration is shown in Figure 7-6. Currently the process is operated with HX-03 in "bypass" mode where the influent feed from the gravity belt thickeners is not pre-heated in the sludge-to-sludge heat exchanger (HX-03) and is routed directly to Anaerobic Digester No. 4 (DIG-04). Furthermore, the transfer pump (TP-2-1) between Anaerobic Digester No. 2 (DIG-02) and Anaerobic Digester No. 1 (DIG-01) is also configured where flow during the transfer pumping is not routed through HX-03. The sludge-to-sludge heat exchanger has been reported by operating staff to be problematic in operation due to extensive "ragging" which leads to clogging, high pressure drop, and extended downtime for maintenance and cleaning. Some of these problems may be attributed to the small openings, 0.500-inches (hot

side) and 0.625-inches (cold side) provided in the spiral heat exchanger. Furthermore, transfer pump (TP-2-1) sizing results in intermittent transfer pumping, while digester feed heat demands are continuous. Loss of the HX-03 heat transfer capacity limits heating capacity in the first stage thermophilic digester to the capacity that can be provided by HX-01 (approximately 1.9 MMBTUH @ 170°F inlet hot water temperature), which is significantly less that the total heating requirement.

Figure 7-6
Anaerobic Digestion Flow Schematic



	HX NO.1	HX NO.2	HX NO.3
TYPE	SPIRAL	SPIRAL	SPIRAL
HOT SIDE	WATER	WATER	SLUDGE
COLD SIDE	SLUDGE	SLUDGE	SLUDGE
AREA, FT²	284	75	660

Anaerobic digestion residence time was evaluated for DIG-04 and DIG-03 operating in thermophilic mode at the 14.5-mgd design basis. As shown in Figure 7-7, residence time falls below a recommended 10-day minimum hydraulic residence time (HRT) under maximum month loading conditions at approximately 11.8-mgd maximum month design flow, which is equivalent to an annual average flow of approximately 9.4-mgd. Thermophilic digestion residence time for DIG-04, DIG-03 and DIG-02 is shown in Figure 7-8. With all three digesters in service (DIG-04,-02, and -02) sufficient volume is in service to maintain digester residence time at approximately 11 days under maximum month loading conditions and 16 days at average annual loading conditions at the 14.5-mgd design flow basis.

Figure 7-7
Thermophilic Digester Hydraulic Residence Time
Digesters #4 and #3 (0.83 MGD Thermophilic Volume)

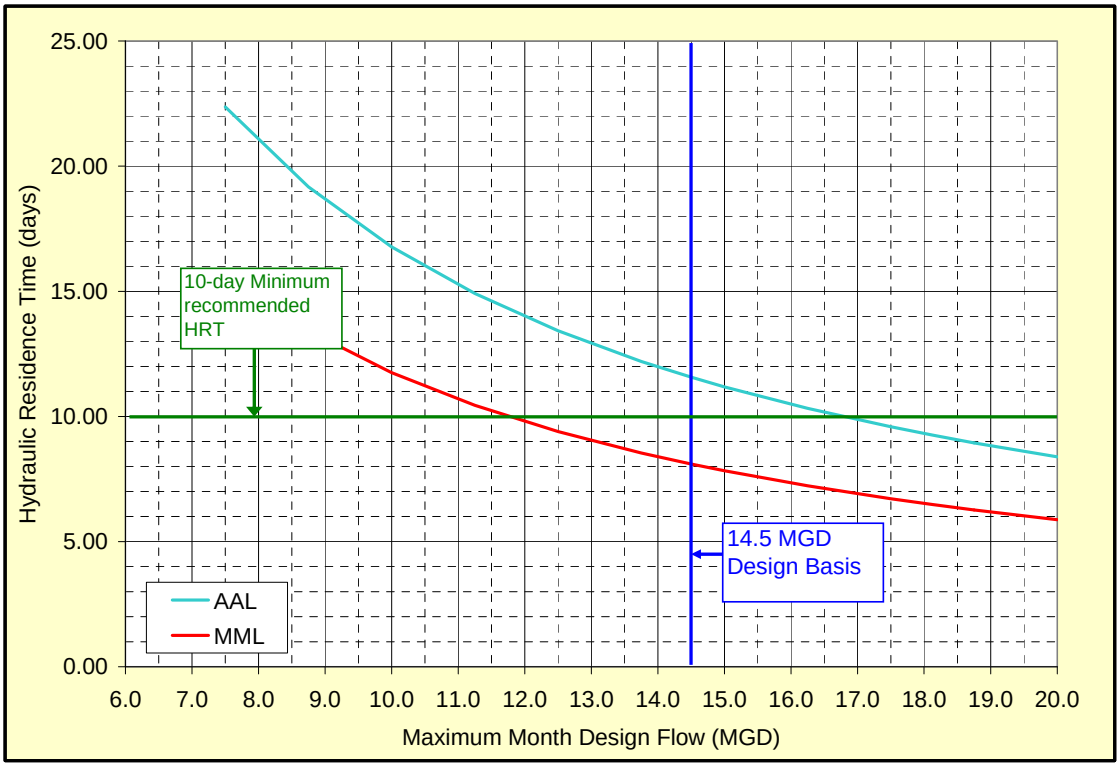
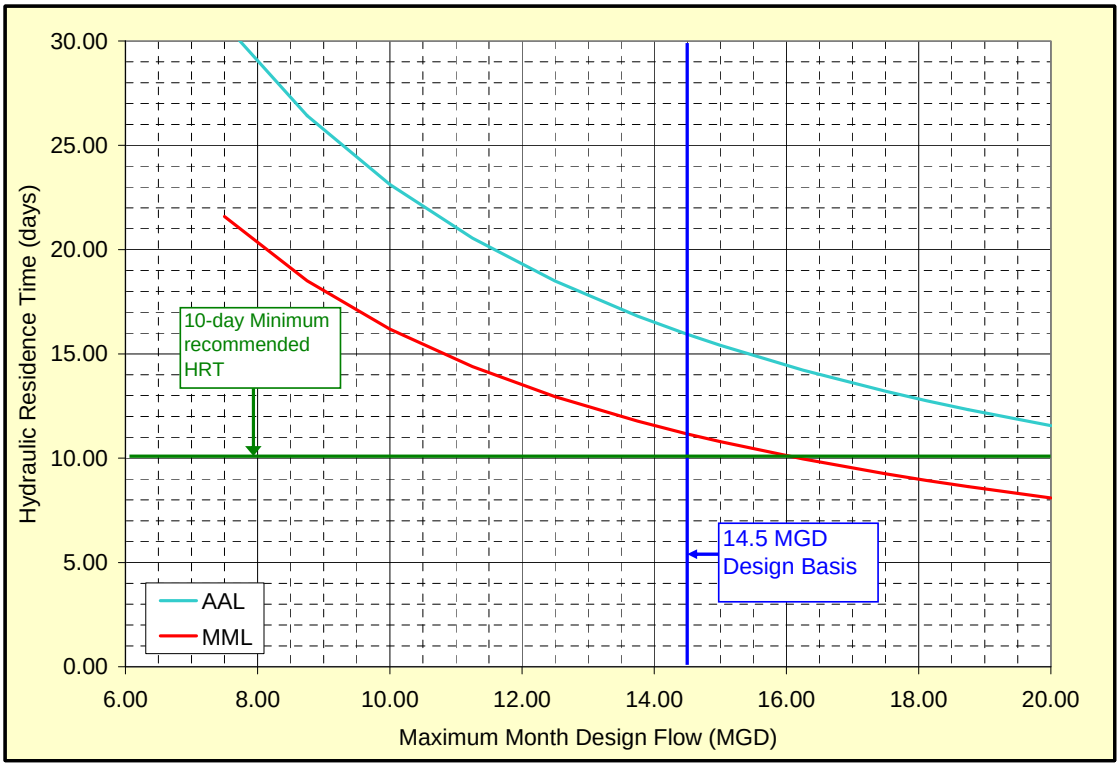


Figure 7-8
Thermophilic Digester Hydraulic Residence Time
Digesters #4 + #3 + #2 (1.13 MGD Thermophilic Volume)

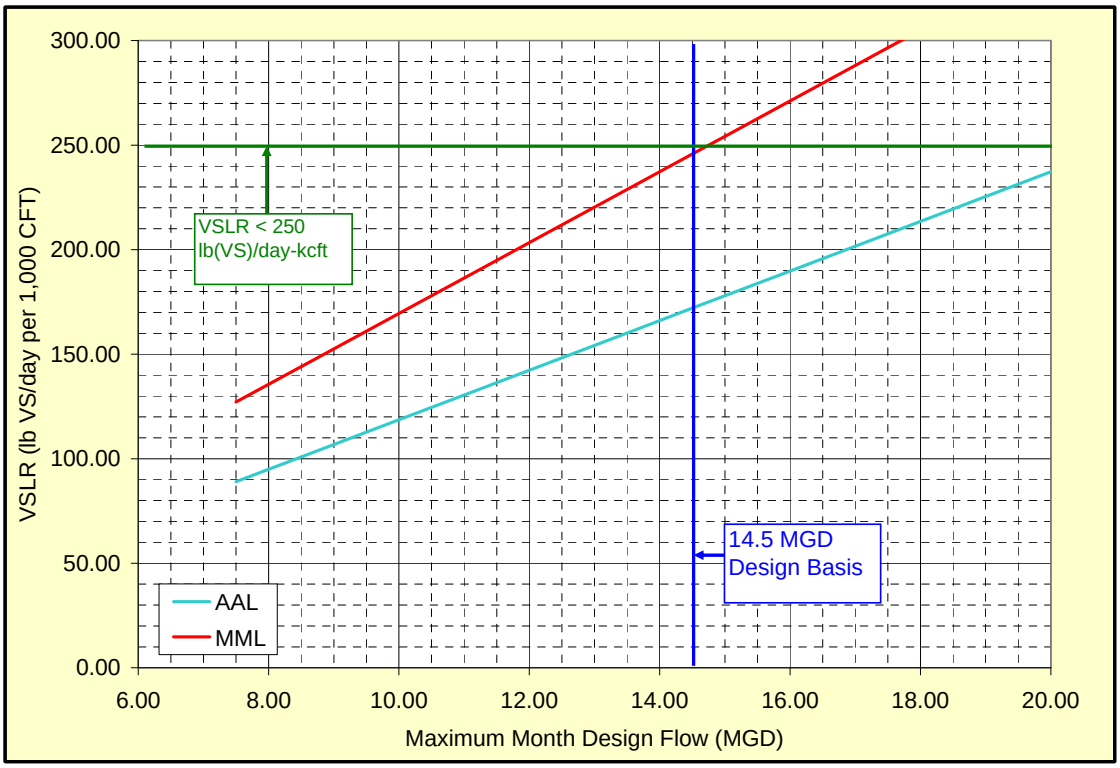


Anaerobic digester volatile solids loading rates (VSLR) where estimated for the thermophilic digestion unit process for annual average and maximum month loadings based on an estimated blended feed sludge VS/TS fraction of 80% as shown in Figure 7-9. Under maximum month loading conditions the estimated VLSR will be slightly below a recommended maximum of 250 pounds volatile solids per 1,000 cubic feet of digester volume. Under annual average loading conditions the estimated VSLR is estimated to average approximately 175 pounds volatile solids per 1,000 cubic feet of digester volume.

Under maximum month loading conditions at the 14.5-mgd design condition the thermophilic digestion unit process will be operating near the minimum recommended hydraulic residence time and maximum recommended VSLR. Therefore, limited potential exists within the existing volume to increase digester capacity beyond the 14.5-mgd design condition, unless OWASA is able to demonstrate that the Mason Farm

anaerobic digesters could reliably function at a higher temperature, lower hydraulic residence time, and higher volatile solids loading rate. Furthermore, at the 14.5-mgd design condition loss of any of the thermophilic digestion tankage could result in a process overload and/or upset condition, or a condition where the ability to meet Class A pathogen reduction requirements via the time and temperature condition could be compromised.

Figure 7-9
Anaerobic Digester – Volatile Solids Loading Rate



Thermophilic digestion heating demand for Digester No. 4 was estimated at maximum month loading and annual average loading conditions for a range of blended feed sludge influent temperatures as shown in Figure 7-10 when operating at a temperature of 135°F. Under maximum month loading conditions at minimum influent feed temperature conditions (50°F / 10°C) the combined liquid and maintenance heating demand for Digester No. 4 is approximately 3.30 MMBTUH based on continuous (24-hour/day) heat transfer.

The bulk of the sludge heating demand, approximately 3.30 MMBTUH, must be accommodated either prior to the feed sludge entering DIG-04 (e.g., sludge-to-sludge heat exchanger) or within DIG-04. The existing DIG-04 heat exchanger (HX-01) was originally selected based on a design heat transfer capacity of approximately 1.9 MMBTUH with a log-mean temperature difference (LMTD) of 26.2°F and a heat transfer conductivity of approximately 72,700 BTUH per °F. The estimated heating capacity when operating at 135°F sludge feed temperature was calculated based on various inlet hot water temperature and the specific heat transfer characteristics of the existing HX-01 unit. These heating capacities and heating characteristics are summarized in Table 7-2, below. For inlet hot water temperatures of 165°F and 170°F, the estimated heat transfer capacity of HX-01 is, respectively, 1.67 MMBTUH and 1.90 MMBTUH. These heat transfer capacities for HX-01 are also shown for reference on Figure 7-10.

As shown in Figure 7-10, the existing HX-01 spiral hot water-to-sludge heat exchanger is unable to carry the initial liquid heating load for the thermophilic digestion unit process at the 14.5-mgd design condition flow. Additional influent sludge heating capacity will be required to meet the total initial heating demand associated with Digester No. 4.

Figure 7-10
Anaerobic Digester No. 4 Heating Demands

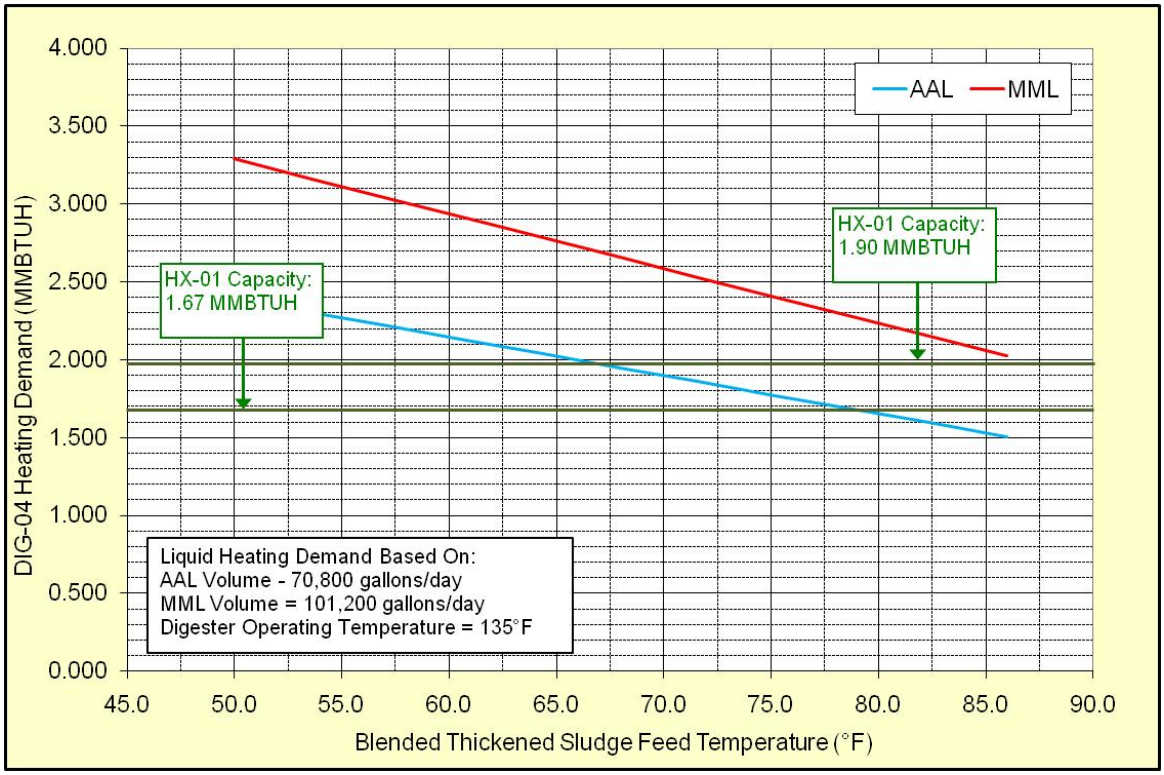


Table 7-2
Heat Exchanger (HX-01) Heat Transfer Capacity

Hot Water Inlet Temperature (°F)	Heat Transfer Capacity (MMBTUH)	Differential Hot Water Temperature (°F)	Hot Water Outlet Temperature (°F)
150	0.83	4.7	145.3
155	1.11	6.3	148.7
160	1.38	7.8	152.2
165	1.67	9.4	155.6
170	1.90	11.0	159.0

Heat transfer capacity was estimated for the HX-02 unit, which provides for maintenance heating demands in Anaerobic Digester No. 3 (DIG-03), based on the specific characteristics of the heat exchanger at the 135°F digester operating temperature. The estimated winter maintenance heating demand is approximately 0.25 MMBTUH for this tank. As shown in Table 7-3, the expected winter heat transfer requirement can be achieved with an inlet temperature to the heat exchanger of approximately 155°F. Under winter operating scenarios, if HX-01 is operated with an inlet hot water temperature greater than 165°F, the outlet hot water (T > 155.6°F) could be utilized to meet inlet hot water demands for HX-02. Alternatively, hot water directly from the boiler hot water loop could be utilized to meet the heating demand.

Table 7-3
Heat Exchanger (HX-02) Heat Transfer Capacity

Hot Water Inlet Temperature (°F)	Heat Transfer Capacity (MMBTUH)	Differential Hot Water Temperature (°F)	Hot Water Outlet Temperature (°F)
150	0.18	6.1	143.9
155	0.24	8.2	146.9
160	0.30	10.2	149.8
165	0.36	12.2	152.8
170	0.42	14.3	155.7

In summary:

- HX-01 can supply between 1.67 and 1.90 MMBTUH heating capacity to DIG-04 at the 135°F digester operating temperature with inlet hot water temperatures of 165°F and 170°F, respectively. This heating capacity is not sufficient to meet winter maximum month loading heating demands of approximately 3.30 MMBTUH required to increase thickened digester feed from ambient (50°F) to thermophilic (135°F) temperatures and meet maintenance heating losses in DIG-04.

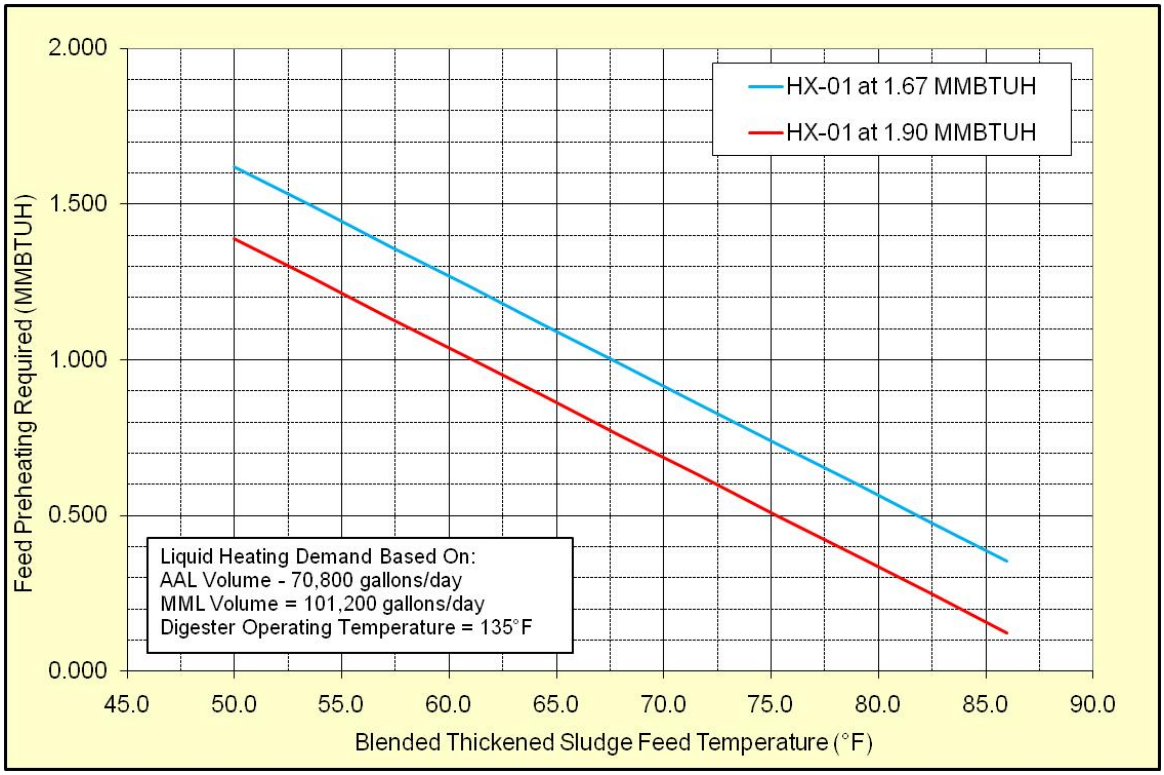
- HX-02 can supply at least 0.250 MMBTUH heating capacity to meet the DIG-03 maintenance heating requirements at the 135°F operating temperature with heat exchanger hot water inlet temperatures in excess of approximately 155.5°F. Additional heat transfer capacity would not be required to maintain DIG-03 operating temperatures under these conditions.

Estimated sludge pre-heating demands for the 14.5-mgd design condition are shown as a function of raw sludge feed temperature and HX-01 capacity in Figure 7-11. It is recommended that additional sludge pre-heating capacity of at least 1.60 MMBTUH be provided to meet the winter heat loading associated with thermophilic digestion operation (135°F) at the 14.5-mgd design condition maximum month loading rate. The additional recommended heating influent sludge heating capacity could be could be provided by:

- Operation of an influent sludge-to-sludge heat exchanger that could simultaneously and continuously cool thermophilic sludge to mesophilic operating temperatures (95°F to 100°F) between DIG-02 and DIG-01 while continuously heating influent gravity belt thickened sludge prior to DIG-04.

- Operation of an influent hot water-to-sludge heat exchanger that could utilize the heated water capacity from the existing hot water boiler system to meet influent heating demands. Under this scenario separate sludge cooling would be necessary between DIG-02 and DIG-01 to transition from thermophilic to mesophilic operating temperatures.

Figure 7-11
Sludge Pre-Heating Demands Upstream of Digester No. 4



7.5.3.1 Existing Sludge-to-Sludge Heat Exchanger

The existing sludge-to-sludge heat exchanger (HX-03) is a spiral heat exchanger with a total heat transfer surface area of 600 square feet and an estimated heat transfer capacity of approximately 66,900 BTUH per °F based on operating information obtained from the manufacturer's shop drawing submittals and O&M manuals. Selected operating characteristics for the existing sludge-to-sludge spiral heat exchanger were extracted from the O&M manuals are summarized in Table 7-4.

Table 7-4
Heat Exchanger (HX-03) Design Summary Information

	A	B	C
"Cold Side" Sludge Flow, gpm	120	120	120
"Cold Side" Inlet Temperature, °F	50.0	60.0	70.0
"Cold Side" Outlet Temperature, °F	83.6	89.4	95.2
"Hot Side" Sludge Flow, gpm	120	120	120
"Hot Side" Inlet Temperature, °F	125.6	125.6	125.6
"Hot Side" Outlet Temperature, °F	92.0	96.3	100.6
Heating Capacity Provided, MMBTUH	2.01	1.76	1.49
Minimum Heat Capacity Required, MMBTUH	1.57	1.23	0.87

Estimated operating conditions to meet the 14.5-mgd design condition would require modification of the operating conditions for this heat exchanger to the following:

- "Hot Side" Inlet Temperature, °F 135.0
- "Hot Side" Sludge Flow, gpm (101,200 gpd @ 24 hrs/day) 70.3
- "Cold Side" Sludge Flow, gpm (101,200 gpd @ 24 hrs/day) 70.3

Increasing the "hot side" temperature from the thermophilic anaerobic digester (DIG-02) will provide additional differential temperature for the heat exchanger and should serve to increase the heat transfer capacity. Reduced flow rates will decrease the Reynolds number and result in less turbulent flow in the heat exchanger and result in some reduced heat transfer capacity due to having an increasingly laminar flow in the heat exchanger. It is assumed for this analysis that these effects offset and that the overall heat transfer capacity for the existing sludge-to-sludge heat exchanger will remain similar to that shown in Table 7-4, above. It appears that the existing sludge-to-sludge heat exchanger has sufficient heat transfer capacity to meet the anticipated heat loadings at the design conditions. If this alternative is selected for additional consideration, it is recommended that additional scrutiny be given to confirming heat transfer capability for this unit.

As noted, operation of the existing spiral heat exchanger has been problematic. Several issues have been identified by operating staff during this investigation, as follows:

- The sludge transfer pump (TP-2-1) is a constant speed progressing cavity style pump (MOYNO) with a design capacity of approximately 120 gallons per minute (gpm) (170,000 gpd under continuous operation). Since this pump serves as both the heat exchanger "hot side" feed pump and the transfer pump between DIG-02 and DIG-01 pump operation is constrained by volumetric loadings to the anaerobic digestion unit process. This constraint causes an operational discontinuity between "heating operations" and "transfer operations". Heating operations are based on continuous operation of the gravity belt thickening unit process; however, transfer operations when total volume is less than the capacity of the transfer pump are not continuous. For example, with a volumetric loading of 101,200 gallons per day (14.5-mgd design condition, maximum month loading) the heating demand would be continuous; however, the transfer pump would only need to operate approximately 14 hours during the 24 hour period.

- The existing heat exchanger is prone to plugging and clogging due to the accumulation of solids in the interstitial areas of the heat exchanger. Plugging is aggravated by the channel spacing in the unit (0.500-inch hot side and 0.625-inch cold side). The spiral hot water-to-sludge heat exchangers, for comparison, have a sludge side channel spacing of 1.00 inches. Plugging results in reduced heat transfer capacity and increased operating pressure on the thickened sludge feed pumps from the gravity belt thickeners and digester transfer pumps Manufacturer's information contained in the O&M manual indicates that maceration (comminutor) should be provided on both hot and cold streams such that the maximum particle size delivered to the unit is less than 0.25-inches. OWASA operating personnel have installed maceration on the cold side feed from the gravity belt thickeners. Operations personnel have reported reduced plugging and fouling from hair and other material following these improvements on the "cold side" GBT feed.

Modifications to accommodate the existing sludge-to-sludge heat exchanger could include the following major items:

- Install a variable speed drive for the sludge transfer pump that transfers sludge from Digester 2 to Digester 1 (TP-2-1) to allow flow pacing and continuous operation of this pumping unit to match the continuous heating demand duty for the sludge pre-heating system.

- Install robust maceration equipment on the hot side feeds to the existing sludge-to-sludge spiral heat exchanger to assure maximum recommended particle sizes of less than 0.25-inches are achieved. Since maceration has already been provided on the cold side feed at the gravity belt thickeners recommended improvements would include installation of hot side maceration equipment.

The estimated capital cost for these improvements is approximately $400,000.

7.5.3.2 New Sludge-to-Sludge Heat Exchanger

As an alternative to continuing service with the existing spiral sludge-to-sludge heat exchanger a replacement sludge-to-sludge heat exchanger based on “tube-in-tube” design could result in a design which would be less prone to the plugging and fouling issues associated with the existing spiral heat exchanger.

The recommended heat exchanger would require a minimum heating capacity of approximately 1.60 MMBTUH to meet the heating demands associated with the winter, maximum month loading at the 14.5-mgd design flow condition. The recommended system would also include the macerator and transfer pump controls items previously identified for the spiral heat exchanger system.

The estimated capital costs for these improvements is approximately $500,000 with the differential cost for the demolition of the existing spiral heat exchanger and the purchase and installation of a new tube-in-tube style sludge-to-sludge heat exchanger.

7.5.3.3 New Hot Water-to-Sludge Heat Exchanger

As an alternative to continuing with a sludge-to-sludge heat exchanger concept described in the previous paragraphs a new hot-water to sludge heat exchanger could be provided to meet feed sludge pre-heating demands. This heat exchanger could be a spiral sludge heat exchanger or a tube-in-tube type heat exchanger unit. The recommended unit provided under this alternative would have a minimum heating capacity of approximately 1.60 MMBTUH to meet the winter operating condition pre-heating requirements.

Under this operating configuration heat recovery for pre-heating between DIG-02 and DIG-01 would be discontinued and additional cooling load would be placed on the DIG-01 cooling system to reduce sludge temperatures from thermophilic to mesophilic conditions upstream of the sludge storage tank or rotary press dewatering. The estimated cooling demand on the DIG-01 system under this configuration would be approximately 1.40 MMBTUH to reduce sludge temperature from 135°F to 95°F at the 101,200 gallon per day maximum throughput capacity. Estimated cooling capacity for the DIG-01 cooling equipment is approximately 0.3 to 0.4 MMBTUH (Brown and Caldwell, 09/24/06, Heat Exchanger Memorandum) and would not be sufficient to meet DIG-01 mesophilic operating temperature goals (95°F to 100°F). It is recommended that additional cooling capacity be installed in DIG-01 to meet the increased cooling demand of this alternative.

Total hot water heating demand under this configuration at the 14.5-mgd winter operating condition is estimated to be approximately 3.8 MMBTUH. The existing low pressure steam boilers have a total capacity of 4.0 MMBTUH output steam energy and should be sufficient to meet the total winter heating demand with all units in service at 95% maximum firing rate. However, loss of either boiler, or reduced boiler capacity due to siloxane or other fouling on the heat transfer surfaces, could result in insufficient hot water generation capacity to meet demands. It is recommended that additional hot water generation capacity be considered to provide a firm capacity of at least 4.0 MMBTUH. This additional capacity could be developed via installation of an additional 2.0 MMBTUH (output) dual fuel capable hot water boiler. Adding a VFD to the digester transfer pump between DIG-02 and DIG-01 (TP-2-1) would not be required since the transfer process would be decoupled from the heating process.

In summary, the major elements of this approach include:

- Hot Water-to-Sludge Preheating System (1.6 MMBTUH minimum heating capacity)
- DIG-01 Cooling System Improvements (1.4 MMBTUH minimum cooling capacity)
- Redundant Dual Fuel (DG/NG) Capable Low Pressure Steam or Hot Water Boiler (2.0 MMBTUH minimum output capacity). Building space would need to be developed to house this equipment and integrate it with the existing hot water system.

The estimated capital cost for these improvements is approximately $1,300,000.

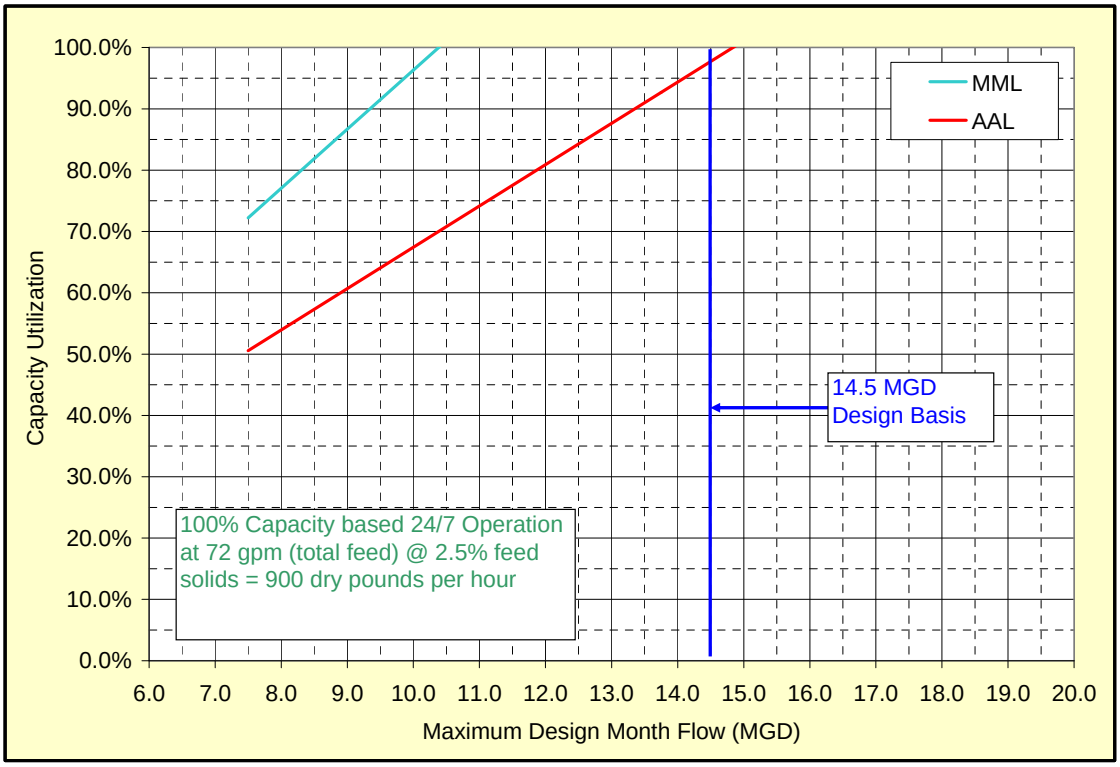
7.6 Post-Digestion Sludge Storage

Post-digestion liquid biosolids storage capacity is evaluated in Section 5 and shown to have sufficient firm capacity to meet anticipated solids loading rates for design maximum month flow conditions beyond the 20-year planning period horizon. Capacity expansion is not needed in this unit process to meet anticipated loading conditions associated with a 14.5-mgd design condition.

7.7 Dewatering Capacity Alternatives

Digested sludge may be managed either as a stabilized liquid product or as dewatered cake. Rotary press equipment installed in the dewatering building provides flexibility in the liquid biosolids land application program by allowing a portion of the biosolids to be dewatered and sent off-site for management. Therefore, dewatering capacity does not limit the capacity of the overall solids treatment train since it is used to provide process flexibility. Figure 7-12 shows the estimated rotary press utilization at selected design loading conditions based on handling 100% of the post-digestion residuals at the feed concentrations and flows shown.

Figure 7-12
Rotary Press Dewatering Capacity Utilization



Performance test reports prepared by Fournier (September 2007) were reviewed and the performance demonstration was conducted at an average flow rate of approximately 45 gpm at a feed solids concentration of approximately 1.8%, or a solids loading rate of approximately 400 dry pounds per hour. Inquiry to plant operating personnel and on-site observations (01/05/2009) appear to indicate that the typical practice has been, and currently is, to operate the dewatering equipment at flows typically in the 40-45 gpm range with solids concentrations between 2.0% and 2.5%. Current practice is to feed the rotary press at a rate significantly below the manufacturer's rated capacity of 72 gpm.

While capital improvements are not recommended for the rotary press dewatering facility at this time to increase capacity, it is recommended that OWASA initiate "stress testing" of the dewatering equipment at increased hydraulic and solids loading rates to establish performance at the higher loading rates and demonstrate unit process capacity. This will allow plant staff to better understand the extent to which the rotary press can be used to provide dewatering as the plant approaches the design conditions.

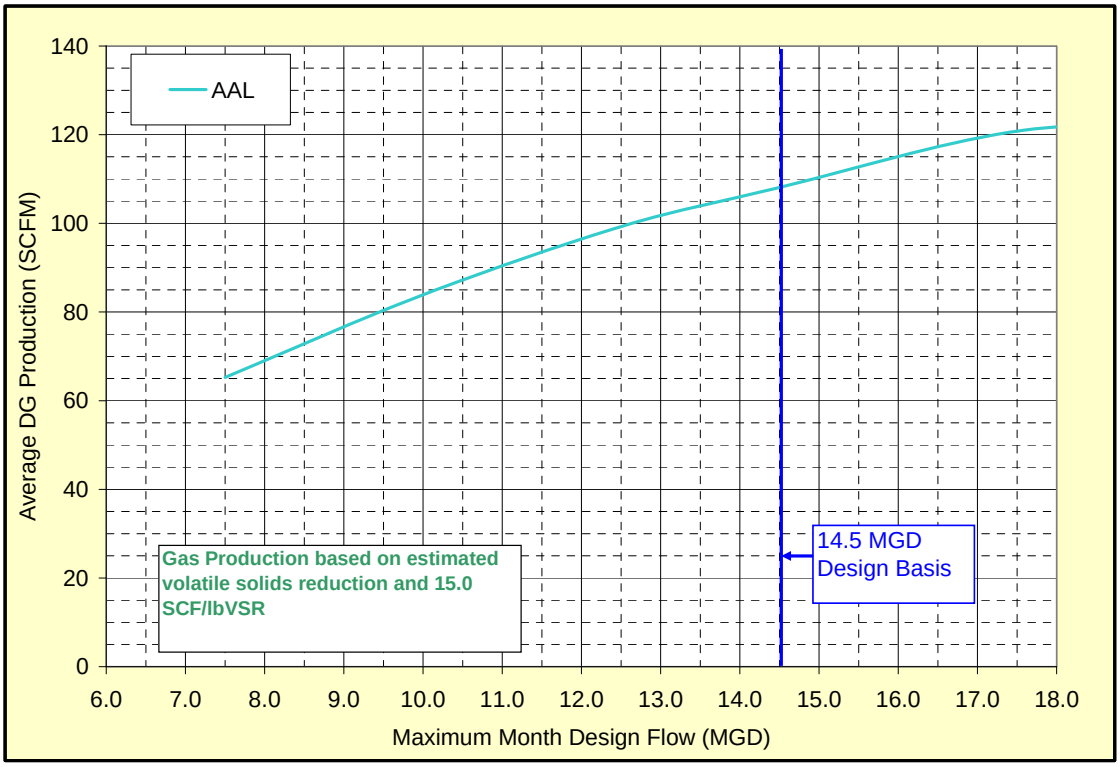
7.8 Digester Gas Utilization

OWASA has the capability to utilize digester gas in the existing engine-driven blower unit. This unit consists of a Waukesha Model L3712GU engine coupled to a 7,000 SCFM multi-stage Lamson centrifugal blower. Currently the engine-driven blower is down for maintenance and is not operable. The existing system does not appear to be configured for full combined heat and power utilizing exhaust gas heat recovery.

Anaerobic digester gas production rates were estimated for average annual loading conditions for the purpose of sizing digester gas utilization equipment for a range of operating flow conditions as shown in Figure 7-13. Average gas production rates at the 14.5-mgd design condition are estimated at approximately 105 SCFM (151,000 SCF/DAY) based on a digester gas production rate of 15 SCF per pound of volatile solids reduction in the anaerobic digestion facility. Anaerobic digester gas total energy availability at the 14.5-mgd design condition is estimated at approximately 90.6 million BTU per day (3.775 MMBTU per hour) based on a digester gas energy density of 600 BTU per cubic foot (BTU/SCF).

Preliminary selections were evaluated for new digester gas utilization equipment that would replace the engine-blower with an engine-generator system to produce electrical energy as an output. Additionally, consideration should be given to configuring the system to incorporate exhaust gas heat recovery and engine jacket heat recovery for a fully outfitted combined heat and power (CHP) configuration. The proposed system could be utilized to generate a hot water stream which could be used for digester feed sludge stream pre-heating in a sludge-to-hot water heat exchanger as a replacement for the existing sludge-to-sludge heat exchanger which is subject to fouling and, as currently configured, can only operate intermittently during transfer pumping operations. Preliminary evaluation are based on a GE/Jenbacher Model 208 engine-generator system with an input energy consumption rate of 3.15 MMBTUH, output electrical power capacity of 330 kW, and output recoverable thermal energy capacity of 1.25 MMBTU.

Figure 7-13
Average Annual Digester Gas Production Rate



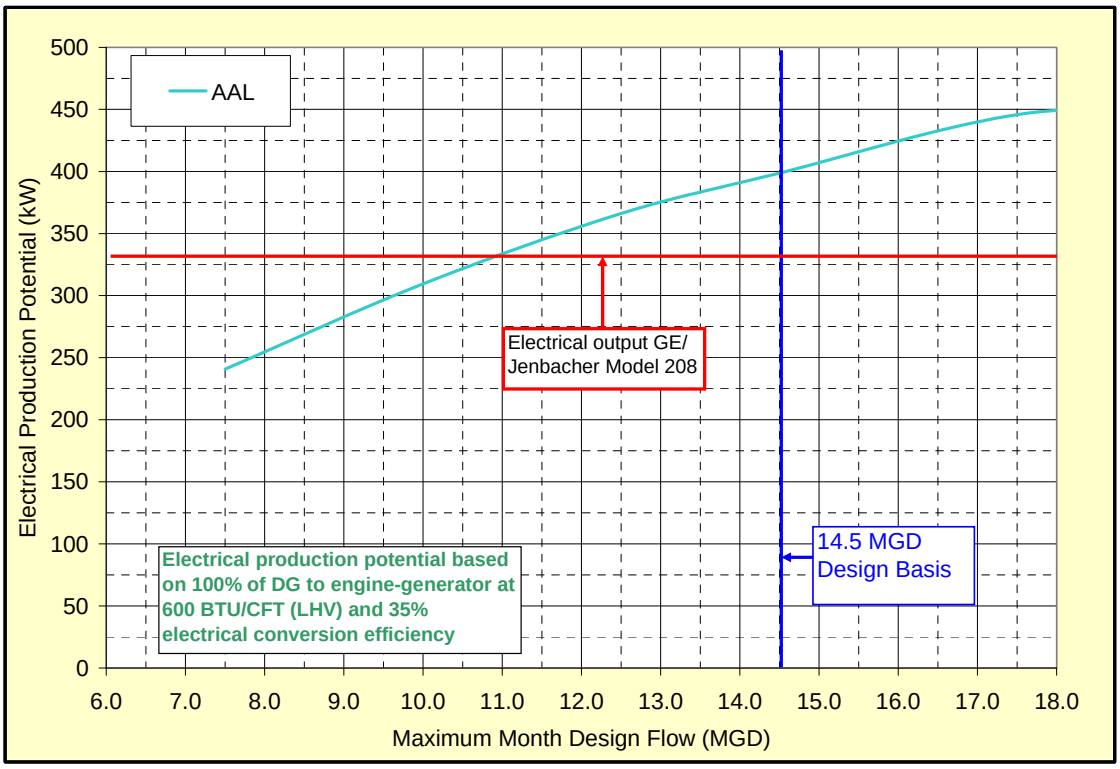
The Jenbacher Model 208 unit, based on a digester gas energy density of 600 BTU/SCF, would require an inlet gas feed rate of approximately 90 SCFM to operate at 100% loading rate and could operate at loading rates down to approximately 75%, or approximately 70 SCFM, based on the manufacturer's recommended operating range. Influent operating flow equivalency for this operating range is summarized in Table 7-5.

Table 7-5
Operating Flow Equivalency – Engine Generator System

	100% Loaded	75% Loaded
Maximum Month Flow (MGD)	10.8	8.0
Annual Average Flow (MGD)	8.6	6.4

Electrical power production potential for the proposed system is shown in Figure 7-14. When plant flow rates exceed the 10.8-mgd design basis flow digester gas production is anticipated to exceed the needs of the proposed digester gas utilization equipment and surplus digester gas could be utilized in the low pressure steam boiler for additional heating in the anaerobic digester unit process. Electrical power production would be capped at approximately 330 kW under these operating conditions. The proposed system should be able to operate at greater than 75% loading (minimum loading) under current conditions and operate at fully loaded conditions within 10 years.

Figure 7-14
Electrical Power Production Potential



Net present benefits from electrical power production were estimated based on the following assumptions:

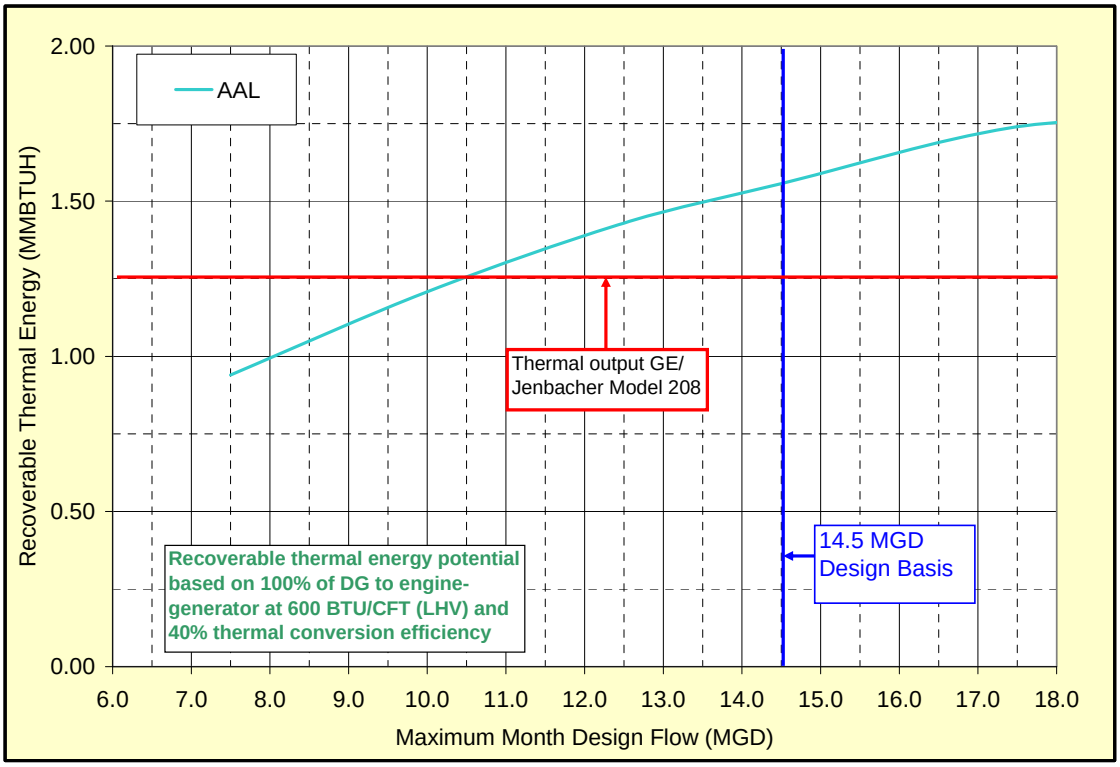
- Power Benefit (with Renewable Energy Premium), $/MWH $75.00
- Maintenance Allowance (Engine + Gas Treatment), $/MWH $20.00
- Net Power Benefit (Power Benefit – Maintenance), $/MWH $55.00

- Inflation Rate Net Power Benefit, % per year 3.50%
- Cost of Capital (Interest Rate), % per year 5.50%
- Annual Downtime Allowance, days 21
- Net Annual Operating Time, hours per year 8,256
- Engine Gas Utilization at 100% Loading, SCFM 87.5
- Engine Output Power Production at 100% Loading, kW 330

The net present worth benefit from electrical power production was estimated based on the annual average gas production curve shown for the 20-year period from 2010 to 2030. Based on the assumptions above, the estimated 20-year net present worth of the benefit associated with the electrical power production is approximately $2.45 million dollars.

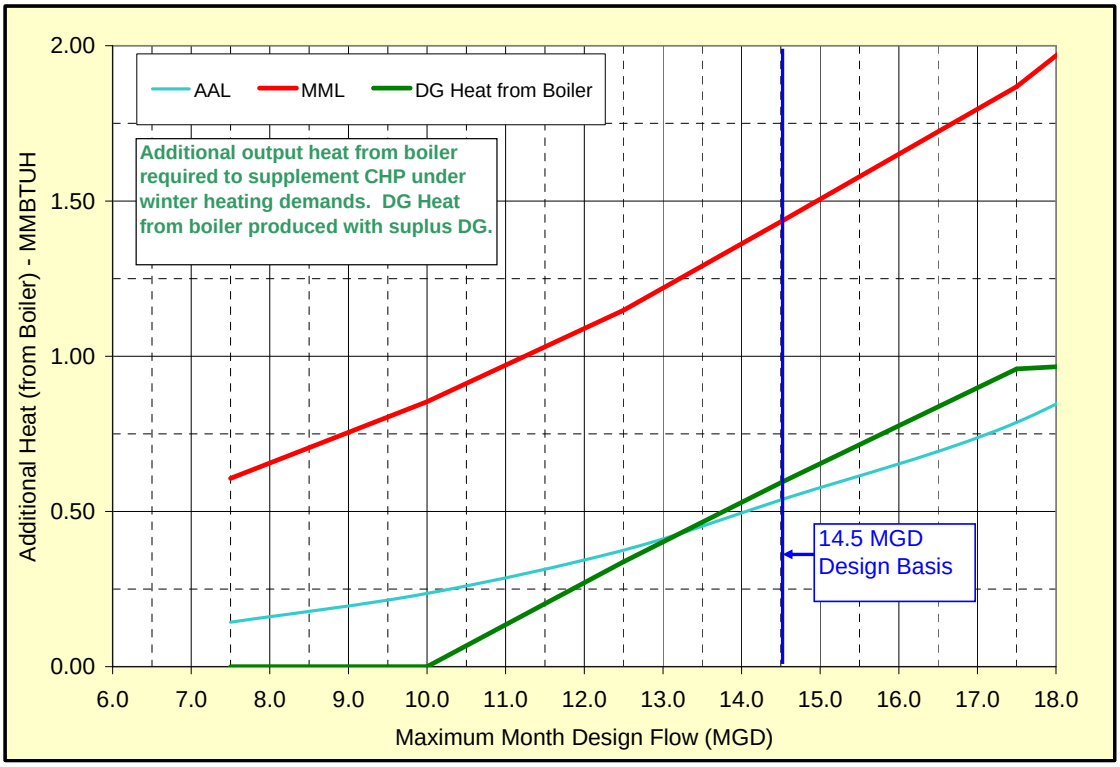
Thermal energy production potential from the proposed CHP system is shown in Figure 7-15. Similar to electrical power production CHP heat energy is limited when digester gas production rates are in excess of the engine input energy needs. When digester gas availability exceeds digester gas needs in the CHP system excess digester gas would be available to utilize in the low pressure steam boilers to generate supplemental heat to meet demands. Supplemental heating will be required to meet winter heating demands shown in Figure 7-15 with the CHP system.

Figure 7-15
Thermal Energy Production Potential



As shown in Figure 7-16, initially digester annual average digester gas production rates will be fully utilized in the CHP system and supplemental heating energy would need to come in the form of purchased fuel (e.g., natural gas) even to meet average loading conditions. However, as digester gas production increases above what can be consumed by the CHP system the excess digester gas can be routed to the low pressure steam boilers to meet supplemental heating requirements. Under conditions below approximately 13.5 mgd design flow purchased supplemental heating fuel would be required to meet average loading conditions. The requirement for purchased fuel across nearly the entire facility life will result in an additional operating expense which will reduce the net present benefit associated with the electrical energy production from the CHP system. This net reduction is difficult to quantify since multiple variables (e.g., seasonality, fluctuating natural gas costs, etc.) influence the net annual purchased fuel requirement.

Figure 7-16
Winter Supplement Heating Demand Requirements



Capital costs were estimated for the internal combustion engine generator alternative based on a GE Jenbacher Model 208 unit capable of producing approximately 330 kW output power at 480VAC, 3-phase in parallel with utility generated electrical power. The engine generation unit would include the following items:

- Low BTU Fuel System
- Remote Fuel Shut-off Valve
- Motor Winding and Bearing Temperature Indication
- Water and Exhaust Flexible Connections
- Low Water Level Gauge and Safety Switch
- Lubrication Oil System (Regulator, Filtration, etc)
- Utility Paralleling Speed Governor
- Exhaust Temperature Indication
- Exhaust Silencer
- Starting and Control Battery Package

- 24V Battery Charger
- Vibration Isolation Dampeners (Spring Type)
- Paralleling Switchgear and Utility Connection Equipment
- Engine Generator Modular Enclosure

Additional major mechanical equipment included in the estimated capital costs are:

- Digester Gas Compression and Drying (Moisture Removal)
- Non-Regenerative Iron Sponge (Hydrogen Sulfide Removal)
- Activated Carbon Filter (Siloxane Removal)

Estimated capital costs for the proposed system are approximately $2.10 million dollars. Estimated 20-year lifecycle net benefit from electrical production, excluding net present costs for purchased supplemental fuel, were approximately $2.45 million dollars. The estimated "pay-back" period, excluding purchased supplemental fuel is approximately 16 years. If the net present cost of supplemental fuel is greater than approximately $350,000, the estimated 20-year net present benefit would be less than the estimated capital cost resulting in a benefit-to-cost ratio of less than 1.0.

Based on the low estimated benefit-to-cost ratio and risks associated with purchased supplemental fuel, installation of a new digester gas utilization system does not appear to have an economic benefit. Therefore, this project component is not recommended to be included as a viable project alternative at this time.

7.9 Solids Handling Recommendations

The primary sludge fermenters, gravity belt thickeners or liquid biosolids storage are not identified as needing improvements at this time. Fermenter detention times will be below typical design criteria at maximum month design loads; however, additional supplemental carbon feed can offset the decrease in VFA production to meet process requirements.

Improvements to the anaerobic digestion process will be required to ensure adequate stabilization as flows and loads increase. It is recommended that a macerator be installed on the hot side feeds to the existing sludge-to-sludge heat exchanger to reduce plugging and clogging associated with particles larger than 0.25 inches. A VFD should also be installed for the sludge transfer pump (TP-2-1) to allow flow pacing and continuous operation to match the continuous heating demand of the sludge pre-heating system. The estimated capital cost of these improvements is $400,000.

If the existing sludge-to-sludge heat exchanger still experiences fouling and operational issues after these improvements are made replacement with a tube-in-tube style sludge-to-sludge heat exchanger or sludge-to-hot water heat exchanger should be considered. The installation of the grinder and VFD will still provide an operational benefit if a new heat exchanger is installed. Locating a new heat exchanger inside the existing room may be challenging due to potential conflicts with existing structures and equipment.

It is recommended that the existing dewatering rotary press be “stress-tested” to determine maximum hydraulic and solids loading rates and demonstrate unit process capacity since it has always been operated well below the manufacturer’s capacity rating. It is anticipated that additional throughput can be achieved with the existing rotary press.

Providing for combined heat and power generation through recovery of digester gas does not appear to provide a clear economic benefit at present. Increased flows and loads, energy cost inflation, potential for renewable energy credits and carbon taxes may result in CHP being more attractive in the future, and OWASA should continue to monitor its potential future application.

Section 8
Recycle Streams Alternatives Evaluation

8.1 Recycle Stream Treatment Evaluation

Soluble total Kjedahl nitrogen (TKN), predominantly in the form of ammonia, and soluble ortho-phosphate (PO4-P) are released during the destruction of biomass volatile suspended solids (VSS) in the anaerobic digestion process. These nutrients are returned to the plant influent (downstream of the current composite sampler location) with the rotary press dewatering filtrate, increasing the nutrient load to the primary and secondary processes. Anaerobically digested filtrates/centrates typically contribute between 10% to 20% of the nitrogen load to the liquid stream process and can account for as much as 40%-50% of the phosphorus load to the process. Increased production of dewatered cake instead of liquid biosolids application will increase the impacts of the solids recycle streams on the nutrient removal process.

This evaluation considers the impacts of the rotary press dewatering filtrate stream on the performance of the liquid train at varying levels of dewatering: 0%, 25%, 50%, 75% and 100% of the total digested solids flow. Filtrate TKN concentrations based on limited data collected since August 2009 indicate an average concentration of 1,250 mg/L. This value will be used in the recycle stream evaluation. About 60% of the digested sludge is currently dewatered; the rest is land applied as liquid biosolids.

Recent filtrate PO4-P and TP concentration data vary widely (between 10 mg/L and 150 mg/L). These concentrations are abnormally low for an anaerobic digestion process and are influenced by the addition of alum to the filtrate. Filtrate PO4-P concentrations from previous sampling efforts when alum was not added to the filtrate indicate an average concentration of approximately 250 mg/L. This value is similar to the predicted filtrate TP concentration of 300 mg/L (before alum addition) predicted by the BioWin model under current conditions when alum addition is modeled. These low filtrate phosphorus concentrations suggest that biological phosphorus removal (BPR) is limited at the Mason Farm WWTP, with the majority of phosphate reduction accomplished through chemical phosphorus removal.

Optimization of the Mason Farm WWTP for biological phosphorus removal will increase the soluble phosphate concentration of the recycle stream since luxury phosphorus uptake associated with BPR will result in a much greater biomass phosphorus content (from 1-2% of volatile solids to 8% or more). This will substantially increase the soluble phosphorus released in the anaerobic digesters, which is returned to the liquid stream via the filtrate stream. The process model predicts that the filtrate phosphate concentration could approach 1,400 mg/L with efficient BPR and no alum addition to the filtrate or secondary treatment. The recycle stream evaluation will consider two filtrate phosphorus conditions. The first condition assumes limited BPR and a filtrate TP concentration of 250 mg/L. The second condition assumes effective BPR could eliminate regular chemical feed for phosphorus removal, resulting in a filtrate total phosphorus concentration of 1,400 mg/L.

Table 8-1 summarizes the filtrate TKN and TP loads for the various flow and phosphorus removal conditions under the 11.6 mgd annual average flow and load conditions. Filtrate loads are based on a TKN concentration of 1,250 mg/L and TP concentrations of 250 mg/L and 1,400 mg/L under chemical phosphorus removal (CPR) and BPR conditions, respectively. Table 8-2 summarizes the impacts of the rotary press recycle streams as a percentage of the total primary influent load based on the annual average conditions under the current design.

Table 8-1
Rotary Press Filtrate Nutrient Loads

Sludge to Dewatering	Filtrate Flow (gpd)	TKN Load (lb/day)	CPR TP Load (lb/day)	BPR TP Load (lb/day)
0%	0	0	0	0
25%	15,000	156	31	175
50%	30,000	313	63	350
75%	45,000	469	94	525
100%	60,000	626	125	700

Table 8-2
Rotary Press Filtrate Impacts as Percentage of Primary Influent Nutrient Loads

Sludge to Dewatering	TKN	CPR TP	BPR TP
0%	0%	0%	0%
25%	3.9%	4.4%	20.3%
50%	7.5%	8.3%	33.8%
75%	10.8%	12.0%	43.3%
100%	13.9%	15.4%	50.5%

The BioWin process model developed for the Mason Farm WWTP was used to evaluate the sensitivity of the current configuration and recommended step feed configuration to changes in dewatering operations. In particular the impacts on effluent TN under annual average conditions and effluent ammonia under maximum month conditions were considered. The process evaluations predicted little change in maximum month effluent ammonia concentrations between no dewatering and 100% dewatering. As shown in Table 8-3, secondary effluent TN concentrations increased as dewatering operations increase. The step feed configuration was less sensitive to an increase in dewatering compared to the current configuration. Greater dewatering filtrate loads to the secondary process may also impact dissolved oxygen control as there may be more significant variations in diurnal nitrogen loads.

Table 8-3
Filtrate Impact on Secondary Effluent TN Concentrations

Sludge to Dewatering	Current Configuration (mg/L)	Step Feed Configuration (mg/L)
0%	16.6	9.4
25%	17.7	9.9
50%	18.7	10.4
75%	19.7	11.1
100%	20.8	11.9

Several recycle treatment approaches to minimize the impacts of the dewatering filtrate on the liquid stream nutrient removal process were considered and are discussed in the following sections. The treatment approaches evaluated include:

- Filtrate Equalization
- Physical-chemical Treatment
- Biological Treatment Alternatives
- CASTion Ammonia Recovery Process
- OSTARA Process

8.2 Filtrate Equalization

Flow equalization of recycle streams is used to attenuate the loading of nutrients back to the liquid stream process and can be particularly effective when applied to batch dewatering processes. Filtrate flows can be stored and fed back to the liquid stream process at a constant rate or fed back during low diurnal loading periods. Feeding phosphorus rich sidestreams back to the liquid stream process during low influent load periods can lead to reduced biological phosphorus removal performance if adequate carbon is unavailable.

Several operational scenarios were evaluated to estimate flow equalization basin sizing and required return flows. The first scenario assumed a constant return flow rate, and the second scenario assumed that flow would be returned from equalization during a six-hour period per day. For each of these scenarios annual average and maximum month filtrate flows at the 14.5 mgd design flow were evaluated based on seven-day per week dewatering operations. Annual average filtrate equalization requirements were also developed for five-day per week dewatering operations. Seven-day per week dewatering will be necessary under maximum month solids production; therefore five-day per week operation was not considered for maximum month conditions. Table 8-4 summarizes the expected dewatering operation, required filtrate equalization volume and filtrate return flows necessary for each scenario.

Table 8-4
Filtrate Flow Equalization Requirements

Scenario	Dewatering Operation (hours/day)	Required EQ Volume (gallons)	Required EQ Return Rate (gpm)
1 - Constant EQ Return			
1a – Annual Average, 7days/wk Dewatering	11.1	32,000	42
1b – Maximum Month, 7days/wk Dewatering	15.6	5,200	86
1c – Annual Average, 5 days/wk Dewatering	23.0	140,000	42
2 - Filtrate Return 6 hrs per day			
2a – Annual Average, 7days/wk Dewatering	11.1	60,000	167
2b – Maximum Month, 7days/wk Dewatering	15.6	97,000	345
2c – Annual Average, 5 days/wk Dewatering	23.0	180,000	167

Providing for 0.2 MG of filtrate flow equalization will allow sufficient flexibility to attenuate filtrate flows associated with five-day per week dewatering. The estimated costs of filtrate equalization facilities including tankage and mechanical costs is $500,000. Benefits of providing filtrate equalization include increased secondary process reliability, flexibility in nutrient return to the liquid stream and the greater dissolved oxygen control.

8.3 Physical-Chemical Treatment

OWASA currently provides for chemical phosphorus removal through alum addition to the rotary press filtrate stream. The precipitated solids and any residual alum in the filtrate stream are returned upfront of the primary clarifiers. Addition of alum upstream of the primary clarifiers can lead to enhanced primary clarifier performance. Historical primary removal rates for TSS and BOD5 are 60% and 55%, respectively. Although TSS removal rates are typical of municipal wastewater, enhanced BOD5 removal has been observed, suggesting substantial removal of colloidal organic matter. Enhanced primary clarifier performance reduces the amount of solids and organics to the secondary process, which can lead to further carbon limitation for the BNR process and increase supplemental carbon requirements through additional removal of slowly biodegradable particulate and colloidal COD. Since primary sludge is fermented at the Mason Farm WWTP, a portion of this removed COD is likely hydrolyzed and returned to the secondary process with the fermentate, reducing the impacts somewhat. The precipitated inert solids also take up valuable aeration basin inventory.

Aluminum and iron salts are commonly used for the chemical removal of phosphorus from municipal wastewater. The most prevalent chemicals used for phosphorus removal are aluminum sulfate ($Al_2(SO_4)_3 \cdot 14H_2O$), commonly referred to as alum, and ferric chloride ($FeCl_3$). In addition, several other compounds can be used for phosphorus removal; ferrous sulfate and ferrous chloride (often referred to as "pickle liquor", a byproduct of steelmaking), sodium aluminate ($Na_2Al_2O_4$), and polyaluminum chloride (PACL). Ferrous sulfate and chloride can contain large quantities of hydrochloric or sulfuric acid, which can result in increased alkalinity reduction and pH depression and will not be considered further. A number of plants were originally designed for phosphorus removal through lime addition; however, lime addition for phosphorus removal is not commonly practiced anymore due to substantial sludge production and operational and maintenance issues associated with lime handling.

Several WWTPs in North Carolina have recently switched from alum and ferric chloride to sodium aluminate addition. The major benefit of sodium aluminate addition is that it is an alkaline solution and increases the alkalinity and pH of the wastewater it is fed to, which reduces sodium hydroxide feed requirements. A potential drawback to sodium aluminate addition is that it may raise the wastewater pH above the optimal pH range for chemical phosphorus removal, leading to reduced chemical phosphorus removal efficiency particularly where very low effluent phosphorus limits need to be met. Recent jar tests performed at other facilities have suggested that the use of sodium aluminate for phosphorus removal in the liquid stream is inefficient due to increasing pH at higher doses; however this phenomenon has not been reported during full scale use. The facilities that are currently adding sodium aluminate have reported greater phosphorus removal efficiency compared to alum addition, which may be due to greater BPR efficiency attributed to increased pH.

Chemical feed rates for alum, sodium aluminate and ferric chloride were estimated for the various dewatering filtrate flow rates under design flow annual average conditions and are presented in Table 8-5.

The addition of alum and ferric chloride consumes alkalinity. The resulting alkalinity consumption is 0.51 mg/L of alkalinity as CaCO3 per mg/L alum added and 0.93 mg/L of alkalinity as CaCO3 per mg/L FeCl3 added. The Mason Farm WWTP is alkalinity limited, and 25% sodium hydroxide is added upstream of the aeration basins to reduce nitrification inhibition due to low pH conditions. Addition of alum and ferric chloride increase the sodium hydroxide feed required, which should be accounted for when evaluating chemical phosphorus removal.

Table 8-5
Estimated Filtrate Chemical P removal Feed Rates

Sludge to Dewatering	Alum (gpd)	Sodium Aluminate (gpd)	Ferric Chloride (gpd)
Chemical Phosphorus Removal			
25%	56	17	37
50%	110	35	74
75%	170	52	110
100%	230	69	150
Biological Phosphorus Removal			
25%	320	97	210
50%	630	190	410
75%	950	290	620
100%	1,260	390	830

Sodium aluminate addition increases wastewater alkalinity and should reduce sodium hydroxide feed requirements. The net alkalinity increase is 0.61 mg/L of alkalinity as CaCO3 per mg/L sodium aluminate added. The estimated increase/decrease in 25% sodium hydroxide addition related to chemical phosphorus removal is summarized in Tables 8-6. Note that alum and ferric chloride have similar theoretical alkalinity consumption requirements per mass of phosphorus removed.

Table 8-6
Estimated 25% Sodium Hydroxide Requirements from Chemical Addition

Sludge to Dewatering	Alum (gpd)	Sodium Aluminate (gpd)	Ferric Chloride (gpd)
Chemical Phosphorus Removal			
25%	45	(-) 15	45
50%	90	(-) 30	90
75%	135	(-) 45	135
100%	180	(-) 60	180
Biological Phosphorus Removal			
25%	250	(-) 85	250
50%	500	(-) 170	500
75%	750	(-) 255	750
100%	1,000	(-) 340	1,000

Estimated costs for the usage of each chemical for filtrate phosphorus removal under annual average design conditions are summarized in Table 8-7. Chemical costs include the associated increase or decrease in sodium hydroxide feed requirements. Chemical costs are based on current costs quoted by Brenntag on 12/31/2009 and are as follows:

- 48% alum - $1.11/gallon
- 38% ferric chloride – $3.51/gallon
- 23.4% (as Al2SO4) sodium aluminate – $4.70
- 25% sodium hydroxide -$0.65/gallon

Table 8-7
Estimated Annual Filtrate Chemical Addition Costs - Chemical P Removal

Sludge to Dewatering	Alum	Sodium Aluminate	Ferric Chloride
Chemical Phosphorus Removal			
25%	$33,000	$26,000	$58,000
50%	$67,000	$52,000	$116,000
75%	$100,000	$79,000	$174,000
100%	$134,000	$105,000	$232,000
Biological Phosphorus Removal			
25%	$187,000	$146,000	$324,000
50%	$374,000	$293,000	$648,000
75%	$561,000	$439,000	$973,000
100%	$748,000	$586,000	$1,297,000

Switching from alum to sodium aluminate for chemical phosphorus removal should reduce overall chemical costs due to reduced supplemental alkalinity requirements. Ferric chloride is a much more expensive alternative at current costs. It is recommended that OWASA initiate bench-scale jar testing of alum and sodium aluminate addition to the filtrate and MLSS to determine field appropriate doses to refine the cost comparison. The jar tests are recommended as part of an overall phosphorus removal investigation and optimization plan.

The existing alum storage and feed facilities may be reused with minimal modifications for sodium aluminate storage and feed provided that the existing FRP storage tanks have a suitable inner resin liner and is structurally able to support the additional density associated with sodium aluminate. Typical pump and piping materials for both alum and sodium aluminate feed are similar and include PTFE (Teflon) pump diaphragms and valve materials, PVC piping and pump body liquid end constructed of PVC or PVDF (Kynar) materials. FRP tank and chemical metering pump materials will need to be verified to ensure compatibility with sodium aluminate. It is also important to make sure the tanks, pumps and piping are flushed out prior to sodium aluminate use to remove any residual alum in the system.

Chemical addition to the digested sludge prior to dewatering should also be considered to reduce inert suspended solids return to the liquid stream process. The impacts on the dewaterability of the sludge will need to be considered, and changes to polymer type or dose may be required. Alternatively a separate reactor clarifier could be constructed to provide mixing, flocculation and settling of the filtrate after chemical addition. This would require extensive re-piping of the filtrate and construction of new clarification facilities on a site with limited available footprint. Removing the impact of filtrate aluminum addition from the liquid stream may also decrease the enhanced primary clarifier effect and provide more carbon to the secondary process.

8.4 Biological Treatment Alternatives

Sidestream biological treatment was considered to reduce the total nitrogen loads returned to the liquid stream. Numerous proprietary recycle stream processes are available, and non-proprietary process configurations can provide for similar nitrogen removal from recycle streams. Biological treatment of high ammonia recycle streams is particularly attractive at facilities with high anaerobic digester operating temperatures such as the Mason Farm WWTP since many of the process configurations rely on high wastewater temperatures to promote the growth of specific bacteria. In addition the treated recycle flow from some processes can be used to seed the secondary process with nitrifiers under cold weather conditions. The various proprietary and non-proprietary biological processes are discussed in the following sections.

8.4.1 Nitrification/Denitrification

Nitrification/denitrification includes providing for separate sidestream nitrification of filtrate. An anoxic zone can be placed after the nitrification zone and supplemental carbon fed to reduce nitrate concentrations returned to the liquid stream. This process can be arranged in limitless configurations including flow through reactors in parallel with RAS/WAS addition to provide biomass, activated sludge configurations with a clarification step and return sludge pumping, fixed media, fixed film/activated sludge hybrids such as IFAS, sequencing batch reactors, etc. The main purpose of the process is to reduce the ammonia (and nitrate if denitrification is provided) loads returned to the main liquid stream. Since anaerobically digested filtrates typically have higher temperatures than the liquid stream process flow, nitrification and denitrification kinetics

are enhanced, and greater removals can be provided in limited volumes. Return of the biomass from the sidestream process can enhance liquid stream treatment performance and is discussed in the next section on Bioaugmentation.

8.4.2 Bioaugmentation

Bioaugmentation processes provide nitrification of filtrate in a separate sidestream reactor and return the nitrifying biomass to the mainstream liquid treatment process to increase the nitrification capacity of the secondary process. The seeding impact allows the secondary process to be operated at a lower solids retention time (SRT), which reduces MLSS concentrations and solids loadings to the secondary clarifiers and can help protect nitrification performance during cold weather. A portion of RAS or WAS flow is directed to the sidestream reactor to provide biomass for treatment. Bioaugmentation is a well developed, widespread technology that is capable of removing greater than 80% of ammonia from a high strength sidestream. Supplemental alkalinity addition is often required for this process.

Sidestream denitrification can also be provided in a bioaugmentation process if anoxic conditions and supplemental carbon are provided downstream of nitrification. This may be particularly beneficial at treatment plants utilizing methanol for denitrification, as methylotrophs will be returned to the liquid stream process, which could reduce the potential for methylotroph washout and loss of denitrification in a post-anoxic zone of a BNR process.

Proprietary bioaugmentation processes are available including InNitri® and BABE®. The InNitri® (Inexpensive Nitrification) process is marketed by Mixing and Mass Transfer (M2T) Technologies and includes primary effluent addition to the sidestream reactor instead of RAS or WAS. The BABE® (BioAugmentation Batch Enhanced) process, developed by DHV and University of Delft (The Netherlands), provides for separate centrate treatment in a sequencing batch reactor (SBR). Additional bioaugmentation processes include Prenitrifcation, BAR (Bio-Augmentation Reaeration) and MAUREEN (Mainstream Autotrophic Recycle Enabling Enhanced N-removal).

8.4.3 Nitritation/Denitritation

In the Nitritation process, the ammonia load in the sidestream is oxidized to nitrite prior to return to the liquid stream. The ammonia oxidation is stopped at the nitrite step by operating the process at an elevated temperature (30 to 40°C) and low SRT (1 to 2 days). At higher temperatures, the ammonia-oxidizing bacteria grow faster than the nitrite-oxidizing bacteria; therefore, the slower nitrite oxidizers are washed out of the system. Limiting ammonia oxidation to nitrite reduces the amount of oxygen required and reduces aeration energy compared to full nitrification to nitrate. Another potential advantage is reduction in the carbon required to reduce nitrite to nitrogen gas compared to reducing nitrate.

Although this process has been widely established in Europe, North American installations are limited. This process can remove more than 90% of the ammonia available in the sidestream. Supplemental alkalinity addition is usually required for this process, and external heating may be necessary depending on filtrate and ambient temperatures. The SRT, pH and temperature must be carefully controlled in this process to prevent the growth of nitrite-oxidizing bacteria.

Denitritation is the reduction of nitrite directly to nitrogen gas. A sidestream anoxic zone and supplemental carbon addition can be provided downstream of the nitritation zone to provide for denitritation. As mentioned previously this reduces supplemental carbon requirements compared to conventional nitrification/denitrification. The SHARON® (Single Reactor for High activity Ammonia Removal Over Nitrite) process, marketed by Mixing and Mass Transfer (M2T) Technologies, is a proprietary combined nitritation/denitritation process which occurs in a single reactor and is capable of reducing filtrate TN concentrations by 80%.

8.4.4 ANAMMOX

The ANAMMOX® (Anaerobic AMMonium Oxidation) process is a variation of the Nitritation/Denitritation process. This process was originally configured as an initial nitritation reactor followed by an unaerated reactor (SHARON®/ANAMMOX® process). The Nitritation process provides for partial nitritation, and typically aeration is limited to convert only 50% of the ammonia to nitrite. The ammonia/nitrite mixture is directed to

the ANAMMOX® reactor, where autotrophic bacteria (Anammox bacteria) reduce the ammonia/nitrite mixture to nitrogen gas. Ammonia is used as an electron donor under anoxic conditions, and the nitrite is used as the electron acceptor. Supplemental carbon addition is not typically required for this process provided it is properly controlled. Supplemental alkalinity addition is required.

Installations of ANAMMOX® process are limited worldwide. There are five known full-scale facilities worldwide, none of which are in North America. Careful temperature control is crucial to the process, and must be maintained near 36°C. The Anammox bacteria are very slow growing, and a comparably large reactor volume must be provided to avoid washout. Total nitrogen removal rates greater than 80% can be achieved.

The ANAMMOX process was developed at the University of Delft (The Netherlands), and Paques Engineering holds the license to the ANAMMOX process. Several different proprietary processes have been developed including DEMON (DEamMONification), CANON (Completely Autotrophic Nitrogen removal Over Nitrite) and OLAND (Oxygen Limited Autotrophic Nitirifcation/Denitrification). The major differences in these processes relate to the control strategy used to maintain optimal process conditions.

8.4.5 Applicability to the Mason Farm WWTP

The existing liquid stream treatment facilities have sufficient capacity to reduce TN concentrations to 3 mg/L through supplemental carbon addition to the denitrification filters even with 100% dewatering. The secondary process can be optimized for TN removal by operating in a step feed mode as discussed previously, which also reduces the sensitivity of the liquid stream process to nitrogen loads returned from dewatering and reduces alkalinity and aeration costs. Each of the sidestream biological treatment processes described herein would require substantial capital investment, and energy and chemical costs for those requiring nitrification/denitrification would be similar. Nitritation/denitritation processes may provide for a savings in energy associated with aeration and supplemental carbon, but require careful operational control.

Since OWASA has the ability to apply liquid biosolids as a means of beneficial reuse and operationally reduce the impacts of nitrogen return, it is recommended that OWASA defer major capital expenses in providing for sidestream nitrogen removal until the facility is expanded beyond 14.5 mgd or liquid biosolids application is no longer feasible. At that time nitritation/denitritation processes will likely be better established in the United States, and these processes may provide for significant energy and carbon footprint reductions compared to treatment of the additional loads in the liquid stream process.

8.5 CASTion Ammonia Recovery Process

The CASTion Ammonia Recovery Process (ARP®) is a proprietary sidestream treatment process marketed by ThermoEnergy specifically intended to reduce ammonia in solids treatment process recycle streams. Reverse Controlled Atmospheric Separation Technology (RCAST®) separates ammonia from the wastewater. This process involves addition of sodium hydroxide to raise the filtrate pH and convert soluble ammonium (NH4+) to aqueous ammonia gas (NH3(aq)) and then providing high temperature, low pressure conditions to drive the ammonia gas out of solution. Sulfuric acid solution is added with the ammonia gas to form a stable ammonium sulfate solution. An ion-exchange process further removes ammonia from the filtrate. The manufacturer claims that filtrate ammonia concentrations can be reduced to less than 1 mg/L with this technology.

Significant pretreatment of the filtrate will likely be required with this technology. Maceration, screening, dissolved air floatation, filtration, ultraviolet disinfection and softening of the filtrate stream have been proposed on other projects considering this technology and would likely be necessary at the Mason Farm WWTP. These create substantial reject streams which are returned to the front of the plant and add to the operations and maintenance costs of the system.

The CASTion ARP process does not appear to have any cost benefit in comparison to providing additional supplemental carbon feed to the filters, and there appears to be significant operations and maintenance concerns as indicated by the need for detailed pretreatment systems. Since limited full scale operational history exists for this process, the CASTion ARP process is not recommended for further consideration at the Mason Farm WWTP unless long term, full scale data from an operating facility similar in magnitude to the Mason Farm WWTP proves otherwise.

8.6 OSTARA process

The OSTARA process is a proprietary phosphorus removal technology in which appropriate stoichiometric doses of magnesium chloride and sodium hydroxide are added to the dewatering filtrate, combining ammonia and orthophosphate in the filtrate to precipitate struvite pellets. A small reduction in ammonia load occurs through this process as well. The struvite pellets are subsequently marketed by OSTARA as fertilizer under the brand name Crystal Green®. The OSTARA process is a good application for plants that have BPR and anaerobic digestion since these processes result in relatively high concentrations of phosphorus in process streams. Advantages of the OSTARA process include:

- Reduction in ferric chloride feed
- Production of a marketable fertilizer
- Multiple procurement options
- Targeted application for specific wastewater characteristics

Disadvantages of the OSTARA process include:

- Newer technology
- High capital cost
- Proprietary process
- Requires equalization storage

The Mason Farm WWTP has the ability to either send digested sludge to the rotary press in order to produce a dewatered cake for composting or liquid land apply digested sludge. Based on information received from OSTARA, 5 treatment trains are required in order to meet maximum filtrate flow and load conditions resulting from sending 100% of sludge to dewatering after BPR in the liquid treatment train. The OSTARA reactor trains are mass loading limited; a single treatment unit can process a maximum of 150 pounds of phosphorus per day. Anticipated process requirements and percent reduction in total phosphorus for three filtrate phosphorus scenarios are presented in Table 8-8.

Table 8-8
OSTARA Process Requirements

	Lower Concentration Feed	Middle Concentration Feed	Higher Concentration Feed
Feed Flow Rate, gpd	60,000	60,000	60,000
Feed Ammonia Concentration, mg-N/L	1,250	1,250	1,250
Feed Ortho-Phosphate Concentration, mg-P/L	250	700	1,400
Treatment Capacity per Reactor, gpd	73,555	25,488	12,279
Number Reactors Required for Full Treatment	0.82	2.35	4.89
Proposed Number of Reactors	1	3	5
Redundant Plant Capacity (Allowance for Growth etc.)	23%	27%	2%
Maximum Crystal Green™ Production Rate per Reactor, lb/d	1,109	1,141	1,113
Expected Reactor Effluent Quality – Ortho-Phosphate, mg-P/L	22	22	27
Expected Removal Efficiency - Ortho-Phosphate	91%	97%	98%
Mass of Phosphorus Removed, lb/d	114	339	686
Expected Reactor Effluent Quality – Ammonia, mg-N/L	1,147	943	629
Expected Removal Efficiency - Ammonia	8%	25%	50%
Mass of Nitrogen Removed, lb/d	52	153	310
Struvite Production Rate, tons/yr	165	490	992
Greenhouse Gas Emissions Reduction, tons CO2e/year	2,736	8,235	16,735

Table 8-9 summarizes the required number of OSTARA reactors required depending upon the percentage of digested sludge sent to the rotary press for dewatering. One OSTARA reactor is not fully utilized under any of the CPR loading scenarios since the maximum loading to the OSTARA process is 125 pounds of phosphorus per day (resulting from sending 100% of sludge to dewatering) and the full capacity of a reactor is 150 pounds of phosphorus per day.

Table 8-9
Required Number of OSTARA Reactors

Percent to Dewatering (%)	Chemical Phosphorus Removal	Biological Phosphorus Removal
25	1	1
50	1	3
75	1	4
100	1	5

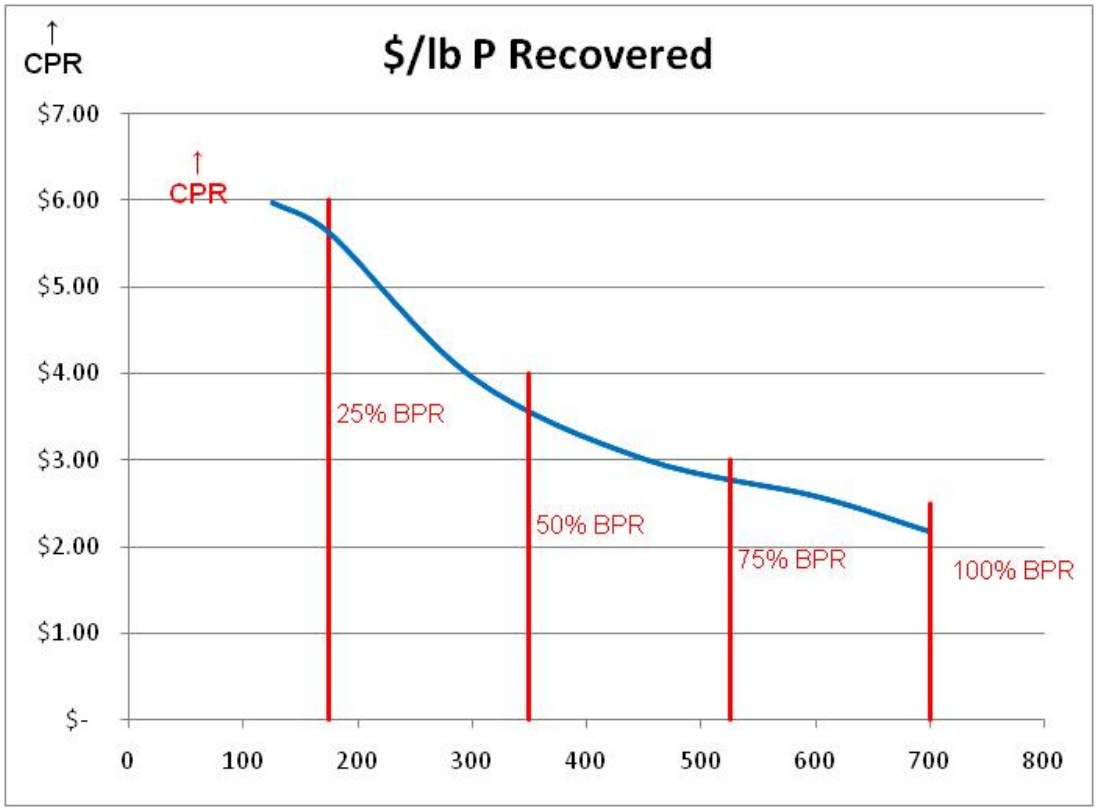
Figure 8-1 presents the cost of the OSTARA process per pound of phosphorus removed provided by OSTARA. Costs include installed costs for the OSTARA equipment amortized over a 15 year period and operational costs including chemicals, power, labor, and equipment maintenance. Fertilizer revenues are accounted for but the cost of a building to house the equipment is not included.

Chemical phosphorus removal facilities already exist at the Mason Farm WWTP. Chemical costs for conventional chemical P removal are $2.90/lb TP removed and $2.30/lb TP removed for alum and sodium aluminate, respectively. Table 8-10 presents approximate costs per pound of TP removed using the OSTARA process. The analysis indicates that BPR has to be optimized and at least 75 percent of digested sludge must be sent to dewatering in order for the OSTARA process to be cost competitive with conventional chemical phosphorus removal.

Table 8-10
Approximate Costs per Pound TP Removed

Percent to Dewatering (%)	Chemical Phosphorus Removal	Biological Phosphorus Removal
25	> $6	$5-$6
50	> $6	$3-$4
75	> $6	$3
100	$6	$2

Figure 8-1
OSTARA Cost Curve



The OSTARA process, with a primary focus on phosphorus removal, also achieves nitrogen removal. The benefits of the removal of nitrogen species in the filtrate stream were not quantified in this initial analysis and will ultimately have a positive impact on the liquid train since the nitrogen loading in the return stream is reduced through the OSTARA process. Aeration, alkalinity and supplemental carbon feed rates are expected to be reduced.

There are two ways in which OSTARA equipment can be procured. The first way is through an outright capital purchase option in which OWASA would pay OSTARA a lump sum and then receive annual credits for the fertilizer produced. Under this option, all operation and maintenance would be the responsibility of OWASA. Under the second procurement option, the treatment fee option, OSTARA will provide the necessary equipment, maintain the facility and charge OWASA a fee for operating the facility. The fertilizer produced would be owned and marketed by OSTARA.

The OSTARA process is feasible for implementation at the Mason Farm WWTP only if biological phosphorus removal is optimized and the most if not all digested sludge is sent to dewatering. It is recommended that this option be revisited upon completion of the proposed phosphorus optimization study.

8.7 Recycle Stream Treatment Recommendations

The need for recycle stream treatment will depend on the fraction of digested biosolids sent to dewatering. If the current practice of sending 50% or more of digested solids to the rotary press is maintained providing for flow equalization to attenuate the impacts of nitrogen returned in the sidestream is recommended prior to 2016 when the Jordan Lake Rules for nitrogen removal become effective. In addition to equalization, OWASA may want to consider other process enhancements to reduce nutrients in the recycle streams. Processes like OSTARA could produce a marketable fertilizer product while reducing nutrients in the recycle streams. Nitritation/denitritation is a promising technology which may provide significant chemical and energy savings to OWASA. The progress of this technology should be followed and evaluated prior to future expansion of the plant.

As shown in Table 8-7, the costs of treatment of recycle streams is expected to increase significantly with the optimization of biological phosphorus removal. The estimated additional impacts of dewatering on chemical feed (and aeration) requirements for TN and TP removal are shown below based on current chemical and energy costs:

- Aeration for additional TKN removal - $4.74/DT
- Acetic acid for additional NO3-N removal - $62.93/DT

- Sodium hydroxide for decreased alkalinity due to additional ammonia oxidation - $22.84/DT
- Alum for additional TP removal (assumes worst case TP load) - $131.47/DT
- Sodium hydroxide for decreased alkalinity due to additional alum - $81.03/DT

The total increase in treatment costs associated with dewatering is estimated at $303/DT. Of this estimated cost, over two-thirds is related to removing high phosphorus concentrations in the recycle streams. There isn't good historical data on how much TP should be expected from the Mason Farm filtrate when BPR is working well. Some research indicates that thermophilic-mesophilic digestion operation substantially reduces soluble P release during digestion. If this proves to be the case at Mason Farm WWTP the additional costs of nutrient removal may be only in the $150/DT range. Further study of this issue is planned during recommended biological phosphorus removal optimization. Understanding the fate of P in the digested sludge is an important step in the overall optimization of the plant from both a nutrient and solids handling perspective.

The OSTARA process or similar phosphorus recovery systems are most cost effective at large facilities with high filtrate phosphorus loads. Although OSTARA is not economically viable under current operations, increased biological phosphorus efficiency and dewatering operations may warrant its consideration if filtrate phosphorus loads increase.

Section 9.0
Future Expansion Considerations

The next logical expansion to the Mason Farm WWTP was identified in the Recommended Facilities Plan, Brown and Caldwell, April 2009 as 18.5 mgd. Review of the plant flow projections as discussed in Section 2.0 indicates that the current capacity of 14.5 mgd will be sufficient through the year 2030 and that the next expansion to a capacity of 18.5 mgd would be reasonable. Expansion to 18.5 mgd would provide sufficient capacity through 2060 based on the current flow projections. This section discusses considerations that for the 18.5 mgd expansion and offers a preliminary site layout.

9.1 Liquid Stream Expansion

The existing major liquid stream unit processes were evaluated to develop preliminary costs and site layouts at an 18.5 mgd design flow. The preliminary evaluation includes the following assumptions, which will need to be reevaluated during preliminary design of the expansion:

- No increase in wastewater strength over current conditions
- Peak sustained flow factor of 2.5 resulting in a peak flow of 46.3 mgd
- No additional effluent limits are imposed on the Mason Farm WWTP
- Current TN and TP mass allocations remain the same

The following major unit processes were evaluated in this section:

- Preliminary treatment
- Primary treatment
- Secondary treatment
- Effluent filtration, disinfection and disposal

The three existing influent screens are rated at a peak flow of 23.3 mgd each. The firm screening capacity is 46.6 mgd; therefore sufficient screening capacity exists at the 18.5 mgd design flow and no expansion to screening facilities is anticipated. Each of the two vortex grit removal systems has a capacity of 50 mgd for a total capacity of 100 mgd, and sufficient grit removal capacity is available at the 18.5 mgd expansion. The existing influent Parshall flume has a flow measurement capacity of 55 mgd. Expansion of the preliminary treatment facilities will not be necessary for expansion of the Mason Farm WWTP to 18.5 mgd.

The existing primary clarifiers have a peak flow capacity of 34.5 mgd based on a typical design peak flow rate of 3,000 gpd/sf. Although the anticipated peak flow associated with the 18.5 mgd design flow is 46.3 mgd, flows greater than 34.5 mgd will be infrequent, and influent wastewater strength will be dilute during peak flow events associated with wet weather. Excess influent flow can also be diverted around the primary clarifiers, limiting hydraulic impacts of the increased flow on primary clarifier performance. The 46.3 mgd peak design flow associated with the 18.5 mgd design condition is slightly greater than the current 43.5 mgd design flow. Construction of additional primary clarifier capacity is not expected to significantly enhance treatment at the 18.5 mgd design flow; therefore additional primary treatment facilities are not recommended as necessary to expand the Mason Farm WWTP to 18.5 mgd.

Section 6 of this report develops several alternatives for expanding the capacity of the secondary process including:

- Construction of additional aeration cells
- Conversion of the NSL basin to aeration cells
- Incorporation of integrated fixed film activated sludge (IFAS)
- Converting the existing process into a membrane bioreactor process (MBR)

The ultimate technology chosen will be dependent on future regulatory limits. It is assumed that no additional effluent limits would be imposed on the Mason Farm WWTP for the purposes of this future expansion evaluation. Incorporation of IFAS media would require that the proposed fine bubble diffused aeration system be replaced with a coarser bubble system to provide suspension and scour of the IFAS media. This would likely increase both the capital and operational costs of the IFAS option. Conversion to

an MBR process would require new membrane tanks, screening facilities and a new membrane equipment building, and significant flow equalization volume (approximately 10 mgd) would be required to limit peak flows to reduce costs associated with membrane capital and replacement requirements. Although MBR is the most expensive option to meet current effluent standards, it will be more economically viable if further effluent restrictions are imposed.

Construction of new aeration cells or conversion of the existing NSL cells to aeration volume would likely be the lowest cost expansion to the Mason Farm WWTP assuming no additional effluent limits. There is no room immediately adjacent to the existing aeration basins to construct additional volume. Additional aeration cells would need to be constructed remotely from the existing basins, and providing efficient flow distribution and process flexibility would be challenging. Modifying the existing NSL cells to aeration volume would essentially allow expansion of the existing aeration basins while maintaining current flexibility and flow distribution. Conversion of the NSL basins to aerated cells will require construction of new anaerobic and anoxic tankage to provide for a similar level of nutrient removal as proposed with the 14.5 mgd conversion to a step feed process.

Conversion of the NSL basins to aeration volume and construction of new anaerobic and anoxic zones would allow OWASA to continue to provide the majority of nutrient removal in the secondary process and limit chemical feed, and is the preferred option for expanding to 18.5 mgd assuming the current effluent limits are maintained. Flexibility to operate as the OWASA process or a more traditional three-stage BNR process could be provided by allowing primary effluent to bypass the anaerobic zone and be fed directly to the anoxic zone. RAS and fermentate would still be distributed to the anaerobic zone. A nitrified recycle (NRCY) pump station would be provided to pump nitrified MLSS from the end of the aeration basins to the anoxic zone. A new primary effluent pump station is proposed to pump primary effluent to the anaerobic or anoxic cells and allow gravity flow from the new structure back to the aeration basins to avoid repumping. The estimated volume required for the new anaerobic/anoxic basins is approximately 3.6 million gallons, and these facilities could be located in the area currently occupied by the existing trickling filters. Flexibility could also be provided to allow post-anoxic operation of some of the aeration cells during lower flow/warm weather conditions. Additional denitrification could also be provided through supplemental carbon addition to these zones.

One additional 120-foot diameter clarifier is proposed under the expansion to 18.5 mgd to limit peak clarifier overflow rates to 800 gpd/sf. Existing facilities are in place to distribute flow to a new clarifier. It is anticipated that the new clarifier could be located to north of the filters.

Construction of two additional filter cells will be required at 18.5 mgd to limit the design hydraulic loading rate (assuming one filter out of service) to 2.3 gpm/sf to provide reliable denitrification across the filters. Expansion of the Mason Farm WWTP will require that an additional UV disinfection channel and equipment be installed to meet NCDWQ redundancy criteria. One additional UV disinfection channel similar to the current design will provide for a firm capacity of 48 mgd and is sufficient for an expansion to 18.5 mgd. Effluent pumping is required during high flood conditions. The existing effluent pumping station has a firm capacity of 52.5 mgd and should have sufficient volume for the 18.5 mgd expansion.

9.2 Hydraulic Considerations

A detailed hydraulic evaluation will need to be conducted for the proposed 18.5 mgd facility in the future once a final facility layout has been determined. A preliminary evaluation of the Mason Farm WWTP site layout at 18.5 mgd indicates that the most significant hydraulic improvements at the plant involve providing pumping facilities associated with the anaerobic/anoxic tankage. The additional tank volume is proposed in order to provide sufficient secondary treatment capacity at 18.5 mgd; forward flow through the plant will flow from these tanks to the existing aeration basins for aerobic treatment. In order to avoid pumping of primary effluent, RAS, and nitrified recycle flows from these tanks to the existing aeration basins, it is recommended that a primary effluent pump station be constructed in order to lift flows into the anaerobic/anoxic tanks in order to subsequently allow flow by gravity to the aeration basins. The existing intermediate pump station would be abandoned at 18.5 mgd. Distribution of mixed liquor to the secondary clarifiers, identified as a hydraulic constraint at 14.5 mgd, may continue to be an issue at 18.5 mgd; this should be further investigated in the future detailed hydraulic evaluation. Piping improvements that are discussed in Section 2 should be incorporated into any expansion of the capacity of the plant to improve peak flow capacity.

9.3 Solids Stream Expansion

Solids handling facilities at the Mason Farm WWTP will need to be re-evaluated in detail for a plant expansion to 18.5 mgd. Existing major unit processes on site that were evaluated in this section are:

- Primary Sludge Fermentation
- Digester Feed Sludge Thickening
- Anaerobic Digestion
- Biosolids Storage
- Biosolids Dewatering

Solids production rates for the projected future flow of 18.5 mgd are presented in Table 9-1.

Table 9-1
Estimated Solids Production Rates – 18.5 mgd

	Annual Average Conditions	Maximum Month Conditions
Plant Influent Flow Rate, MGD	14.8	18.50
Primary Sludge Production, lbs(dry)/MG	1,500	1,700
Raw Primary Sludge Flow, gallon/day	133,094	188,549
Secondary Sludge Production, lbs(dry)/MG	1,300	1,500
Raw Secondary Sludge Flow, gallon/day	230,695	332,734
Note: Raw Primary Sludge Flow based on 2.0% solids Raw Secondary Sludge Flow based on 1.0% solids		

The existing primary sludge fermenter has adequate capacity to handle future annual average loadings. Fermenter residence time approaches the recommended duration of 2 days at the future 18.5 mgd condition but additional tankage is not recommended. Following fermentation primary sludge solids are combined with waste activated sludge solids for thickening in the gravity belt thickening facility. Blended sludge solids mass and volumes are estimated as shown in Table 9-2

Table 9-2
Estimated Blended Solids Production Rates – 18.5 mgd

	Annual Average Conditions	Maximum Month Conditions
Plant Influent Flow Rate, MGD	14.8	18.50
Blended Sludge Rate, lbs(dry)/day	41,440	59,200
Raw Blended Sludge Volume, gallons/day	363,789	521,283
Raw Blended Sludge Concentration, %TS	1.4%	1.4%

Gravity belt thickening process firm capacity would be estimated at 54,000 dry pounds per day based on continuous operation of two 1.5-meter width GBT thickening units at 750 lbs(dry)/hour-meter solids loading rate. This capacity is sufficient under annual average loads associated with the future 18.5 mgd flow. However, the process units would not be adequate under maximum month conditions at this solids loading rate. Operating the existing equipment at maximum month conditions corresponds to a surface loading rate of 825 lb/hr-meter with one unit out of service. It is recommended that the plant staff stress test the existing equipment at the higher loading rate in order to confirm the existing processing capacity and performance. If existing equipment can handle the increased loading rates without a significant decrease in performance, additional thickening capacity would not be required for the 18.5 mgd expansion.

Additional anaerobic digester volume will be required for the 18.5 mgd expansion. Two additional digesters, each with a minimum capacity of 510,000 gallons, are recommended in order to meet a recommended hydraulic residence time of 15 days and 5 days under thermophilic and mesophilic temperatures, respectively. Table 9-3 presents a summary of the existing anaerobic digester volumes and proposed minimum sizing for additional digester volume.

Table 9-3
Anaerobic Digester Volumes

	Volume (gal)	Temperature
Digester #1	657,000	Mesophilic
Digester #2	310,000	Thermophilic
Digester #3	310,000	Thermophilic
Digester #4	510,000	Thermophilic
Digester #5 (Proposed)	510,000	Thermophilic
Digester #6 (Proposed)	510,000	Thermophilic
Total	**2,807,000**	**--**

Digested solids are currently stored prior to land application. At the future 18.5 mgd design condition, the existing post-digestion sludge storage can provide more than 30 days storage at annual average conditions. Table 9-4 provides the anticipated sludge storage volume requirements for the expanded flow.

Table 9-4
Estimated Post Digestion Solids Production Rates – 18.5 mgd

	Annual Average Conditions	Maximum Month Conditions
Plant Influent Flow Rate, MGD	14.8	18.50
Post Digestion Solids Mass, lbs(dry)/day	25,991	37,130
Raw Blended Sludge Volume, gallons/day	88,535	126,479
Raw Blended Sludge Concentration, %TS	3.5%	3.5%
Note: Blended Sludge Feed to Digesters from GBT = 5.5% GBT Solids Capture Efficiency = 98% Pre-Digestion VS/TS Fraction = 80% Anaerobic Digestion VS Reduction = 45%		

At 18.5 mgd, digested sludge may be handled as a stabilized liquid product or as dewatered cake. Existing dewatering equipment, a rotary press, may be maintained but does not have sufficient capacity to dewater all digested solids at 18.5 mgd. Additionally, expansion is not expected for another 20 years and the rotary press may be in need of replacement at that time. The Mason Farm WWTP should give further consideration to modifying the existing dewatering building to accommodate a high solids centrifuge as part

of the expansion to 18.5 mgd. A high solids centrifuge with a mass capacity of at least 2,000 lb(dry) per hour (48,000 dry pounds per day) would have sufficient capacity to meet a 100% dewatering requirement with near continuous operation. If OWASA determines to dewater all sludge at the Mason Farm WWTP at some point in the future, a redundant dewatering centrifuge and associated dewatering building expansion should be considered.

9.4 Site Constraints

Expansion of the Mason Farm WWTP to 18.5 mgd will likely require a significant effort to resolve issues relating to floodplain and floodway boundaries and site permitting. A new Flood Insurance Rate Map (FIRM) panel containing the plant site was released by the Federal Emergency Management Agency (FEMA) in 2006. The map, generated using Light Detection and Ranging (LIDAR) technology to collect topographic data, omitted important topographic features including the Morgan Creek bypass channel and existing flood control berm. A flood study was conducted as part of the Town of Chapel Hill's special use permitting process for the reuse project constructed in 2007.

An agreement was made between OWASA and the Town of Chapel Hill that prior to any additional construction on site, a full letter of map revision (LOMR) will be submitted to FEMA in order to formally have the FIRM panel revised. Location of facilities required for the 18.5 mgd expansion will need to be confirmed in order to ensure that they are not located in the Morgan Creek floodway.

9.5 Cost Estimate

A planning level cost estimate for the expansion of the Mason Farm WWTP to 18.5 mgd is presented in Table 9-5. Additionally, there is a column in Table 9-5 that indicates whether the recommended improvement presented herein was also included in the Recommended Facilities Plan, 2009, prepared by Brown and Caldwell. Figure 9-1 presents a proposed site layout for the Mason Farm WWTP at 18.5 mgd. Costs for the 18.5 mgd expansion are estimated at approximately $60 million in January 2010 dollars.

Table 9-5

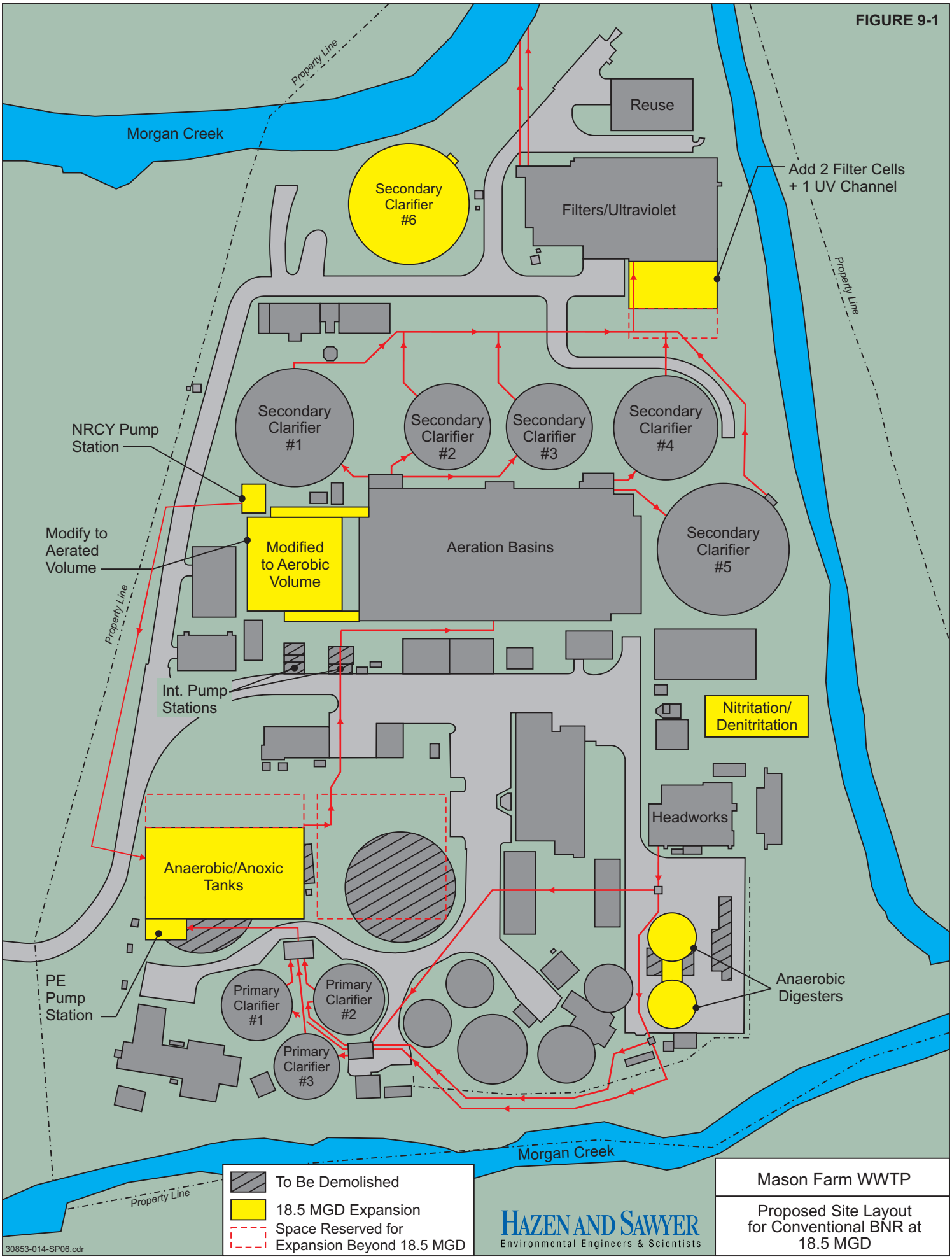
Cost Estimate for 18.5 mgd Expansion

Item	Cost Estimate	Included in B&C Facilities Plan for 18.5 mgd?
New Primary Effluent Pump Station	$6,200,000	Yes**
New Biological Process Volume	$7,200,000	Yes
NSL Basin Modifications	$1,100,000	Yes
NRCY Pump Station	$1,670,000	Yes
Aeration Basin Hydraulic Modifications	$200,000	Yes
Secondary Clarifier Improvements (includes RAS/WAS)	$2,690,000	Yes
Deep Bed Filters	$1,430,000	Yes
UV Disinfection	$2,500,000	Yes
Gravity Belt Thickener	$750,000	No
Anaerobic Digesters	$9,500,000	No
Dewatering Centrifuge	$1,500,000	No
Yard Piping	$1,000,000	Yes
SCADA System Expansion*	$100,000	Yes
Electrical Improvements	$4,500,000	Yes
Morgan Creek Bypass Improvements*	$500,000	Yes
Subtotal	**$40,840,000**	-
Contingency (20%)	$8,170,000	-
Subtotal	**$49,010,000**	-
Engineering and Administration (20%)	$9,800,000	-
Total	**$58,810,000**	-

***from Brown and Caldwell Recommended Facilities Plan**

****Recommended improvement is replacement of existing intermediate pump station**

FIGURE 9-1



HAZEN AND SAWYER
Environmental Engineers & Scientists

Mason Farm WWTP

Proposed Site Layout for Conventional BNR at 18.5 MGD

Section 10
Recommendations

OWASA engaged Hazen and Sawyer to perform a Hydraulic and Treatment Capacity Study for the Mason Farm WWTP in light of the proposed additional limitations on discharge of nutrients under the Jordan Lake Rules. Hazen and Sawyer evaluated Nutrient Removal Optimization Alternatives, Aeration Capacity Alternatives, Secondary Treatment Capacity Expansion Alternatives, and Recycle Stream Treatment Alternatives for the Mason Farm WWTP. Additional engineering investigations were also identified as needed to allow OWASA to optimize treatment while minimizing operating costs. This section summarizes the recommendations for improvements to the plant resulting from the Hydraulic and Treatment Capacity Study. Some initial modifications have been implemented; further recommendations stemming from these modifications are also identified in this section.

Nutrient Removal Process Configuration

The Mason Farm WWTP has a unique aeration basin design with flexibility to be operated in several process configurations. It was recommended that OWASA operate the existing secondary treatment facility in the 3:4 configuration (3 treatment trains of 4 aeration cells each) and provide for step feed of primary effluent flow to the first and second cells of each treatment train as discussed in the nutrient removal optimization evaluation (Section 6.5.6). The second and third cells of each train should be operated anoxically under typical conditions. The third cell can be operated aerobically under cold weather conditions as influent loads increase to provide full nitrification. Operation in the proposed step feed configuration will promote further denitrification in the aeration basins and limit the amount of denitrification and supplemental carbon feed required in the denitrification filters. The step feed configuration will also decrease aeration requirements and sodium hydroxide feed for alkalinity addition, further decreasing operational costs.

The Mason Farm WWTP process was transitioned to the step feed configuration in June 2010 as described in the Process Transition Plan presented in Appendix L. Detailed

process data and nutrient profiles were collected once the system began operating in the step feed configuration to verify system performance and to allow further process optimization if necessary. Operation in the step feed configuration resulted in significantly greater total nitrogen removal through the secondary process and substantially reduced chemical feed requirements. The success of the process transition to step feed is documented in the Process Transition Results Technical Memorandum provided in Appendix M. Additional opportunities for process optimization and operational cost savings were identified. Further optimization of biological phosphorus removal and cold weather step feed operation to ensure year-round compliance with the facility's ammonia limits while maximizing total nitrogen removal are proposed to continue after the completion of this study. In addition, capital improvements are recommended to enhance treatment and meet various OWASA operational goals as discussed below.

Chemical Phosphorus Removal Improvements

The Mason Farm WWTP has to meet more stringent total phosphorus limits effective in 2010 as part of the Jordan Lake Rules. Chemical jar testing was conducted to verify whether alum to the appropriate chemical precipitant to use for chemical phosphorus removal. Sodium aluminate was investigated as an alternate chemical because of its more favorable impact on process pH and alkalinity; however, testing indicated reduced phosphorus removal efficiency with the use of sodium aluminate when compared to alum. An evaluation of the existing alum feed facility's materials of construction indicates that they are compatible with sodium aluminate and the feed of sodium aluminate can be accommodated in the future if desired. However, it is recommended that OWASA continue to use alum for the foreseeable future.

Chemical jar testing performed during the process transition indicated lower alum utilization for phosphorus removal when dosed to the secondary effluent compared to the MLSS feed point current utilized. This finding indicates that a multi-point dosing approach would allow lower chemical usage as compared to the current operation. A chemical feed point for alum or other metal salts addition should be added upstream of the existing filters to allow additional phosphorus trim through the filters to improve phosphorus removal reliability and minimize chemical costs. Piping exists to feed alum to the filters but chemical feed pump equipment must be installed in order to create a

fully functioning system.

Investigations also indicate that excessive feed of alum to the filtrate return stream is hindering the performance of biological phosphorus removal. Guidance was developed to reduce the alum feed rate to the filtrate return stream and to perform the majority of the phosphorus removal in the MLSS and at the filters.

The estimated cost of installing alum pumps to allow dosing upstream of the filters is $30,000.

Biological Phosphorus Removal & Cold Weather Step Feed Optimization

The efficiency of the biological phosphorus removal (BPR) process at the Mason Farm WWTP requires further investigation. The historical data, specifically substantial chemical feed requirements and low filtrate phosphorus concentrations, indicate an inefficient BPR process, and the majority of phosphorus removal appears to be achieved through chemical addition. Excessive alum has historically been fed to the filtrate return stream and this may be limiting the phosphate available and conditions required for the BPR organisms to proliferate in the secondary treatment process. In addition, the use of acetic acid to supplement the VFA's produced during fermentation may cause proliferation of glycogen-accumulating organisms (GAOs), which compete with the phosphate-accumulating organisms (PAOs) for VFAs.

Modifications to the treatment process were recommended as part of the Process Transition Plan presented in Appendix L. The step feed process successfully reduced nitrate concentrations through the secondary process, resulting in reduced nitrate load to the NSL cells. The reduction in nitrate to the NSL cells decreases the amount of carbon utilized for denitrification in the NSL cells, providing more available carbon for BPR. Increased BPR performance was observed during the process transition as evidenced by greater P release in the anaerobic zone, better aerobic zone uptake and greater phosphorus content of the biomass. Acetic acid and alum feed rates were reduced; however, further optimization of the biological phosphorus removal process is recommended to decrease the dependence on external chemical addition to drive phosphorus removal.

Further optimization of BPR should focus on balancing the use of carbon in the facility. Optimizing the fermentation process to enhance VFA production should be investigated to further decrease the reliance on acetic acid. Recovery of endogenous carbon from waste sludge should also be considered. The impacts of thermophilic-mesophilic anaerobic digestion on the release of soluble phosphorus during digestion should also be evaluated in order to optimize treatment of phosphorus being returned to liquid treatment from the solids processing operations. Further optimization of step feed operations for cold weather conditions would also be a part of the BPR optimization. Reductions in nitrate loading to the NSL basins from the RAS flow should have an added benefit in improving BPR performance.

The estimated cost of additional investigation and optimization of the biological phosphorus removal process is approximately $75,000.

Aeration Capacity Improvements

Evaluations of the existing treatment capacity indicated that there is insufficient aeration capacity for the 14.5 mgd capacity rating, as discussed in Section 6.3. In addition, the existing jet aeration equipment is significantly less efficient than fine bubble diffused aeration equipment. It is recommended that the existing jet aeration system be replaced with fine bubble diffusion within the next three years to meet greater aeration demands as flows increase while reducing energy usage and costs. Costs associated with the conversion to fine bubble diffused aeration are presented in Table 10-1.

Table 10-1

Conversion to Fine Bubble Aeration Cost Estimate

	Estimated Cost
Four (4) High Speed Blowers (Installed)	$1,200,000
Fine Bubble Diffusers (Installed)	$800,000
Piping and Miscellaneous Mechanical	$200,000
Modifications/Demolition to Existing Blower Building	$100,000
Vertical Mixers	$1,000,000
Electrical	$100,000
Subtotal	**$3,400,000**
Engineering and Administration	$680,000
Total	**$4,080,000**

New blowers are recommended to replace the existing blowers since the existing blowers do not develop sufficient operating pressure for a fine bubble aeration system. Replacing the jet mixing equipment with vertical mixers is also recommended to reduce maintenance and accessibility issues with the existing submerged pumping equipment.

Supplemental Carbon Storage and Feed Facility Improvements

The Mason Farm WWTP will have to meet more stringent total nitrogen limits effective in 2016 to comply with the Jordan Lake Rules. OWASA constructed denitrification filters in the last major plant upgrade to address future total nitrogen limits. These filters have the ability to utilize acetic acid as a carbon source to promote biological denitrification in the filters. This study compared using acetic acid as a carbon source with other carbon sources such as methanol. Using methanol in lieu of acetic acid as the carbon source for denitrification in the existing filters will resurlt in substantial operational savings as discussed in the evaluation of alternative supplemental carbon products (Section 6.12). The costs of a new methanol feed facility is estimated to be offset by operational savings within 4 years. Methanol feed to denitrification filters is the most proven carbon source for denitrification filters. It is also noted that methanol is a flammable chemical and has increased process safety risks. OWASA has demonstrated that acetic acid is capable of producing sufficient total nitrogen removal to meet the regulatory requirements so the

timing for making any change in supplemental carbon chemical type should be based on potential for operational cost savings. Recent developments in the industry suggest that glycerin based carbon products may have potential as a carbon source for denitrification filtration in lieu of methanol and provide much of the potential cost savings without the safety risks. Since OWASA will not need to feed supplemental carbon until 2016, it is recommended that the potential to utilize an alternate carbon source to the current acetic acid feed system be reevaluated in a few years, giving the market time to advance the development of glycerin based carbon products in dentrification filters. Table 10-2 presents the estimated costs for a methanol storage and feed facility for OWASA's CIP budget purposes. Potentially, some of the existing acetic acid storage tanks could be used to store glycerin and other supplemental carbon sources, which would significantly result the costs of implementing an alternate supplemental carbon source at the Mason Farm WWTP.

Table 10-2

Methanol Facility Cost Estimate

	Estimated Cost
Structural	$200,000
Storage Tanks	$400,000
Equipment	$230,000
Subtotal	**$830,000**
Engineering and Administration (20%)	$170,000
Total	**$1,000,000**

Nutrient Process Monitoring and Control Improvements

Nutrient analyzers are recommended at the Mason Farm WWTP in order to provide a greater level of monitoring and control of the process to increase process reliability while minimizing operational costs. Specifically, an orthophosphate analyzer, an ammonium analyzers, a pH probe and a nitrate analyzer would be installed in the effluent channel of the aeration basins as discussed in Appendix P. Provisions should also be made to allow the existing analyzer downstream of the effluent filters to measure filter influent (prior to the proposed chemical addition point) and effluent phosphorus concentrations.

The orthophosphate analyzer will be used to automate dosing of alum to the mixed liquor in the aeration basins. The pH probe should be integrated into the plant automation system to control caustic feed to the aeration basins. The existing analyzer downstream of the effluent filters should also be used to control chemical feed to the filters; this ability should be integrated into the plant automation system.

The estimated costs associated with installation of the proposed instrumentation is $77,500.

Solids Treatment Evaluations

The Mason Farm WWTP currently uses anaerobic digestion for sludge stabilization to meet Class A sludge stabilization criteria. Section 7.0 addresses solids treatment evaluations and recommendations in detail. Improvements to the anaerobic digestion process will be required to ensure adequate stabilization as flows and loads increase. It is recommended that a macerator be installed on the hot side feed to the existing sludge-to-sludge heat exchanger to reduce plugging and clogging associated with particles greater than 0.25 inches. A VFD should also be installed for the sludge transfer pump (TP-2-1) to allow flow pacing and continuous operation to match the continuous heating demand of the sludge pre-heating system. Also, a three-way valve will be installed to allow heated sludge leaving the heat exchanger to be directed to Digester 1 or Digester 2 depending on transfer needs. Table 10-3 summarizes the costs associated with recommended solids process improvements at approximately $400,000. If the heat exchanger continues to experience a degree of clogging, OWASA should replace the spiral heat exchanger with a tube-in-tube style heat exchanger, which is estimated at a cost of $100,000.

Table 10-3

Recommended Solids Improvements Cost Estimate

	Estimated Cost
Anaerobic Digester HX Improvements	
Equipment	$180,000
Piping and Valving	$43,000
Electrical	$110,000
Subtotal	**$333,000**
Engineering and Administration (20%)	$67,000
Total Cost	**$400,000**

Recycle Stream Treatment Evaluations

Recycle streams from the solids dewatering process have a significant impact on the total waste loads to the secondary treatment process, especially the nutrient loadings that must be treated to meet the requirements of the Jordan Lake Rules. Alternatives for addressing these impacts are addressed in Section 8.0. Recycle stream treatment should be considered if OWASA elects to process more than half of its digested sludge through dewatering. Based on current operational practices, it is anticipated that OWASA will continue to land apply liquid biosolids to the extent possible and capital investment in dedicated treatment of recycle streams would not be cost effective. If the operating practice significantly increases the percentage of the sludge that is dewatered prior to disposal in the future, several dedicated recycle stream treatment approaches are viable. Flow equalization of filtrate streams can provide for low capital cost attenuation of TN and TP returned to the liquid stream.

Biological sidestream treatment for nitrogen removal should be considered when the plant is expanded to 18.5 mgd, particularly if more stringent nutrient standards are applied at that time. Nitritation/denitritation processes may provide for substantial aeration and chemical savings and should also be evaluated during preliminary design of the future expansion. Table 10-4 presents estimated costs for improvements associated with recycle stream equalization that may be needed if OWASA modifies its current practice and pursues dewatering 100% of digested solids generated at the

Mason Farm WWTP. These improvements are only required if OWASA shifts the biosolids disposal program to favor dewatering a majority of the sludge.

Table 10-4

Recycle Stream Treatment Options Cost Estimate

	Estimated Cost
Filtrate Flow Equalization	
Structural	$200,000
Equipment	$190,000
Electrical	$30,000
Subtotal	**$420,000**
Engineering and Administration (20%)	$80,000
Total	**$500,000**

Odor Control

OWASA has previously developed plans for capital improvements to cover and odor scrub the NSL basins and four of the aeration cells as Phase 3 Odor Control Improvements to reduce odor emissions from the Mason Farm WWTP. Because the process transition to step feed would potentially impact source and intensity of odors emitted from the plant, additional odor modeling and analysis was conducted as described in Appendix O and summarized in Section 6.7 to determine whether changes should be made to the Phase 3 Odor Control Improvements. The findings of the additional modeling indicate that the NSL basins should be covered as previously planned and that six aeration cells should be covered instead of the four cells previously planned. The proposed odor control system improvements would consist of one 6,700 cfm carbon adsorption unit drawing air from the NSL basins and two 15,000 cfm carbon adsorption units drawing air from the aeration basins. It is anticipated that these improvements would be constructed in conjunction with aeration and mixing improvements. The mixing improvements will include mixer/walkway support beams that will also support the cover equipment, eliminating the need for concrete columns within the tanks to support the covers. The estimated cost of the odor control system is approximately $2.7 million including engineering, which is an increase of approximately

$500,000 over the facilities recommended in the Brown and Caldwell design per their 95% Design cost estimate dated 12/21/2009. If OWASA decides to reduce the D/T at the property boundaries gradually over a period of time, the Phase 3 improvements could be implemented in multiple phases with the higher priority improvements being constructed first.

Secondary Capacity Expansion Evaluations

Expansion of the Mason Farm WWTP to 18.5 mgd is anticipated around 2030 based on current wastewater strength and predicted growth. Future expansion considerations are discussed in Section 9.0. Expansion will be required to limit solids loading to the secondary clarifiers when operating in the step feed configuration. Providing internal or external anoxic zones with nitrified recycle return should be considered during expansion of the facility to 18.5 mgd. Providing for an external anoxic zone will reduce the amount of additional aeration basin volume that would need to be constructed for the expansion. In addition, since the Mason Farm WWTP has limited site area for future expansion, incorporating IFAS, MBR or converting the NSL cells to aeration volume may eliminate the need for constructing additional nitrification volume at 18.5 mgd. Future regulatory drivers including stricter nutrient regulations, reclaimed water requirements and removal of emerging contaminants will dictate the technologies to be considered and ultimately selected at future flows.

Capital Costs and Schedule of Implementation

The fiscal year in which each capital or operational optimization recommendation is required is identified below. Costs are based on January 2010 dollars (Engineering News Record (ENR) Construction Cost Index of 8,660). Note that the cost or need for some of the proposed improvements is dependent on future regulatory drivers or changes in plant operations. The estimated capital costs of recommended improvements through the next ten years is expected to be approximately $8.3 million dollars and are expected to result in substantial operational cost savings.

Fiscal Year 2010

Recommended improvements for FY2010 include:

- Jar Testing and Optimization Operational Assistance
    - Cost - $30,000
    - Status – Complete
    - Discussed in Section 8.7
- Rotary Press Stress Testing
    - Cost - $0
    - Status – On-Going
    - Discussed in Section 7.7

The total capital cost of all recommended improvements or modifications for FY 2010 is $30,000.

Fiscal Year 2011

Recommended improvements for FY2011 include:

- Provide Chemical Phosphorus Feed Point at Filters
    - Cost - $30,000
    - Status – Planned (to be designed by Hazen & Sawyer)
    - Discussed in Section 6.11
- Install Nutrient Analyzers and Integrate with Plant Control System
    - Cost - $77,500
    - Status – Planned (to be designed by Hazen & Sawyer)
    - Discussed in Appendix P
- Construct Anaerobic Digester Heat Exchanger Improvements
    - Cost - $400,000
    - Status – Planned (to be designed by Hazen & Sawyer)
    - Discussed in Section 7.5

The total capital cost of all recommended improvements or modifications for FY 2011 is $507,500.

Fiscal Year 2012

Recommended improvements for FY2012 include:

- Modify Existing Aeration System
    - Cost - $4,080,000

    - Discussed in Section 6.3
- Modify Odor Control Design to Accommodate Step Feed
    - Cost - $2,700,000
    - Discussed in Appendix O

The total capital cost of all recommended improvements or modifications for FY 2012 is $6,780,000.

Fiscal Year 2018

Recommended improvements for FY2018 include:

- Construct Supplemental Carbon Storage and Feed Facilities
    - Cost - $1,000,000
    - Discussed in Section 6.12

Fiscal Year 2030 or when necessary based on flow development

Recommended improvements for FY2030 (or when necessary) include:

- Construct Filtrate Flow Equalization
    - Cost - $500,000
    - Discussed in Section 8.2
    - Necessary when dewatering majority of sludge
- Expand Secondary Treatment Process to 18.5 mgd
    - Cost - $60 million (conventional technology)
    - Discussed in Section 9.0
- Construct Filtrate Nitritation/Denitritation Facilities
    - Cost – Undetermined
    - Discussed in Section 8.4
- Construct Filtrate Phosphorus Recovery Facilities
    - Cost – Undetermined
    - Discussed in Section 8.4
    - Necessary when dewatering majority of sludge

OWASA Mason Farm WWTP
Flows and Flow Peaking Factors

Flow Criteria	Historical[1]	
	Flow (MGD)	Peaking Factor
Minimum Day	5.0	0.66
Average Annual	7.7	1.00
Maximum Month	8.9	1.16
Maximum 30-Day	9.0	1.18
Maximum 7-Day	10.1	1.33
Maximum Day	15.7	2.06

[1] Historical data based on plant operations data, Jan-04 to Dec-07

OWASA Mason Farm WWTP
Annual Average[2] Influent Concentrations

Criteria	Historical[1]	Calculated[3]
	AA Concentration	AA Concentration
BOD, mg/L	217.8	218.5
CBOD, mg/L	217.8	218.5
TSS, mg/L	325.1	328.1
VSS, mg/L	293.2	296.1
TKN, mg/L	42.5	41.5
NH3, mg/L	25.5	25.7
TP, mg/L	7.7	7.8

[1] Historical data based on plant operations data, Jan-04 to Dec-07

[2] Annual Average data based on data within 2

[3] Calculated from historical average annual flow

OWASA Mason Farm WWTP
Historical Average Influent Ratios, Sorted

Ratio	Value
Influent VSS/TSS	0.89
Influent NH3-N/TKN	0.63
Influent BOD/TKN	5.21
Influent BOD/TP	29.06
Influent BOD/NH3	8.72

OWASA Mason Farm WWTP
Summary of Influent Temperature

Criteria	Historical[1,2] Temperature, C
Minimum Day	13.2
Minimum 7-Day	14.6
Average Annual	20.4
Maximum 7-Day	26.8

[1] Historical data based on plant operations data, Jan-04 to Dec-07

[2] Historical temperature data NOT sorted.

OWASA Mason Farm WWTP
Historical Load Peaking Factors, Sorted Data

Criteria	BOD_5	CBOD	TSS	VSS	TKN	NH_3	TP
Minimum Day	0.48	0.48	0.27	0.28	0.66	0.59	0.47
Average Annual	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Maximum Month	1.38	1.38	1.50	1.48	1.85	1.31	1.54
Maximum 30-Day	1.43	1.43	1.64	1.61	No Data	1.34	1.60
Maximum 7-Day	1.67	1.67	2.13	2.12	1.29	1.48	2.04
Maximum Day	1.93	1.93	3.44	3.36	1.85	1.70	3.26

OWASA Mason Farm WWTP
Historical Loads, Sorted Data

Criteria	BOD_5	CBOD	TSS	VSS	TKN	NH_3	TP
	ppd	ppd	ppd	ppd	ppd	ppd	ppd
Minimum Day	6,700	6,700	5,900	5,500	1,520	960	260
Average Annual	14,100	14,100	21,200	19,100	2,680	1,660	510
Maximum Month	19,600	19,600	32,100	28,900	4,890	2,190	790
Maximum 30-Day	20,300	20,300	33,900	30,500	No Data	2,270	820
Maximum 7-Day	23,300	23,300	42,800	38,600	3,600	2,440	950
Maximum Day	27,500	27,500	65,700	57,300	4,890	2,740	1,510

Statistical Data Analysis of Sorted Data (Influent Concentrations, mg/L)

	BOD_5	CBOD	TSS	VSS	TKN	NH_3	TP
Original Data Set							
Number of Data	996	996	995	986	77	994	988
Average (Mean)	223.7	223.7	343.4	308.0	42.8	26.1	8.3
Standard Deviation	54.0	54.0	129.3	125.4	8.9	5.3	3.0
Median	219.0	219.0	310.0	280.0	42.3	25.5	7.5
Maximum	559.0	559.0	1004.0	1606.0	68.0	54.8	25.1
Maximum Date	1/17/06	1/17/06	8/14/07	8/9/07	1/9/06	11/27/07	1/28/07
Minimum	75.0	75.0	88.0	87.0	19.8	13.2	2.5
Minimum Date	3/17/05	3/17/05	11/22/05	11/22/05	2/5/06	3/17/05	3/17/05
99^{th} Percentile	378.0	378.0	822.1	741.2	63.7	43.3	19.6
95^{th} Percentile	329.3	329.3	588.3	511.5	57.4	36.1	14.4
90^{th} Percentile	291.3	291.3	520.0	457.0	54.0	32.7	11.6
Regular Data Screening Method							
Mean + 2*SD	331.7	331.7	602.0	558.7	60.6	36.7	14.2
Percentile	95.30%	95.30%	95.50%	96.70%	96.80%	95.40%	94.80%
Count > Mean + 2*SD	47	47	45	33	3	46	52
Mean + 3*SD	385.7	385.7	731.4	684.1	69.5	42.0	17.2
Percentile	99.50%	99.50%	98.20%	98.40%	100.0%	98.60%	97.00%
Count > Mean + 3*SD	5	5	18	16	0	13	30
Mean - 2*SD	115.7	115.7	84.7	57.3	25.0	15.5	2.4
Percentile	0.80%	0.80%	0.00%	0.00%	1.80%	0.60%	0.00%
Count < Mean - 2*SD	9	9	0	0	2	7	0
Mean - 3*SD	61.7	61.7	-44.6	-68.1	16.1	10.2	-0.6
Percentile	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Count < Mean - 3*SD	0	0	0	0	0	0	0
2 SD Screened Data Set							
Number of Data	940	940	950	953	72	941	936
Average (Mean)	217.8	217.8	325.1	293.2	42.5	25.5	7.7
Standard Deviation	42.9	42.9	97.7	89.9	7.2	4.2	1.8
Median	217.6	217.6	305.0	277.0	41.7	25.3	7.4
Maximum	331.0	331.0	596.0	550.0	58.0	36.5	14.0
Maximum Date	2/26/06	2/26/06	3/7/07	2/28/07	8/14/07	12/27/05	6/12/05
Minimum	119.9	119.9	88.0	87.0	29.6	16.0	2.5
Minimum Date	12/30/07	12/30/07	11/22/05	11/22/05	7/10/05	8/30/04	3/17/05
99^{th} Percentile	318.5	318.5	576.1	517.0	57.5	35.6	13.2
95^{th} Percentile	295.0	295.0	528.0	475.0	55.6	33.2	11.6
90^{th} Percentile	276.0	276.0	470.0	427.8	52.8	31.3	10.1
3 SD Screened Data Set							
Number of Data	991	991	977	970	77	981	958
Average (Mean)	222.6	222.6	334.1	298.8	42.8	25.9	7.9
Standard Deviation	51.4	51.4	110.4	98.6	8.9	4.8	2.2
Median	218.9	218.9	308.0	279.5	42.3	25.5	7.4
Maximum	380.0	380.0	730.0	677.0	68.0	42.0	17.0
Maximum Date	2/28/07	2/28/07	2/25/07	4/28/05	1/9/06	11/20/07	1/10/07
Minimum	75.0	75.0	88.0	87.0	19.8	13.2	2.5
Minimum Date	3/17/05	3/17/05	11/22/05	11/22/05	2/5/06	3/17/05	3/17/05
99^{th} Percentile	365.5	365.5	650.7	592.2	63.7	39.3	15.8
95^{th} Percentile	320.5	320.5	559.2	495.1	57.4	35.1	12.3
90^{th} Percentile	290.0	290.0	498.0	448.1	54.0	32.2	10.6

Summary of Historical Influent Concentrations, Loads and Load Peaking Factors

Year 1

January-04 to December-04

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	6.06	0.75	112.80	8066.45	0.57	171.00	10916.35	0.46	146.00	10254.62	0.48	32.20	1882.52	0.66	14.50	1054.28	0.64	4.44	293	0.55	14
Average Annual[1]	8.06	1.00	210.68	14201.89	1.00	353.37	23950.83	1.00	317.37	21541.52	1.00	41.19	2863.87	1.00	24.48	1657.19	1.00	7.84	531	1.00	20.11
Maximum Month[1]	8.74	1.08	285.10	19555.38	1.38	424.90	28941.01	1.21	374.95	25670.51	1.19	56.70	4889.81	1.71	28.50	1922.21	1.16	9.11	624	1.18	24.82
Max 30-Day[2]	8.99	1.12	297.56	20258.86	1.43	470.63	29422.38	1.23	422.74	26433.68	1.23	No Data	No Data	No Data	28.81	1986.16	1.20	9.14	631	1.19	No Data
Max 7-Day[2]	9.73	1.21	324.60	21892.37	1.54	591.00	42760.59	1.79	532.50	38528.73	1.79	No Data	No Data	No Data	31.38	2154.47	1.30	10.12	701	1.32	25.21
Maximum Day[2]	14.14	1.76	378.00	27425.51	1.93	725.00	55703.63	2.33	672.00	49612.24	2.30	56.70	4889.81	1.71	36.20	2459.95	1.48	13.29	919	1.73	25.80
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Year 2

January-05 to December-05

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	6.00	0.78	75.00	7308.54	0.54	88.00	6167.86	0.32	87.00	6097.77	0.36	29.60	1679.91	0.74	13.20	996.41	0.60	2.53	254.91	0.54	13.20
Average Annual[1]	7.65	1.00	211.05	13468.24	1.00	297.76	19046.28	1.00	266.95	17047.29	1.00	36.08	2255.19	1.00	25.77	1647.03	1.00	7.36	472.00	1.00	20.12
Maximum Month[1]	8.78	1.15	254.28	16183.17	1.20	411.89	27235.76	1.43	365.11	24150.02	1.42	62.40	3717.85	1.65	31.11	2020.28	1.23	9.62	622.03	1.32	25.70
Max 30-Day[2]	8.88	1.16	264.25	16625.58	1.23	420.47	27846.48	1.46	372.53	24679.85	1.45	No Data	No Data	No Data	31.24	2031.78	1.23	9.89	632.46	1.34	No Data
Max 7-Day[2]	9.73	1.27	299.75	19164.36	1.42	578.40	40497.99	2.13	517.60	36170.67	2.12	No Data	No Data	No Data	38.47	2173.83	1.32	13.02	740.49	1.57	25.55
Maximum Day[2]	15.51	2.03	378.70	25753.75	1.91	698.00	65606.28	3.44	677.00	57241.01	3.36	62.40	3717.85	1.65	39.50	2437.62	1.48	16.43	1059.56	2.24	26.00
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Year 3

January-06 to December-06

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	6.16	0.81	130.90	7934.52	0.54	124.00	9476.33	0.47	98.00	8342.39	0.46	19.80	1513.27	0.68	16.50	957.90	0.59	4.39	293.43	0.63	15.00
Average Annual[1]	7.61	1.00	230.88	14735.93	1.00	314.81	20153.54	1.00	285.33	18293.64	1.00	36.97	2241.57	1.00	25.28	1614.79	1.00	7.27	462.93	1.00	20.48
Maximum Month[1]	8.86	1.16	275.80	18180.69	1.23	460.84	30214.58	1.50	407.26	26663.42	1.46	68.00	4150.50	1.85	32.44	2122.45	1.31	11.11	713.49	1.54	26.07
Max 30-Day[2]	8.96	1.18	285.46	18526.47	1.26	516.94	33109.21	1.64	459.44	29452.70	1.61	No Data	No Data	No Data	34.53	2165.33	1.34	12.16	741.04	1.60	No Data
Max 7-Day[2]	10.11	1.33	317.43	21528.47	1.46	596.00	38654.57	1.92	523.33	33953.34	1.86	No Data	No Data	No Data	38.47	2392.63	1.48	15.00	946.15	2.04	25.00
Maximum Day[2]	15.68	2.06	378.30	24623.25	1.67	722.00	52675.91	2.61	670.00	46222.28	2.53	68.00	4150.50	1.85	40.60	2739.49	1.70	16.51	1509.53	3.26	26.26
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Year 4

January-07 to December-07

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	5.03	0.66	113.20	6669.98	0.48	99.00	5833.29	0.27	93.00	5479.76	0.28	22.20	1850.52	0.66	13.30	1061.53	0.63	3.33	256.34	0.47	14.60
Average Annual[1]	7.56	1.00	218.39	13855.29	1.00	335.18	21393.88	1.00	304.00	19450.59	1.00	45.09	2794.52	1.00	26.61	1694.67	1.00	8.55	541.23	1.00	20.76
Maximum Month[1]	8.59	1.14	262.41	18306.50	1.32	458.26	32075.73	1.50	410.68	28804.58	1.48	49.92	3415.16	1.22	34.17	2181.42	1.29	11.09	785.46	1.45	26.50
Max 30-Day[2]	8.64	1.14	278.22	19741.72	1.42	484.47	33850.04	1.58	427.58	30490.73	1.57	No Data	No Data	No Data	34.91	2261.97	1.33	11.57	812.47	1.50	No Data
Max 7-Day[2]	10.06	1.33	346.00	23201.09	1.67	585.20	41869.36	1.96	538.00	36289.48	1.87	52.95	3594.12	1.29	38.13	2433.29	1.44	13.76	927.72	1.71	26.75
Maximum Day[2]	13.45	1.78	380.00	26197.82	1.89	730.00	53223.04	2.49	618.00	45057.32	2.32	58.00	3739.49	1.34	42.00	2683.97	1.58	16.99	1332.51	2.46	27.00
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Summary of Historical Influent Concentrations, Loads and Load Peaking Factors

Years 1 through 5 - Maximum
Maximum from Above Annual Data

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	6.16	0.81	130.90	8066.45	0.57	171.00	10916.35	0.47	146.00	10254.62	0.48	32.20	1882.52	0.74	16.50	1061.53	0.64	4.44	293.43	0.63	15.00
Average Annual[4]	8.06	1.00	230.88	14735.93	1.00	353.37	23950.83	1.00	317.37	21541.52	1.00	45.09	2863.87	1.00	26.61	1694.67	1.00	8.55	541.23	1.00	20.76
Maximum Month[1]	8.86	1.16	285.10	19555.38	1.38	460.84	32075.73	1.50	410.68	28804.58	1.48	68.00	4889.81	1.85	34.17	2181.42	1.31	11.11	785.46	1.54	26.50
Max 30-Day[2]	8.99	1.18	297.56	20258.86	1.43	516.94	33850.04	1.64	459.44	30490.73	1.61	No Data	No Data	No Data	34.91	2261.97	1.34	12.16	812.47	1.60	No Data
Max 7-Day[2]	10.11	1.33	346.00	23201.09	1.67	596.00	42760.59	2.13	538.00	38528.73	2.12	52.95	3594.12	1.29	38.47	2433.29	1.48	15.00	946.15	2.04	26.75
Maximum Day[2]	15.68	2.06	380.00	27425.51	1.93	730.00	65606.28	3.44	677.00	57241.01	3.36	68.00	4889.81	1.85	42.00	2739.49	1.70	16.99	1509.53	3.26	27.00
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Years 1 through 5 - Average
Averages from Above Annual Data

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	5.81	0.75	107.98	7494.87	0.53	120.50	8098.46	0.38	106.00	7543.63	0.39	25.95	1731.55	0.68	14.38	1017.53	0.62	3.67	274.50	0.55	14.14
Average Annual[1,3]	7.72	1.00	217.83	14,066	1.00	325.08	21,125	1.00	293.17	19,063	1.00	42.53	2,675	1.00	25.50	1,652	1.00	7.74	501	1.00	20.37
Maximum Month[1]	8.74	1.13	269.40	18056.43	1.28	438.97	29616.77	1.41	389.50	26322.13	1.39	59.26	4043.33	1.61	31.55	2061.59	1.25	10.23	686.29	1.37	25.77
Max 30-Day[2]	8.87	1.15	281.37	18788.16	1.34	473.13	31057.03	1.48	420.57	27764.24	1.46	No Data	No Data	No Data	32.37	2111.31	1.28	10.69	704.17	1.41	No Data
Max 7-Day[2]	9.91	1.28	321.95	21446.57	1.52	587.65	40945.62	1.95	527.86	36235.55	1.91	52.95	3594.12	1.29	36.61	2288.55	1.38	12.98	828.95	1.66	25.63
Maximum Day[2]	14.70	1.91	378.75	26000.08	1.85	718.75	56802.21	2.72	659.25	49533.21	2.63	61.28	4124.41	1.64	39.58	2580.26	1.56	15.81	1205.13	2.42	26.27
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Years 1 through 5 - Minimum
Minimum from Above Annual Data

	Influent Flow		Influent CBOD			Influent TSS			Influent VSS			Influent TKN			Influent Ammonia			Influent Total Phosphorus			Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Temperature Deg C
Minimum Day[2]	5.03	0.66	75.00	6669.98	0.48	88.00	5833.29	0.27	87.00	5479.76	0.28	19.80	1513.27	0.66	13.20	957.90	0.59	2.53	254.91	0.47	13.20
Average Annual[4]	7.56	1.00	210.68	13468.24	1.00	297.76	19046.28	1.00	266.95	17047.29	1.00	36.08	2241.57	1.00	24.48	1614.79	1.00	7.27	462.93	1.00	20.11
Maximum Month[1]	8.59	1.08	254.28	16183.17	1.20	411.89	27235.76	1.21	365.11	24150.02	1.19	49.92	3415.16	1.22	28.50	1922.21	1.16	9.11	622.03	1.18	24.82
Max 30-Day[2]	8.64	1.12	264.25	16625.58	1.23	420.47	27846.48	1.23	372.53	24679.85	1.23	No Data	No Data	No Data	28.81	1986.16	1.20	9.14	630.71	1.19	No Data
Max 7-Day[2]	9.73	1.21	299.75	19164.36	1.42	578.40	38654.57	1.79	517.60	33953.34	1.79	52.95	3594.12	1.29	31.38	2154.47	1.30	10.12	701.43	1.32	25.00
Maximum Day[2]	13.45	1.76	378.00	24623.25	1.67	698.00	52675.91	2.33	618.00	45057.32	2.30	56.70	3717.85	1.34	36.20	2437.62	1.48	13.29	918.93	1.73	25.80
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

[1]Calculated using data within 2 standard deviations of the mean.
[2]Calculated using data within 3 standard deviations of the mean.
[3]Values shown are the average of all data points.
[4]Minimum Day from Data set.
[5]Maximum from Data Set.
[7]Values shown are actual concentration numbers pulled from the sorted data set. Not to be used for design purposes.
[8]Values shown are calculated from the corresponding load divided by 8.34 times the average annual flow.

Monthly Average Flow, Concentrations and Loadings[1]

	Influent Flow	Influent BOD		Influent CBOD		Influent TSS		Influent VSS		Influent TKN		Influent Ammonia		Influent Total Phosphorus		Influent Temp.
Month	MGD	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Temp C
Jan-04	8.17	253.38	17,521	253.38	17,521	422.25	28,941	374.70	25,671	No Data	No Data	28.07	1,922	9.11	624	15.93
Feb-04	8.43	234.95	16,118	234.95	16,118	327.58	22,796	303.55	20,990	40.90	2,945	27.31	1,886	8.33	574	14.35
Mar-04	7.90	203.49	13,421	203.49	13,421	344.13	22,970	302.06	20,702	No Data	No Data	28.50	1,900	8.74	581	15.67
Apr-04	7.83	208.59	13,746	208.59	13,746	377.15	24,949	341.10	22,565	43.10	2,321	25.31	1,670	7.85	519	17.84
May-04	7.80	223.08	14,939	223.08	14,939	400.84	27,159	352.47	23,753	51.30	4,890	22.47	1,492	8.38	556	20.40
Jun-04	7.37	217.34	13,339	217.34	13,339	378.64	23,302	343.68	21,153	32.20	1,883	21.55	1,324	7.30	449	23.13
Jul-04	7.45	205.51	12,961	205.51	12,961	367.80	23,304	335.45	21,221	33.70	2,043	20.07	1,267	6.49	410	24.67
Aug-04	8.23	207.71	14,276	207.71	14,276	341.55	23,458	296.30	20,344	42.60	2,613	21.36	1,462	7.40	507	24.82
Sep-04	8.74	177.84	12,845	177.84	12,845	317.71	23,755	285.71	21,369	37.70	2,726	22.85	1,650	6.94	513	24.66
Oct-04	8.29	213.73	15,022	213.73	15,022	338.50	23,655	315.18	22,018	34.30	2,373	24.51	1,720	8.15	575	23.07
Nov-04	8.55	185.78	13,360	185.78	13,360	296.00	21,431	267.35	19,354	39.40	2,907	26.59	1,916	7.16	518	20.12
Dec-04	7.95	212.63	14,270	212.63	14,270	324.06	21,734	286.67	19,279	56.70	3,938	25.53	1,717	8.27	547	17.03
Jan-05	7.86	197.58	12,549	197.58	12,549	256.56	16,253	227.06	14,362	36.50	1,986	25.19	1,597	7.81	497	15.22
Feb-05	8.25	186.58	12,646	186.58	12,646	277.85	19,180	241.80	16,689	No Data	No Data	25.28	1,748	7.84	542	15.00
Mar-05	8.78	185.29	13,191	185.29	13,191	269.91	20,118	241.36	17,933	No Data	No Data	23.54	1,699	5.52	407	15.33
Apr-05	8.17	201.19	13,759	201.19	13,759	265.21	18,082	232.11	15,786	No Data	No Data	25.76	1,741	7.08	481	17.51
May-05	7.15	221.59	13,290	221.59	13,290	334.64	20,066	296.32	17,773	43.40	2,760	28.23	1,697	8.33	500	20.02
Jun-05	7.02	204.70	12,091	204.70	12,091	316.67	18,672	289.41	17,070	33.00	2,017	26.30	1,553	8.41	497	22.35
Jul-05	7.16	175.52	10,639	175.52	10,639	270.95	16,528	236.60	14,403	29.60	1,680	21.28	1,290	6.76	411	24.47
Aug-05	7.52	203.82	12,960	203.82	12,960	267.04	17,078	242.61	15,477	34.60	1,914	23.28	1,460	6.10	389	25.70
Sep-05	7.51	242.44	15,538	242.44	15,538	316.25	20,207	291.05	18,591	38.70	2,392	25.76	1,647	6.95	443	25.60
Oct-05	7.34	249.06	15,222	249.06	15,222	348.14	21,224	314.27	19,161	37.40	2,312	28.35	1,727	7.89	482	23.38
Nov-05	7.39	233.94	14,796	233.94	14,796	278.15	17,650	259.70	16,439	No Data	No Data	26.67	1,696	6.95	441	20.40
Dec-05	7.69	232.82	15,504	232.82	15,504	380.19	24,062	335.38	21,254	35.40	2,981	30.06	1,999	9.34	613	16.58
Jan-06	7.71	259.67	17,077	259.67	17,077	426.31	27,580	398.50	26,115	No Data	No Data	30.06	1,970	9.85	642	15.64
Feb-06	7.95	257.69	17,120	257.69	17,120	317.61	20,824	290.83	19,064	No Data	No Data	27.00	1,778	7.63	499	15.50
Mar-06	7.36	259.20	16,108	259.20	16,108	289.64	18,005	265.05	16,479	38.80	2,269	27.42	1,706	7.27	453	16.75
Apr-06	7.66	252.54	16,378	252.54	16,378	277.95	18,064	251.05	16,315	42.60	2,682	25.67	1,661	6.62	428	18.83
May-06	7.02	239.89	14,226	239.89	14,226	313.68	18,666	285.32	17,040	33.80	2,156	24.58	1,465	6.79	404	20.54
Jun-06	7.23	226.88	13,848	226.88	13,848	270.10	16,352	238.29	14,421	35.20	2,093	21.98	1,339	6.31	385	22.85
Jul-06	7.17	207.17	12,531	207.17	12,531	308.10	19,134	276.29	17,058	35.90	2,063	21.53	1,300	6.49	396	25.01
Aug-06	7.25	185.56	11,453	185.56	11,453	237.26	14,572	212.22	13,013	31.70	1,720	22.17	1,368	6.10	375	26.07
Sep-06	7.86	196.25	13,114	196.25	13,114	266.74	17,812	234.47	15,631	35.40	2,179	24.58	1,631	6.39	424	24.78
Oct-06	7.63	221.71	14,109	221.71	14,109	348.70	22,384	306.74	19,601	33.30	2,257	26.39	1,677	7.59	484	22.77
Nov-06	8.86	235.51	17,023	235.51	17,023	384.00	28,455	352.33	25,907	46.00	2,756	26.06	1,884	8.98	649	19.24
Dec-06	7.67	246.41	15,503	246.41	15,503	401.73	24,991	360.20	22,353	No Data	No Data	28.29	1,796	9.71	600	17.25

Monthly Average Flow, Concentrations and Loadings[1]

	Influent Flow	Influent BOD		Influent CBOD		Influent TSS		Influent VSS		Influent TKN		Influent Ammonia		Influent Total Phosphorus		Influent Temp.
Month	MGD	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Temp C
Jan-07	8.59	229.16	16,227	229.16	16,227	418.60	30,086	382.95	27,159	48.60	3,346	28.31	2,039	9.37	677	16.00
Feb-07	8.25	240.41	16,764	240.41	16,764	419.31	29,342	388.88	27,243	36.60	2,389	29.88	2,071	10.27	714	14.83
Mar-07	8.21	231.72	15,176	231.72	15,176	409.37	26,651	366.57	24,159	49.92	3,415	30.71	2,121	10.83	737	16.15
Apr-07	8.12	211.21	14,654	211.21	14,654	300.29	20,612	267.81	18,409	43.08	2,742	23.68	1,574	7.18	489	18.46
May-07	7.25	212.74	12,944	212.74	12,944	345.27	21,116	318.55	19,479	41.13	2,401	23.50	1,425	9.08	555	20.73
Jun-07	7.04	226.27	13,441	226.27	13,441	257.25	15,235	236.10	13,965	40.50	2,369	21.48	1,272	6.67	395	22.93
Jul-07	6.88	214.47	12,363	214.47	12,363	327.36	18,984	287.00	16,659	46.22	2,707	23.67	1,367	8.16	472	24.90
Aug-07	7.60	230.52	14,867	230.52	14,867	312.85	20,379	288.92	18,813	45.95	2,922	24.19	1,548	9.02	573	26.50
Sep-07	7.50	212.19	13,297	212.19	13,297	338.05	21,256	307.70	19,345	43.15	2,672	28.91	1,814	8.64	541	26.15
Oct-07	7.55	213.59	13,258	213.59	13,258	347.22	21,549	319.30	19,804	46.92	2,906	30.90	1,943	8.96	554	24.26
Nov-07	6.91	223.55	13,235	223.55	13,235	333.33	19,599	274.44	16,120	48.68	2,893	30.38	1,762	8.55	505	20.89
Dec-07	6.90	179.40	10,572	179.40	10,572	213.33	12,647	191.72	11,320	44.65	2,722	30.18	1,798	7.25	429	17.86

[1]Based on data within 2 SD of mean.

OWASA Mason Farm WWTP
Flows and Flow Peaking Factors

Flow Criteria	Historical[1]	
	Flow (MGD)	Peaking Factor
Minimum Day	5.7	0.75
Average Annual	7.8	1.00
Maximum Month	9.9	1.25
Maximum 30-Day	9.9	1.25
Maximum 7-Day	11.5	1.51
Maximum Day	22.5	2.95

[1] Historical data based on plant operations data, Jan-08 to Sep-09

OWASA Mason Farm WWTP

Criteria	Historical[1]	Calculated[3]
	AA Concentration	AA Concentration
BOD, mg/L	196.3	195.2
CBOD, mg/L	196.3	195.2
TSS, mg/L	266.5	266.3
VSS, mg/L	242.3	241.9
TKN, mg/L	38.3	38.1
NH3, mg/L	28.2	28.0
TP, mg/L	7.1	7.0

[1] Historical data based on plant operations data, Jan-08 to Sep-09

[2] Annual Average data based on data within 2 standard deviations of the mean.

[3] Calculated from historical average annual flow and average annual load.

OWASA Mason Farm WWTP
Historical Average Influent Ratios, Sorted

Ratio	Value
Influent VSS/TSS	0.91
Influent NH3-N/TKN	0.74
Influent BOD/TKN	5.04
Influent BOD/TP	28.25
Influent BOD/NH3	7.09

OWASA Mason Farm WWTP
Summary of Influent Temperature

Criteria	Historical[1,2] Temperature, C
Minimum Day	14.6
Minimum 7-Day	15.5
Average Annual	20.2
Maximum 30-Day	No Data
Maximum 7-Day	26.1

[1] Historical data based on plant operations data, Jan-08 to Sep-09

[2] Historical temperature data NOT sorted.

OWASA Mason Farm WWTP
Historical Load Peaking Factors, Sorted Data

Criteria	BOD_5	CBOD	TSS	VSS	TKN	NH_3	TP
Minimum Day	0.42	0.42	0.34	0.34	0.63	0.60	0.48
Average Annual	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Maximum Month	1.30	1.30	1.52	1.51	1.26	1.16	1.19
Maximum 30-Day	1.37	1.37	1.56	1.55	No Data	1.18	1.21
Maximum 7-Day	1.85	1.85	2.15	2.01	1.42	1.45	1.45
Maximum Day	2.35	2.35	3.31	3.29	1.70	1.69	2.18

OWASA Mason Farm WWTP
Historical Loads, Sorted Data

Criteria	BOD_5	CBOD	TSS	VSS	TKN	NH_3	TP
	ppd	ppd	ppd	ppd	ppd	ppd	ppd
Minimum Day	5,000	5,000	5,700	5,200	1,590	1,080	220
Average Annual	12,700	12,700	17,300	15,700	2,470	1,810	460
Maximum Month	15,500	15,500	25,500	22,900	3,200	2,090	560
Maximum 30-Day	16,700	16,700	28,000	24,900	No Data	2,150	570
Maximum 7-Day	21,800	21,800	35,600	30,600	3,600	2,640	650
Maximum Day	32,600	32,600	54,800	49,800	4,130	3,070	980

Statistical Data Analysis of Sorted Data (Influent Concentrations, mg/L)

	BOD_5	CBOD	TSS	VSS	TKN	NH_3	TP
Original Data Set							
Number of Data	417	417	419	418	88	420	419
Average (Mean)	204.4	204.4	286.3	260.0	39.0	28.7	7.3
Standard Deviation	66.8	66.8	121.0	110.7	8.3	6.3	1.9
Median	201.6	201.6	260.0	235.0	37.8	28.1	7.0
Maximum	538.0	538.0	980.0	889.0	61.5	84.7	19.4
Maximum Date	4/17/08	4/17/08	5/18/08	5/18/08	1/12/09	6/8/08	6/8/08
Minimum	68.7	68.7	88.0	80.0	17.9	12.0	3.4
Minimum Date	3/29/09	3/29/09	9/7/08	9/7/08	3/16/09	7/6/08	5/19/08
99^{th} Percentile	402.0	402.0	765.6	704.2	60.2	48.6	14.9
95^{th} Percentile	328.3	328.3	508.0	462.1	54.7	37.6	10.8
90^{th} Percentile	283.1	283.1	426.6	382.2	49.8	34.3	9.5
Regular Data Screening Method							
Mean + 2*SD	338.0	338.0	528.3	481.3	55.6	41.3	11.1
Percentile	95.90%	95.90%	95.20%	95.40%	95.70%	97.30%	96.00%
Count > Mean + 2*SD	18	18	20	19	4	12	17
Mean + 3*SD	404.8	404.8	649.4	592.0	63.9	47.5	13.0
Percentile	99.10%	99.10%	97.70%	97.90%	100.0%	98.80%	98.00%
Count > Mean + 3*SD	4	4	10	9	0	5	9
Mean - 2*SD	70.8	70.8	44.3	38.6	22.4	16.2	3.6
Percentile	0.00%	0.00%	0.00%	0.00%	1.00%	0.90%	0.20%
Count < Mean - 2*SD	1	1	0	0	1	5	1
Mean - 3*SD	4.1	4.1	-76.7	-72.1	14.1	9.9	1.7
Percentile	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Count < Mean - 3*SD	0	0	0	0	0	0	0
2 SD Screened Data Set							
Number of Data	398	398	399	399	83	403	401
Average (Mean)	196.3	196.3	266.5	242.3	38.3	28.2	7.1
Standard Deviation	53.9	53.9	80.5	73.8	6.9	4.2	1.3
Median	199.6	199.6	256.0	232.0	37.5	28.0	6.9
Maximum	336.0	336.0	526.0	476.0	55.0	40.7	11.0
Maximum Date	2/5/09	2/5/09	12/15/08	12/15/08	1/22/08	11/10/08	1/15/08
Minimum	74.2	74.2	88.0	80.0	22.9	17.4	3.6
Minimum Date	3/15/09	3/15/09	9/7/08	9/7/08	6/30/08	6/14/09	6/5/08
99^{th} Percentile	324.6	324.6	486.1	444.2	54.2	39.4	10.8
95^{th} Percentile	290.6	290.6	430.8	388.3	50.5	35.1	9.7
90^{th} Percentile	269.9	269.9	376.2	342.2	46.8	33.6	8.9
3 SD Screened Data Set							
Number of Data	413	413	409	409	88	415	410
Average (Mean)	202.1	202.1	274.5	249.7	39.0	28.3	7.2
Standard Deviation	62.5	62.5	94.3	86.7	8.3	4.9	1.5
Median	201.0	201.0	258.0	234.0	37.8	28.0	6.9
Maximum	402.0	402.0	641.0	583.0	61.5	47.5	12.6
Maximum Date	6/8/08	6/8/08	9/10/08	5/7/08	1/12/09	12/10/08	3/11/09
Minimum	68.7	68.7	88.0	80.0	17.9	12.0	3.4
Minimum Date	3/29/09	3/29/09	9/7/08	9/7/08	3/16/09	7/6/08	5/19/08
99^{th} Percentile	386.3	386.3	592.4	548.4	60.2	43.8	11.5
95^{th} Percentile	315.2	315.2	464.0	422.4	54.7	37.1	10.3
90^{th} Percentile	278.4	278.4	392.4	358.4	49.8	34.1	9.1

Year 1
January-08 to December-08

	Influent Flow		Influent CBOD				Influent TSS				Influent VSS				Influent TKN				Influent Ammonia				Influent Total Phosphorus				Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Temperature Deg C
Minimum Day[2]	5.74	0.75	82.50	4937.14	0.42	77.47	88.00	5631.17	0.34	88.37	80.00	5136.71	0.34	80.61	22.90	1589.19	0.63	24.94	12.00	1119.49	0.62	17.57	3.35	216	0.48	3.39	15
Average Annual[1]	7.64	1.00	186.46	11797.62	1.00	185.13	261.12	16550.18	1.00	259.71	239.05	15142.95	1.00	237.63	39.45	2531.84	1.00	39.73	28.77	1816.51	1.00	28.51	7.08	448	1.00	7.02	20.54
Maximum Month[3]	9.02	1.18	250.56	15371.63	1.30	241.22	391.56	25079.46	1.52	393.55	357.28	22830.56	1.51	358.26	49.25	3190.37	1.26	50.06	31.91	2043.68	1.13	32.07	8.10	512	1.14	8.04	25.68
Max 30-Day[5]	9.37	1.23	261.67	16128.91	1.37	253.10	397.75	25763.85	1.56	404.29	363.19	23462.59	1.55	368.18	No Data	No Data	No Data	No Data	34.35	2149.25	1.18	33.73	8.25	513	1.15	8.05	No Data
Max 7-Day[5]	11.53	1.51	312.50	21782.93	1.85	341.82	531.60	35535.16	2.15	557.63	478.40	30507.65	2.01	478.73	50.60	3596.36	1.42	56.44	39.47	2635.33	1.45	41.35	10.64	647	1.45	10.15	26.10
Maximum Day[5]	22.54	2.95	402.00	27512.49	2.33	431.73	641.00	54742.43	3.31	859.03	583.00	49789.13	3.29	781.30	60.00	4127.09	1.63	64.76	47.50	3069.20	1.69	48.16	12.00	975	2.18	15.30	26.50
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Year 2
January-09 to December-09

	Influent Flow		Influent CBOD				Influent TSS				Influent VSS				Influent TKN				Influent Ammonia				Influent Total Phosphorus				Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Temperature Deg C
Minimum Day[2]	5.97	0.75	68.70	7152.23	0.52	108.36	106.00	8149.96	0.45	123.47	104.00	8109.82	0.50	122.87	17.90	1580.52	0.67	23.95	12.70	1079.23	0.60	16.35	4.05	297.49	0.64	4.51	14.60
Average Annual[1]	7.91	1.00	211.34	13867.61	1.00	210.10	274.42	18194.17	1.00	275.65	247.23	16375.91	1.00	248.10	36.54	2360.54	1.00	35.76	27.43	1792.53	1.00	27.16	7.08	466.54	1.00	7.07	19.72
Maximum Month[3]	9.88	1.25	239.64	15448.76	1.11	234.05	386.80	25440.67	1.40	385.43	344.10	22553.53	1.38	341.69	42.45	2763.34	1.17	41.87	32.72	2086.26	1.16	31.61	8.45	554.42	1.19	8.40	25.38
Max 30-Day[5]	9.90	1.25	259.35	16680.45	1.20	252.71	424.47	27940.15	1.54	423.30	378.35	24856.79	1.52	376.59	No Data	No Data	No Data	No Data	32.72	2121.02	1.18	32.13	8.80	563.13	1.21	8.53	No Data
Max 7-Day[5]	11.45	1.45	343.75	21783.16	1.57	330.02	503.00	31946.12	1.76	483.99	451.00	28645.61	1.75	433.99	45.90	2919.77	1.24	44.24	35.98	2299.17	1.28	34.83	9.77	619.84	1.33	9.39	15.85
Maximum Day[5]	13.80	1.74	402.00	32549.16	2.35	493.13	630.00	42226.82	2.32	639.75	568.00	37341.75	2.28	565.74	61.50	4003.26	1.70	60.65	40.20	2494.04	1.39	37.79	12.60	929.98	1.99	14.09	25.60
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Years 1 through 5 - Maximum
Maximum from Above Annual Data

	Influent Flow		Influent CBOD				Influent TSS				Influent VSS				Influent TKN				Influent Ammonia				Influent Total Phosphorus				Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Temperature Deg C
Minimum Day[2]	5.97	0.75	82.50	7152.23	0.52	108.36	106.00	8149.96	0.45	123.47	104.00	8109.82	0.50	122.87	22.90	1589.19	0.67	24.94	12.70	1119.49	0.62	17.57	4.05	297.49	0.64	4.51	15.40
Average Annual[1]	7.91	1.00	211.34	13867.61	1.00	210.10	274.42	18194.17	1.00	275.65	247.23	16375.91	1.00	248.10	39.45	2531.84	1.00	39.73	28.77	1816.51	1.00	28.51	7.08	466.54	1.00	7.07	20.54
Maximum Month[3]	9.88	1.25	250.56	15448.76	1.30	241.22	391.56	25440.67	1.52	393.55	357.28	22830.56	1.51	358.26	49.25	3190.37	1.26	50.06	32.72	2086.26	1.16	32.07	8.45	554.42	1.19	8.40	25.68
Max 30-Day[5]	9.90	1.25	261.67	16680.45	1.37	253.10	424.47	27940.15	1.56	423.30	378.35	24856.79	1.55	376.59	No Data	No Data	No Data	No Data	34.35	2149.25	1.18	33.73	8.80	563.13	1.21	8.53	No Data
Max 7-Day[5]	11.53	1.51	343.75	21783.16	1.85	341.82	531.60	35535.16	2.15	557.63	478.40	30507.65	2.01	478.73	50.60	3596.36	1.42	56.44	39.47	2635.33	1.45	41.35	10.64	646.92	1.45	10.15	26.10
Maximum Day[5]	22.54	2.95	402.00	32549.16	2.35	493.13	641.00	54742.43	3.31	859.03	583.00	49789.13	3.29	781.30	61.50	4127.09	1.70	64.76	47.50	3069.20	1.69	48.16	12.60	974.82	2.18	15.30	26.50
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Years 1 through 5 - Average
Averages from Above Annual Data

	Influent Flow		Influent CBOD				Influent TSS				Influent VSS				Influent TKN				Influent Ammonia				Influent Total Phosphorus				Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Temperature Deg C
Minimum Day[2]	5.86	0.75	75.60	6044.69	0.47	92.92	97.00	6890.57	0.39	105.92	92.00	6623.26	0.42	101.74	20.40	1584.86	0.65	24.44	12.35	1099.36	0.61	16.96	3.70	256.66	0.56	3.95	15.00
Average Annual[1,3]	7.75	1.00	196.34	12,619	1.00	195.24	266.49	17,214	1.00	266.32	242.33	15,637	1.00	241.93	38.29	2,464	1.00	38.12	28.21	1,807	1.00	27.95	7.08	455	1.00	7.05	20.22
Maximum Month[3]	9.45	1.21	245.10	15410.19	1.21	237.63	389.18	25260.06	1.46	389.49	350.69	22692.05	1.44	349.98	45.85	2976.86	1.22	45.96	32.32	2064.97	1.14	31.84	8.27	533.40	1.17	8.22	25.53
Max 30-Day[5]	9.64	1.24	260.51	16404.68	1.28	252.91	411.11	26852.00	1.55	413.80	370.77	24159.69	1.53	372.38	No Data	No Data	No Data	No Data	33.53	2135.14	1.18	32.93	8.53	538.17	1.18	8.29	No Data
Max 7-Day[5]	11.49	1.48	328.13	21783.05	1.71	335.92	517.30	33740.64	1.95	520.81	464.70	29576.63	1.88	456.36	48.25	3258.07	1.33	50.34	37.72	2467.25	1.37	38.09	10.20	633.38	1.39	9.77	20.98
Maximum Day[5]	18.17	2.35	402.00	30030.82	2.34	462.43	635.50	48484.62	2.81	749.39	575.50	43565.44	2.78	673.52	60.75	4065.17	1.66	62.71	43.85	2781.62	1.54	42.97	12.30	952.40	2.09	14.69	26.05
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

Years 1 through 5 - Minimum
Minimum from Above Annual Data

	Influent Flow		Influent CBOD				Influent TSS				Influent VSS				Influent TKN				Influent Ammonia				Influent Total Phosphorus				Influent Temp.
	Flow MGD	Peaking Factor	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Actual Concentration[6] mg/L	Load, ppd	Load Peaking Factor	Calculated Concentration[7] mg/L	Temperature Deg C
Minimum Day[2]	5.74	0.75	68.70	4937.14	0.42	77.47	88.00	5631.17	0.34	88.37	80.00	5136.71	0.34	80.61	17.90	1580.52	0.63	23.95	12.00	1079.23	0.60	16.35	3.35	215.84	0.48	3.39	14.60
Average Annual[1]	7.64	1.00	186.46	11797.62	1.00	185.13	261.12	16550.18	1.00	259.71	239.05	15142.95	1.00	237.63	36.54	2360.54	1.00	35.76	27.43	1792.53	1.00	27.16	7.08	447.54	1.00	7.02	19.72
Maximum Month[3]	9.02	1.18	239.64	15371.63	1.11	234.05	386.80	25079.46	1.40	385.43	344.10	22553.53	1.38	341.69	42.45	2763.34	1.17	41.87	31.91	2043.68	1.13	31.61	8.10	512.38	1.14	8.04	25.38
Max 30-Day[5]	9.37	1.23	259.35	16128.91	1.20	252.71	397.75	25763.85	1.54	404.29	363.19	23462.59	1.52	368.18	No Data	No Data	No Data	No Data	32.72	2121.02	1.18	32.13	8.25	513.22	1.15	8.05	No Data
Max 7-Day[5]	11.45	1.45	312.50	21782.93	1.57	330.02	503.00	31946.12	1.76	483.99	451.00	28645.61	1.75	433.99	45.90	2919.77	1.24	44.24	35.98	2299.17	1.28	34.83	9.77	619.84	1.33	9.39	15.85
Maximum Day[5]	13.80	1.74	402.00	27512.49	2.33	431.73	630.00	42226.82	2.32	639.75	568.00	37341.75	2.28	565.74	60.00	4003.26	1.63	60.65	40.20	2494.04	1.39	37.79	12.00	929.98	1.99	14.09	25.60
Peak Hour	No Data	No Data	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--

[1]Calculated using data within 2 standard deviations of the mean.
[2]Calculated using data within 3 standard deviations of the mean.
[3]Values shown are the average of all data points.
[4]Minimum Day from Data set.
[5]Maximum from Data Set.
[6]Values shown are actual concentration numbers pulled from the sorted data set. Not to be used for design purposes.
[7]Values shown are calculated from the corresponding load divided by 8.34 times the average annual flow.

Monthly Average Flow, Concentrations and Loadings[1]

	Influent Flow	Influent BOD		Influent CBOD		Influent TSS		Influent VSS		Influent TKN		Influent Ammonia		Influent Total Phosphorus		Influent Temp.
Month	MGD	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Conc. mg/L	Avg. Load ppd	Avg. Temp C
Jan-08	7.13	196.23	11,734	196.23	11,734	240.62	14,428	219.71	13,176	42.30	2,512	30.70	1,831	7.69	460	16.40
Feb-08	7.64	215.94	13,906	215.94	13,906	266.90	17,102	242.65	15,526	39.85	2,657	30.45	1,938	7.38	474	16.24
Mar-08	8.13	185.91	12,263	185.91	12,263	267.48	17,899	240.67	15,995	42.36	2,836	29.11	1,948	7.40	486	16.42
Apr-08	8.43	152.01	10,575	152.01	10,575	233.59	16,325	214.68	14,995	34.90	2,524	27.01	1,867	6.70	470	18.13
May-08	7.27	179.12	10,873	179.12	10,873	248.13	15,134	225.69	13,742	37.65	2,436	29.07	1,761	7.14	431	20.29
Jun-08	6.83	194.61	11,120	194.61	11,120	293.86	16,854	263.68	15,103	31.34	1,849	28.71	1,646	7.54	434	23.25
Jul-08	7.36	163.71	9,994	163.71	9,994	231.14	14,238	209.62	12,891	38.38	2,455	26.07	1,575	6.03	372	24.60
Aug-08	7.64	145.38	9,436	145.38	9,436	207.75	13,650	189.30	12,300	42.20	2,559	25.87	1,650	5.98	383	25.68
Sep-08	9.02	168.36	12,273	168.36	12,273	243.10	17,023	221.43	15,494	40.35	3,037	28.65	2,023	6.33	449	24.73
Oct-08	7.29	195.82	11,970	195.82	11,970	256.23	15,588	238.09	14,485	48.88	3,002	30.14	1,840	7.33	450	23.02
Nov-08	7.23	241.94	14,873	241.94	14,873	305.94	18,634	281.00	17,113	36.55	2,240	31.66	1,948	7.52	461	20.10
Dec-08	7.75	214.21	13,523	214.21	13,523	369.88	23,463	347.35	22,320	39.46	2,456	28.14	1,783	8.10	512	17.18
Jan-09	7.72	199.27	13,600	199.27	13,600	362.89	24,014	322.33	21,245	36.10	2,284	28.28	1,848	8.45	554	15.54
Feb-09	7.62	222.43	14,198	222.43	14,198	292.00	18,623	263.56	16,800	39.00	2,450	32.72	2,086	8.12	517	15.38
Mar-09	9.88	168.98	13,621	168.98	13,621	255.95	21,292	230.23	19,102	31.93	2,633	24.22	1,905	6.25	516	15.41
Apr-09	8.26	193.68	13,394	193.68	13,394	268.37	18,411	249.05	17,068	39.65	2,763	28.71	1,980	7.07	488	18.08
May-09	7.41	238.86	14,832	238.86	14,832	268.50	16,744	239.40	14,906	40.78	2,495	28.37	1,768	6.94	433	20.83
Jun-09	7.81	224.66	14,097	224.66	14,097	246.19	15,398	222.67	13,923	35.76	2,166	25.41	1,592	6.58	411	22.83
Jul-09	7.11	214.25	12,915	214.25	12,915	251.40	15,158	225.00	13,572	32.93	1,912	25.76	1,551	6.81	410	24.58
Aug-09	7.48	228.69	14,234	228.69	14,234	263.82	16,525	238.64	14,948	36.30	2,240	26.28	1,647	6.64	414	25.38

[1]Based on data within 2 SD of mean.

LONG-RANGE WATER SUPPLY PLAN

FINAL REPORT

APRIL 8, 2010

Orange Water and Sewer Authority
Carrboro, North Carolina



Cane Creek Reservoir

OWASA

A public, non-profit agency providing water, sewer and reclaimed water services to the Carrboro-Chapel Hill community.

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ORANGE WATER AND SEWER AUTHORITY

A public, non-profit agency providing water, sewer and reclaimed water services to the Carrboro-Chapel Hill community.

FINAL REPORT
LONG-RANGE WATER SUPPLY PLAN

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Long-Range Water Supply Plan
Executive Summary

The threat of water shortages during the historic droughts of 2001-02 and 2007-08 confronted OWASA with challenging questions. Do we have enough water to meet our community's present and future needs? What are the most reliable, cost-effective, and sustainable supply and demand methods for meeting those needs? How will our options be affected by future climate and land use changes? This report answers those questions with a very positive outlook for our water supply future.

Thanks largely to the 25 percent reduction in water use achieved by all OWASA customers since 2002, our locally protected Cane Creek, University Lake, and Quarry Reservoir supplies can meet our expected needs for the next 50 years under most circumstances; *but, it is essential that the recent gains in water efficiency be sustained in the future for this Plan to be fully realized.* To that end, OWASA will continue to promote water conservation and efficiency through customer awareness and education, targeted technical assistance, conservation pricing, and support for increased water efficiency standards in new and renovated buildings.

Expanding the Quarry Reservoir continues to offer the most effective supply supplement for the least investment and will provide full local control of a substantial amount of high quality water. However, we will still need the additional "insurance policy" of Jordan Lake in the event of severe drought, critical facility failure, or other unforeseen/emergency situation. The only economically feasible access to our Jordan Lake water supply storage allocation will be through partnership arrangements with area utilities; that is, by securing the permanent ability to obtain water under appropriate conditions of supply and demand – either through purchases or other arrangements. OWASA's continued participation in the Jordan Lake Partnership offers important opportunities to develop such agreements. It is essential that we retain our water supply allocation and acquire cost-effective access to it.

Investing OWASA funds to expand the reclaimed water system or to establish financial incentive programs, such as plumbing fixture rebates, to promote additional water conservation is not recommended at this time, because these options are less cost-effective than others evaluated. Nevertheless, we will continue to examine such opportunities on a case-by-case basis.

This report presents a positive water supply future for OWASA and the conservation-minded community we serve. The continued and proactive practice of this conservation ethic will enable future generations to enjoy a reliable and more sustainable supply of high quality drinking water with far less capital investment than anticipated in previous reports.

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SECTION 1
BACKGROUND AND PURPOSE

The purpose of OWASA's Long-Range Water Supply Plan is to determine the optimum mix of strategies that will ensure a reliable, cost-effective, and sustainable water supply to meet the needs of Carrboro, Chapel Hill, and the University of North Carolina at Chapel Hill (UNC) through 2060. During the 10 years since OWASA's 50-year water supply plan was last revised, two historic droughts have occurred; OWASA's rate structure has been significantly modified; permanent process water recycling has been implemented at the Jones Ferry Road Water Treatment Plant (WTP); a major reclaimed water (RCW) system has begun operating; non-potable water sources and advanced water use efficiency technologies are now increasingly being incorporated into new commercial and residential development; and customer consumption patterns have changed markedly.

This update recognizes the need to revisit underlying assumptions – especially in view of such substantially changed conditions – and the relative benefits and costs of supplemental supply and demand management options that were evaluated previously, as well as others that have not been considered. This report focuses on untreated (raw) water. It does not address other important aspects of OWASA's drinking water system, such as treatment, storage, and distribution, that also determine the overall reliability of our system.

Guiding Principles

Consistent with OWASA's Mission Statement and longstanding support of proactive resource planning, source water protection, and water conservation, this report has been guided by the following principles:

- *Making the highest and best use of our local water resources: University Lake, Cane Creek, and the Quarry Reservoir.*
- *Cost-effectiveness for OWASA and its customers, with consideration for economic, environmental, and social costs and benefits.*
- *Reliability, redundancy, and flexibility to maintain a full range of water supply options for future generations.*

Assumptions

The technical and economic methods on which the major findings and recommendations of this report were based are described below. Key assumptions that underlie the entire report include:

- *OWASA's utility service area, as defined by the urban services boundaries of Carrboro, Chapel Hill, and Orange County, will remain unchanged through 2060.*
- *Water demand projections do not anticipate any retail or wholesale water, wastewater, or RCW sales outside of the existing service area.*
- *Future State and Federal regulations will allow OWASA to continue recycling water treatment plant process water and operating its RCW program.*

Report Preparation

OWASA retained the engineering firm of Hazen and Sawyer, P.C. to conduct the underlying technical and economic analyses for this report. That work is presented in a series of Technical Memoranda (TMs) in the Appendices. Additionally, Hazen and Sawyer developed specialized spreadsheet tools for analyzing the financial costs and benefits of each supplemental supply and demand reduction option under a range of variables. OWASA staff then applied these tools to generate the cost information presented in this report. Some of the TMs are followed by two versions of the cost analyses: (1) the original Hazen and Sawyer spreadsheets as reflected in the text and tables of the corresponding TMs (these did not evaluate the costs of meeting the water demand projections subsequently developed by OWASA); and (2) OWASA staff's application of the spreadsheets as reflected in the text and tables in the body of this report.

Fifty-year water demand projections were developed by OWASA in consultation with planning and economic development staffs of Carrboro, Chapel Hill, Orange County, UNC, and UNC Hospitals.

This report was compiled and written by OWASA staff and reviewed by Hazen and Sawyer for technical accuracy.

Engineering and Cost Estimates

A goal of the planning level economic estimates and financial evaluations was to apply uniform methods and assumptions that would provide valid relative comparisons among the individual water supply and demand reduction options. All costs in the individual TMs represent order-of-magnitude estimates to be used for comparing the options and for long-range planning purposes. These estimates were developed to a sufficient level of detail that incorporates the major components of each alternative for planning-level purposes. Estimates were not based on the more detailed levels of engineering and cost analyses customarily applied to preliminary and final stages of project design work. Additional descriptions of the approach and methods are presented in **Appendix I.**

Financial Cost/Benefit Analysis

All financial evaluations incorporate the net present life-cycle costs (NPC) of each supply or demand reduction option. These include capital costs, periodic rehabilitation/replacement, and fixed and variable operating and maintenance costs. Capital improvements are assumed to have a useful life of 50 years, with salvage value (useful remaining life of the facilities in 2060) credited toward life-cycle costs.

Capital costs are assumed to be debt-financed for 25 years at a uniform annual interest rate of 5%. All costs are discounted back to the present at an assumed annual rate of 5%. NPC estimates for temporary water shortage restrictions (Option 5C) include the projected net reductions (unbudgeted) of water and sewer revenues and corresponding reductions in variable operating costs; but the NPC estimates for expanding the RCW system (Option 5A) or for additional conservation investments (Option 5B) do not include any revenue losses or reduced

operating costs, because the expected demand reductions of these options would likely be accounted for when planning/budgeting for those projects and therefore incorporated into OWASA's longer term financial decisions, including future water rates.

Net present costs (in 2009 dollars) are generally presented in two ways for each identified option: (1) as the capital cost per million gallons per day (mgd) of additional yield (or demand reduction), and (2) as the total life-cycle cost per 1,000 gallons supplied to meet any deficits projected during the 50-year planning period.

Future Updates

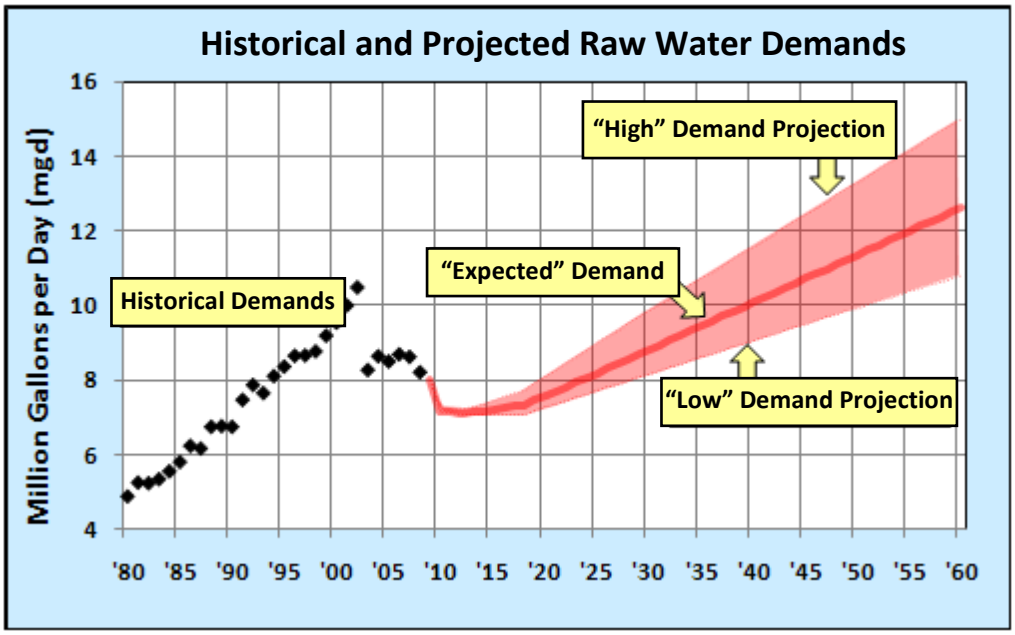
OWASA's Long-Range Water Supply Plan is not a static report, but will be periodically reviewed and updated to reflect changes to the assumptions, conditions, and information on which this 2010 edition is based.

SECTION 2
50-YEAR WATER DEMAND PROJECTIONS

Demand projections were developed from historical and recent OWASA consumption data by major user groups (single family detached, condominium/townhouse, multifamily, UNC Central Campus, UNC Hospitals, "other" non-residential users); historical, recent, and anticipated development trends as documented by local building permit and certificate of occupancy data, published planning documents of Carrboro, Chapel Hill, and UNC; and consultation with planning and economic development staff of Carrboro, Chapel Hill, Orange County, UNC, and UNC Hospitals. The demand projections incorporated the most current information available about development plans for Carolina North.

Historical and projected demands are presented in Figure 1. Black diamond symbols indicate average annual water withdrawals from OWASA's reservoirs from 1980 through 2008, and the red bands represent High, Low, and Expected demand projections through 2060. Principal assumptions for those scenarios are listed below the graph. Additional details are provided in **Appendix II.**

Figure 1. 50-Year Raw Water Demand Projections



"Expected Demand" Assumptions:

- The pace of local development activity will return to the 20-year average of approximately 560 new meter equivalents (MEs) per year within the next two to three years and will continue at this linear rate through 2060. (One ME represents the water demand exerted by a typical single family residential customer. A non-residential or institutional customer with greater needs requires a larger meter, and therefore represents multiple MEs.) Although this may be an unrealistically high projection of future growth,

none of the long-range planning documents of Carrboro, Chapel Hill, and Orange County, with planning horizons that end in 2035, provide any further guidance.

- The future development profile of OWASA's service area will follow recent trends with respect to the overall mix of single versus multi-family residential, commercial, and other uses.

- UNC Central Campus and UNC Hospitals building space will increase by 4.6 million gross square feet (GSF) (per the *2006 UNC Campus Master Plan)*, at a constant rate of approximately 0.16 million GSF per year through 2028, which is the projected Central Campus buildout date.

- Carolina North will build out to a total of 8 million GSF at a constant rate of 0.17 million GSF per year through 2060. Until more detailed information is available, demand projections assume the same mix of water use and consumption rates (gallons per day per GSF) as currently observed on the Central Campus and UNC Hospitals. The "Expected Demand" projection assumes that 25% of the total water needs at Carolina North will be met with reclaimed (RCW) or other non-potable water (NPW).

- RCW demand by UNC/UNC Hospitals will increase through 2028 (the estimated date of Central Campus buildout) per previous estimates provided by UNC.

- Water consumption rates assumed for major user groups are based on actual OWASA averages observed from 2004-2007, when annual demand remained stable. (Consumption during 2008 was 11% less than the preceding 4 years.)

"Lower Demand" Scenario – Same assumptions as for "Expected Demands," except for the following:

- 15% less residential and non-residential (non-UNC) development through 2060.

- 50% (rather than 25%) RCW and NPW use at Carolina North.

- Additional conservation of 5 to 12% continues to accrue through 2060 among all existing and new customers.

"Higher Demand" Scenario – Same assumptions as for "Expected Demands," except for the following:

- One new high density mixed use development project – similar in scale to the East 54 project currently under construction in Chapel Hill – with water demands equivalent to approximately 150 new single family homes added each year through 2060.

- New non-residential (non-UNC) development through 2060 is 25% greater than expected.

- UNC Central Campus and UNC Hospitals growth through 2028 is 25% greater than expected.

- Carolina North water demand is 25% higher than expected and no RCW or non-potable water use occurs.

50-YEAR WATER DEMAND PROJECTIONS

Key Findings

Demand projections through 2060 are substantially less than OWASA's 2001 *Comprehensive Water and Sewer Master Plan* projections for 2050 due to three main factors:

1. Increased water use efficiency of 20 – 25% among all sectors of OWASA's customer base during recent years: These changes appear to be permanent and should be considered to represent "new normal" consumption patterns for OWASA's customers.

2. OWASA/UNC RCW system: Began operating in April 2009 and projected to replace 1 mgd of potable water demand by 2013 and up to 2 mgd by 2028.

3. Slower pace of development activity: The installation rate of new OWASA service connections has declined steadily since 2004, reflecting the lower pace of development in Carrboro and Chapel Hill. Community growth is expected to continue at this reduced pace through the current economic recession, but is then assumed to return to pre-recession levels.

The combined effects of these trends are reflected in the net decline and flattened rate of raw water demands projected for the next 10 years or more as shown in Figure 1, and in the 2060 demands that are lower than previously forecast for 2050.

Key Actions

- Continue monitoring critical indicators of community growth (certificates of occupancy, new OWASA meter installations, etc.).

- Continue monitoring monthly water use patterns among and within major customer groups.

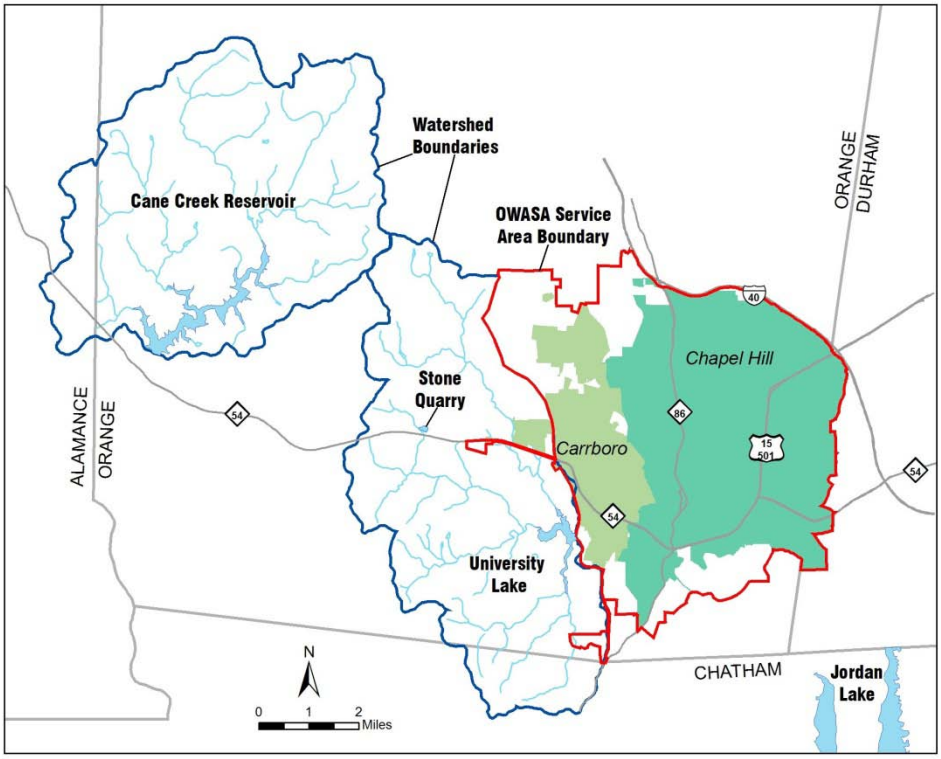
- Compare these data annually to demand projections of this report.

SECTION 3
CAPACITY OF EXISTING SYSTEM

OWASA's existing supply sources include University Lake, Cane Creek Reservoir, and the Quarry Reservoir, as shown in Figure 2.

Raw water from University Lake is pumped to the Jones Ferry Road Water Treatment Plant (WTP) in Carrboro. Cane Creek water can be pumped directly to the WTP; into Phil's Creek near the existing Quarry Reservoir, where it flows downstream to University Lake for re-pumping to the treatment plant; or, into the existing Quarry Reservoir and stored for later use. Water from the Quarry Reservoir can be pumped directly to the Jones Ferry Road WTP or to University Lake via Phil's Creek.

Figure 2. OWASA Water Supply Sources, Watersheds, and Service Area Boundary



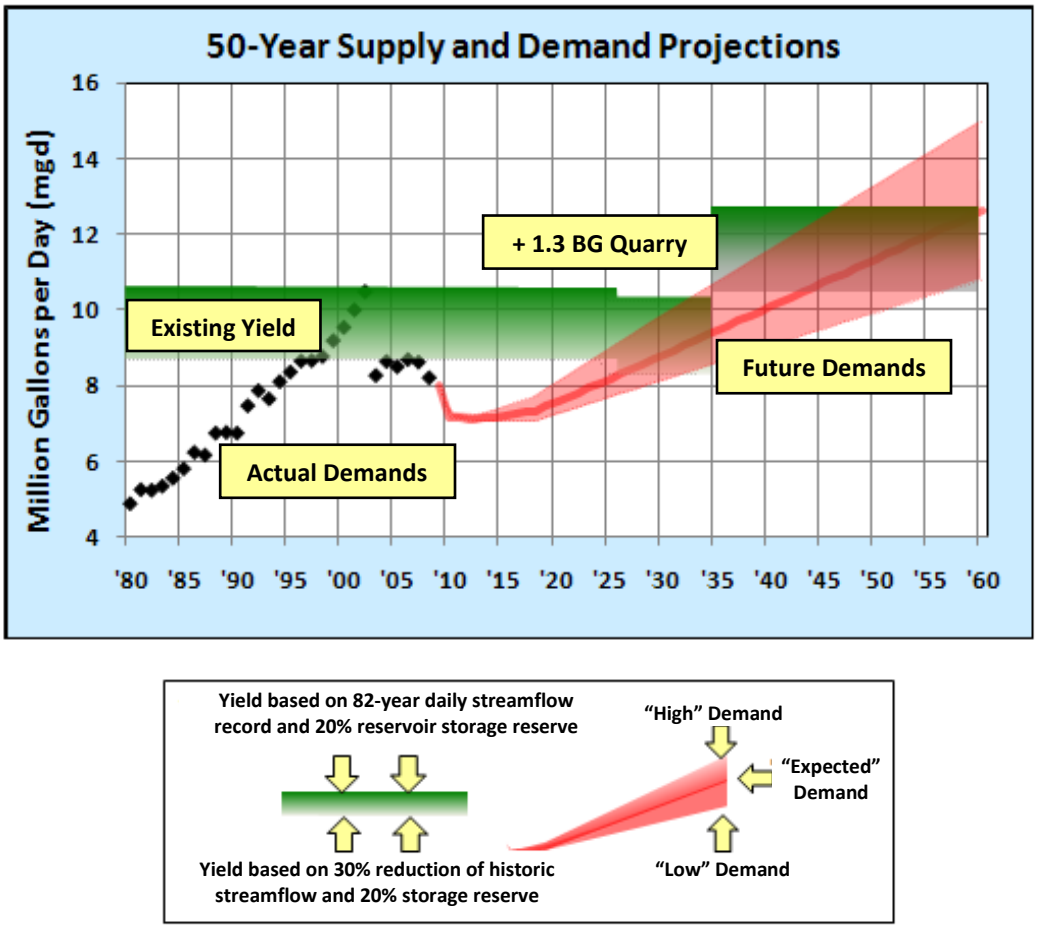
University Lake was created by UNC in 1932. It drains a 30-square mile watershed and has a usable storage capacity of 450 million gallons (MG). The lake and about 500 acres of adjacent lands are still owned by UNC, but OWASA controls all land within 100 feet of the shoreline and is entitled to use University Lake as a water supply source through a contractual agreement with UNC.

Cane Creek Reservoir was created by OWASA and filled in 1989. It can store approximately 3 billion gallons (BG) of water from its 32-square mile drainage area. More than 2,000 acres of surrounding watershed land is either owned by OWASA or protected through permanent conservation easements held by OWASA.

The existing Quarry Reservoir, located about 3 miles west of Carrboro in the University Lake watershed, was acquired in 1979 to supplement OWASA's water supply during severe droughts or emergencies. It has a usable storage volume of 200 MG. Pumping capacity improvements completed in 2007 provide additional operational flexibility. Approvals were obtained in 2001 to expand the nearby active quarry, which is operated (on OWASA-owned property) by the American Stone Company, in the direction of the existing Quarry Reservoir. Per OWASA's agreement with American Stone and per stipulations of the Orange County Special Use Permit that authorized the quarry's expansion, mining operations will cease by 2030, and the large quarry pit will be available for use as a supplementary water source. OWASA owns the entire 190 acres on which the Quarry Reservoir and active pit are located. American Stone Company operates the quarry under a lease agreement with OWASA.

Hydrological modeling conducted for this study determined that the existing system can provide 10.5 million gallons per day (mgd) while still maintaining a 20% storage reserve during a recurrence of the 2001–02 drought of record (see also **Appendix V-A**). Previous yield estimates of 11.7 mgd calculated for the same drought conditions were based on complete reservoir drawdown; i.e., with no water left in reserve. Figure 3 illustrates yield (supply) in relation to the demands depicted earlier in Figure 1.

Figure 3. Supply and Demand Projections through 2060



The green bar extending from the left represents the yield of OWASA's existing University Lake/Cane Creek/Quarry Reservoir system. The right hand bar extending from 2035 to 2060 represents additional yield that would be available with 1.3 BG of expanded Quarry Reservoir storage (see Option 1: Expanded Quarry Reservoir on page 11 for further details). The upper margin of each bar represents total system yield based on 82 years of historic streamflow data. The lower margins are calculated on the basis of a hypothetical 30% reduction of actual streamflow in order to approximate future conditions that might result from major changes in weather patterns (climate change) and/or changes in watershed land use that might affect streamflow.

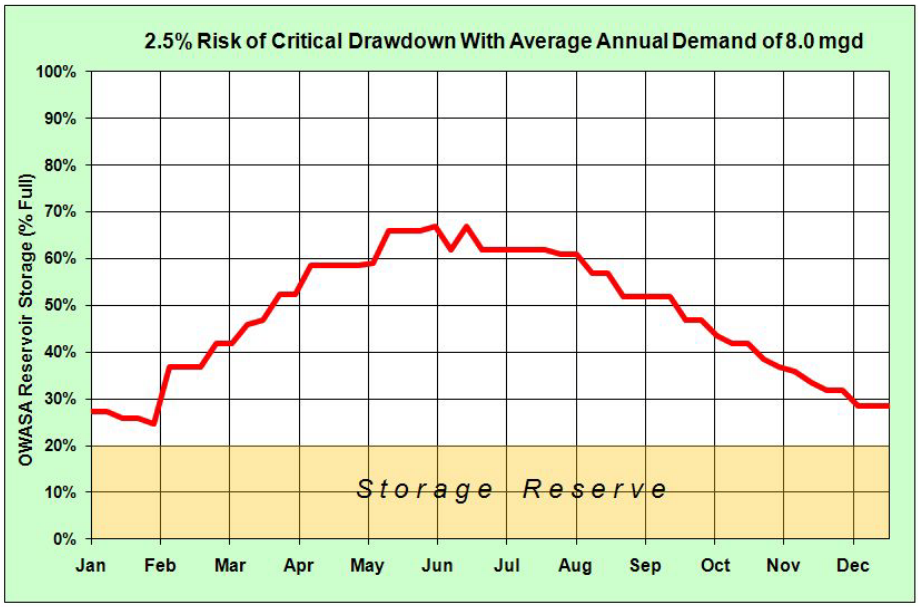
Figure 3 indicates that OWASA's current supply system can meet the community's water supply needs substantially further into the future than previously thought – even under hypothetical High Demand projections. This observation and the potential future benefits of an expanded Quarry Reservoir are discussed later.

For planning purposes, it is recommended that OWASA adopt the 10.5 mgd yield, which includes a 20% storage reserve (700 million gallons) that is believed to provide adequate time to implement emergency supply measures during an extreme drought.

For operational purposes, it is recommended that OWASA maintain sufficient water in storage so that the risk of depletion ("critical drawdown") to 20% or less during any succeeding 12-month period does not exceed 2.5%. This is consistent with the Stage 1 Water Shortage trigger adopted in OWASA's 2009 Water Conservation Standards.

The red line in Figure 4 indicates reservoir storage levels that correspond to a 2.5% probability (risk) that drawdowns to 20% or less of total storage will occur during the following 12 months, assuming an average demand of 8.0 mgd.

Figure 4. 2.5% Risk of Reservoir Drawdown to 20% or Less of Total System Storage



The graph was derived from a statistical analysis of 82 years of streamflow (1926–2007), which indicated that drawdowns to 20% or less would have occurred in 2 of those 82 years (i.e., 2.5%). For the OWASA system, those would have been during the drought years of 2002 and 2007. **Appendix III** includes additional risk graphs for withdrawal rates of up to 12 mgd.

What Does This Mean?

These graphs provide decision-making guidance regarding drought vulnerability. For example, Figure 4 represents a year (such as 2008) with an average demand of 8.0 mgd. The risk of critical drawdown (i.e., to 20% or less of total storage within 12 months of any point on the graph) will be less than 2.5% as long as reservoir levels remain above the red line; i.e., during a year with average demands of 8.0 mgd, OWASA would not declare a Stage I Water Supply Shortage as long as storage levels remained above the red line.

Reservoirs are designed to accumulate and store water during periods of normal and high streamflow so that it can be available when flows decrease or decline to zero during a drought. It is apparent from these analyses that the existing system can reliably meet OWASA's water supply needs when streamflow and reservoir levels are significantly lower than what has traditionally been considered to be "normal."

CAPACITY OF EXISTING SYSTEM

Key Findings

- OWASA's existing system can provide 10.5 million gallons of water per day (mgd) while still maintaining a 20% storage reserve during a recurrence of the 2001–02 drought of record and can reliably meet OWASA's water supply needs when streamflow and reservoir levels are significantly lower than "normal."

- The current system can meet the community's water supply needs substantially further into the future than previously thought – even under hypothetical High Demand projections.

Key Actions

- Continue to follow a risk-based approach when considering drought management decisions during periods of low streamflow and declining reservoir levels. Use the critical drawdown graphs to support important decisions, such as water purchases or Water Supply Shortage declarations.

SECTION 4
SUPPLEMENTAL SUPPLY AND DEMAND REDUCTION OPTIONS

As discussed in the previous section, OWASA's existing system can meet virtually all of our expected needs for the next 25 years, even under a recurrence of the 2001-02 drought of record – provided that the 25 percent reduction in water use achieved since 2002 is sustained in the future. Additional water will be needed after 2035, but the amount and timing of those longer-term needs will depend on actual demands at that time.

The following section includes a brief description of 11 supply and demand-side options for meeting long-term future needs and a summary of their respective benefits and costs. Overall findings are summarized in Tables 2 and 3, which follow the descriptions. It is assumed that the alternative supply sources meet all regulatory standards for water quality and can be appropriately treated at OWASA's WTP without requiring additional process water. **Appendix IV** contains more information about the quality of OWASA's existing sources. *Making the highest and best use of our local water resources* is a guiding principle of these evaluations and subsequent recommendations.

Option 1: Expanded Quarry Reservoir

American Stone Company's ongoing operation at OWASA's Quarry Reservoir (Figure 5) is expected to produce a total storage capacity of 2.2 – 3.0 BG when mining is completed in 2030. The final volume will depend on actual production through 2030, but American Stone is contractually committed to a rate of rock extraction that will result in a final volume of at least 2.2 BG. Between 1.3 and 1.9 BG of this new storage capacity will be readily accessible with existing OWASA pumping facilities, which can withdraw water from a maximum depth of 100 feet. Withdrawals from greater depths would require additional facilities. Modeling results

Figure 5. Existing Quarry Reservoir and Active Quarry Expansion



Existing reservoir is on the left. Active quarry pit is advancing from the right.

indicate that this "shallow version" of Quarry Reservoir expansion will provide between 2.1 and 2.9 mgd of additional yield for an estimated capital investment of less than $2 million (2009), or about $500,000 – $700,000 per mgd of additional yield (see also **Appendix V-A**).

Access to an additional 1.3 – 2.5 mgd of yield from a "deep version" of the Quarry Reservoir would require the construction of a 250-foot vertical shaft and a multi-level pumping gallery for a capital cost of approximately $34 million (2009), or $8.5 to $10 million per additional mgd. A "hybrid" option could include construction of the deep vertical shaft – without installation of the pumps and accessory equipment – before the shallow quarry is filled with water and placed in service in order to preserve the deep quarry as a future option. All versions of Quarry Reservoir expansion would provide additional yield, but would not add substantial redundancy to the overall system if another major supply source or transmission component were out of service for maintenance, equipment failure, or other emergency conditions.

All anticipated configurations could be refilled from the Cane Creek Reservoir through existing pumping and conveyance facilities. Previous studies had proposed refilling the expanded Quarry from University Lake, but this would require major capital improvements to existing infrastructure with little or no increase in operational yield. Increasing the existing pumping and transmission capacity from Cane Creek would reduce the time needed to refill the expanded quarry after a prolonged drought, but would offer no additional yield.

The Expanded Quarry (shallow version) offers the greatest water supply benefit for the lowest economic and environmental costs of all the options, and represents the least challenging regulatory/political hurdles.

Options 2 and 3: Jordan Lake Water Supply Development

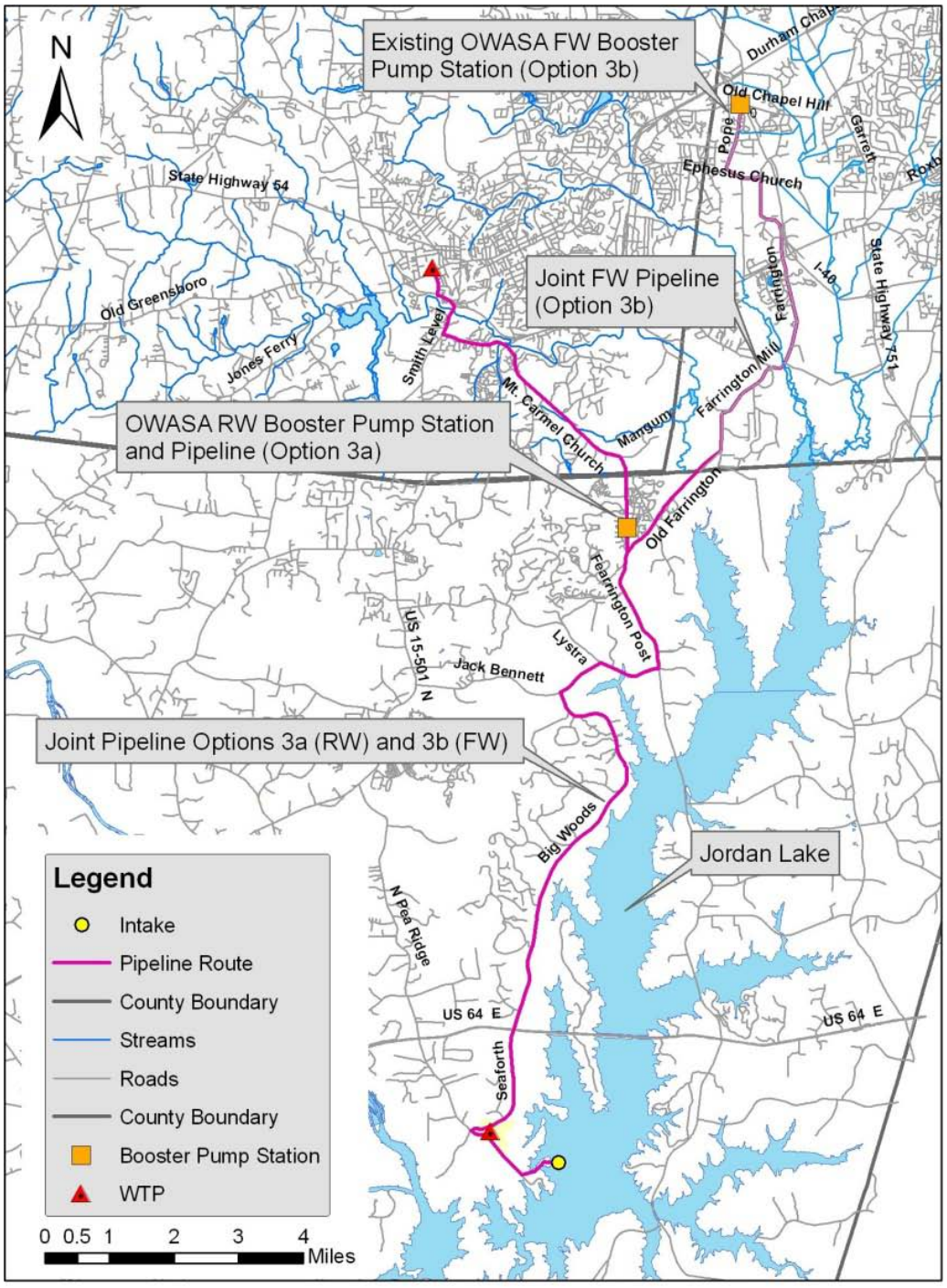
OWASA holds a Level II ("future use") allocation of 5% of Jordan Lake's water supply storage capacity, and owns 125 acres of land adjacent to US Army Corps of Engineers property on the west side of Jordan Lake in Chatham County. The storage allocation corresponds to an estimated yield of 5 mgd on an annual basis. With allowable peak day withdrawals of 10 mgd, analyses conducted for this project determined that OWASA's allocation could actually provide up to 6.2 mgd of additional yield, due to the optimization benefits of our multiple reservoir system.

One set of cost analyses evaluated raw water (RW) intake/pumping/transmission facilities only; a second set considered the additional construction of a new Jordan Lake water treatment plant (WTP). Two hypothetical scenarios were examined: (Option 2) independent OWASA-only facilities **(Appendix VI)**, and (Option 3) shared facilities developed in partnership with other entities **(Appendix VII)**.

Figure 6 shows potential routes of jointly developed raw and finished water pipelines from Jordan Lake to the OWASA and Durham systems. Option 3A would provide raw water intake/pumping/transmission facilities, which could be the initial phase of an Option 3B project that ultimately included a WTP (in which case the RW pipeline would be converted for finished water (FW) transmission to the existing point of interconnection between OWASA and Durham). Option 3A (RW facilities only) would involve the construction of approximately 13 miles of jointly owned (OWASA + partners) pipeline plus an additional 6-mile spur that would only serve OWASA. For the purposes of this study, it was assumed that OWASA would retain a 10 mgd share of jointly owned facilities developed with a total peak day capacity of 65 mgd. OWASA's share of capital costs under this scenario would be approximately $40 million,

compared to the estimated $54 million for an independently developed OWASA-only RW line from Jordan Lake to the Jones Ferry Road WTP. Corresponding unit costs would be $6.4 and $8.8 million per mgd. Based on these planning level estimates and assumptions, the jointly developed project would provide economy of scale savings to OWASA of 25%.

Figure 6. Potential Raw and Finished Water Pipeline Routes from Jordan Lake to the OWASA System



It is unlikely that State/Federal regulatory authorities would approve an OWASA-only project or any other project by a single entity acting on its own. While a joint development project with cooperating entities would provide economies of scale, OWASA would derive little or no economic benefit from sharing the equivalent portion of an Option 3B project (new regional WTP), due to the additional fixed costs of an additional treatment plant and the excess capacity that will remain for decades to come at OWASA's existing Jones Ferry Road WTP. Such a facility would, however, further enhance our water system reliability.

Raw water intake, pumping, and transmission facilities developed in partnership with others appears to represent OWASA's least expensive capital option for obtaining Jordan Lake water in the future, but this could be precluded if the other utilities decide to build a regional WTP and pump FW, rather than RW, to their respective service areas. Nevertheless, the addition of Jordan Lake to OWASA's water supply "portfolio" – as a source of either raw or finished water – would provide significant flexibility and redundancy to the overall system. It is essential for OWASA to retain its Jordan lake storage allocation in order to ensure such future flexibility and reliability.

The eventual form or institutional structure of such a joint arrangement is neither known nor proposed at this time. Possibilities might include the creation of a new regional Jordan Lake development entity; an equity partnership similar to the existing Cary-Apex WTP; or a set of multi-lateral water purchase and sales agreements among individual agencies.

The Jordan Lake Partnership

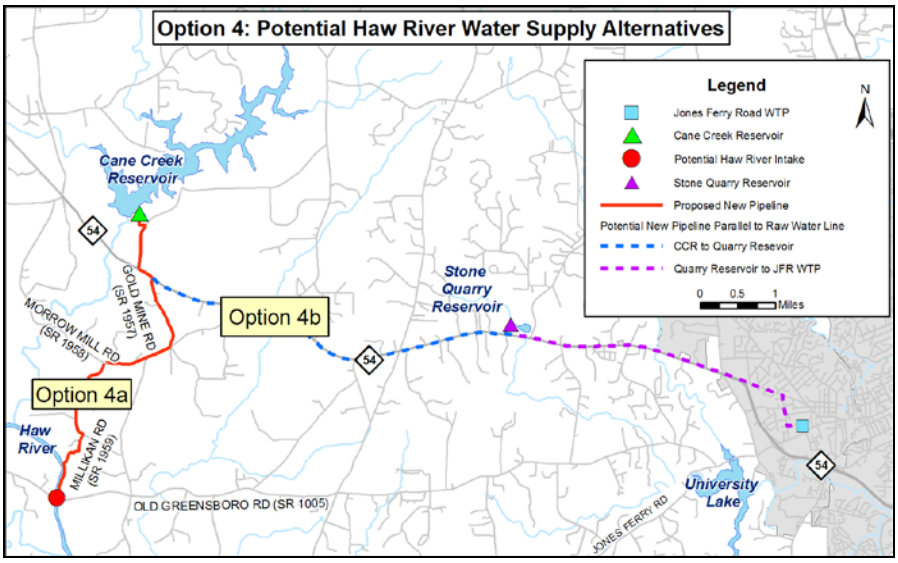
The Jordan Lake Regional Water Supply Partnership was created in 2009 by local jurisdictions and water systems in the Triangle area to jointly plan for the expanded use of the Jordan Lake water supply. OWASA is participating in the Partnership in order to obtain access to its 5 mgd water supply storage allocation, which will only be feasible through a joint arrangement with other local utilities. The Partnership's efforts are currently focused on gathering, reviewing, and refining relevant water supply and demand information of its members to support a new round of Jordan Lake allocation requests and a potential *Jordan Lake Western Intake Preliminary Planning Project.*

Option 4: Permanent Haw River Intake and Pipeline

Permanent facilities to supplement the Cane Creek Reservoir with Haw River water would provide 7.7 mgd of additional yield and add significant system flexibility and redundancy.

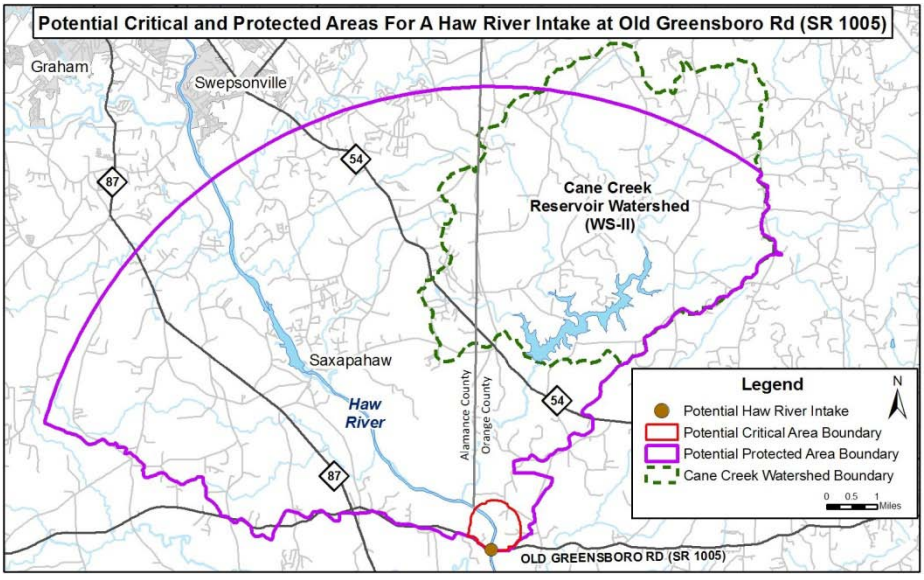
This option would involve the construction of a permanent intake on the Haw River in the vicinity of Old Greensboro Road in Orange County; installation of approximately 5 miles of pipeline to the Cane Creek Reservoir; improvements to the Cane Creek pumping station; and approximately 11 miles of new pipeline parallel to the existing RW transmission main from Cane Creek to the Jones Ferry Road WTP (Figure 7). **Appendix VIII** provides additional detail.

Figure 7. Potential Haw River Intake and Pipeline



Capital and unit costs ($60 million, or $7.7 million per mgd) would be in the same range as those of the Option 3 Jordan Lake joint development scenario, but a permanent supply from the Haw River would likely face significant regulatory and public acceptance challenges, including Section 401/404 review and the need to reclassify a 10-mile portion of the Haw River from Class V ("protected as a water supply which is generally upstream and draining to Class WS-IV waters") to WS-IV ("suitable as a drinking water source"). Prior to considering such a reclassification, the NC Environmental Management Commission would require resolutions of support from the Orange and Alamance County Boards of Commissioners, who exercise planning and zoning jurisdiction in the potential WS-IV Watershed Protected Area, and who would be required to adopt additional State-mandated land use regulations (Figure 8). We believe it is unlikely that such local governmental support could be gained.

Figure 8. Potential Critical and Protected Area Delineations for a Haw River Intake Near Old Greensboro Road



Rather than pursuing a permanent supply source, it is recommended that OWASA retain this option as a worst-case contingency plan for emergency conditions; i.e., for the temporary installation of Haw River withdrawal, pumping, and above ground pipeline facilities to the Cane Creek Reservoir if storage in our existing reservoir/quarry system declined to 20 percent or less and no other emergency supply options were available.

The need for this or any other emergency supply alternative will be less critical if OWASA is able to secure permanent access to Jordan Lake water.

Option 5A: Expand OWASA's Reclaimed Water (RCW) System

RCW use can increase our available water supply to the extent that it replaces existing or future potable water use. To investigate the potential benefits and costs of expanding RCW treatment and distribution capacity, OWASA staff evaluated the hypothetical extension of RCW service to an area in the vicinity of Highway 54 East, Meadowmont, and the Friday Center. Based on existing and anticipated development for this area, we estimated that RCW would reduce long-term potable water use by approximately 0.25 mgd. This would require a substantial capital investment ($7.7 million, or $30 million per mgd of water savings) to expand RCW treatment and pumping facilities at the Mason Farm Wastewater Plant and to install new underground infrastructure in an already developed area. The economic viability of extending the RCW system would be enhanced if the improvements were funded by third party (non-OWASA) sources, but it is unlikely that this alternative will be economically feasible even with such support. Additional details are available in **Appendix IX**.

Financial and budgetary plans for this and other demand reduction projects with associated capital and operating costs, such as Option 5B below, must recognize the likelihood of water and sewer revenue reductions if previous budget and longer term projections have been based on higher (pre-reduction) demand forecasts.

It is recommended that OWASA remain "opportunistic" on a case-by-case basis with respect to additional non-UNC customers, such as the St. Thomas More School located next to the RCW transmission main, who find it cost-effective to extend or connect to the system.

Option 5B: Additional Water Conservation Investments

To evaluate the potential benefits and costs of OWASA-funded conservation incentives, we analyzed the hypothetical replacement of older large volume toilets with high efficiency units. Water savings of 0.5 mgd would require the replacement of approximately 28,000 toilets. With an assumed OWASA cost of $200 per unit ($150 rebate + $50 administrative), the capital costs of this program would be approximately $5.6 million, or $11.2 million per mgd of water savings. For rebates of $75, rather than $150 per unit replaced, total costs would be $3.5 million, or $7.0 million per mgd of water savings.

As discussed under 50-Year Demand Projections, substantial increases in water efficiency have occurred in recent years among all OWASA customers. These are expected to continue in

response to State and local regulatory requirements, construction trends, conservation pricing, etc. without further device give-aways or OWASA financial incentives.

It is recommended that OWASA continue to promote water conservation through customer education, conservation pricing, and limited technical assistance, but without any direct financial incentives (subsidies).

Option 5C: Temporary Water Shortage Restrictions

OWASA successfully reduced water demands during the droughts of 2001-02 and 2007-08 through the temporary imposition of mandatory use restrictions and potable water rate surcharges (2007-08). Although this approach effectively lowers consumption, it has the drawbacks of inconvenience and hardship on customers, as well as a significant loss of water and sewer revenues when the restrictions are imposed in response to severe drought or other unforeseen (and unbudgeted) conditions.

Reducing demand through temporary restrictions is achieved at a net cost of $8.40 in unrealized water and sewer revenue for every thousand gallons of *water saved (i.e., adjusted for surcharge revenues and avoided treatment costs). This is nearly three times the cost of achieving the same effective reduction through the purchase of water at a net cost of $3.00 per thousand gallons. (See Option 6 below and* ***Appendices XI and XII*** *for additional details.)*

Option 6: Purchase Water from Neighboring Suppliers

OWASA's existing interconnections provide the capacity to receive a total of about 10 mgd of treated water from Durham, Hillsborough, and Chatham County (Table 1).

Table 1. Interconnection Capacities Among Neighboring Utilities

OWASA to Durham	5 mgd
Durham to OWASA	7 mgd
OWASA to Hillsborough	2 mgd
Hillsborough to OWASA	2 mgd
Chatham County to OWASA	1 mgd
OWASA to Chatham County	0
Durham to Chatham County	3 mgd
Durham to Hillsborough	1 mgd
Durham to Cary *	7 mgd
Cary to Durham *	10 mgd

* Upon completion of projects currently underway

At an assumed cost of $3.00 per thousand gallons (2009), temporary purchases (e.g., from Cary via Durham) offer a cost-effective option compared to the capital-intensive development of a

new supply source. This is especially true for the infrequent and limited amounts of water that OWASA is expected to need during the next 25 or more years. It is more cost-effective to reduce the drawdown of OWASA's reservoirs (equivalent to supplementing the supply) by purchasing available water than by implementing temporary water use restrictions. Purchases offer an additional degree of fiscal control by enabling the supplemental source to be effectively turned on or off as needed, in contrast to temporary restrictions, whose residual effects on OWASA customer behavior (and on water/sewer revenues) have persisted substantially longer than the temporary need to reduce consumption.

If sufficient water is available from other sources, it is recommended in the future that OWASA purchase water before declaring a Water Supply Shortage in order to minimize the inconvenience and potential hardship of additional use restrictions and rate surcharges on OWASA customers and to moderate the financial effects on OWASA.

The OWASA Board of Directors fully intends to keep the community informed of critical water supply issues and will continue to notify its customers and local elected boards in advance of any impending need to purchase water when indicated by reservoir storage and demand conditions. It is expected that such notice would provide additional customer motivation to reduce water use and reinforce our efforts to avoid the need for a Water Supply Shortage declaration with its attendant use restrictions and rate surcharges.

In addition to drought conditions, the ability to receive or send water to other local utilities provides important mutual support during maintenance or failure of critical facilities, extreme weather, or other unforeseen/emergency circumstances. The reliable availability of supplemental water from neighboring utilities requires the development of secure and permanent purchase/sale/conveyance agreements. Additional information about purchases is available in **Appendix XII**.

OWASA is a party to the 2001 *Water and Sewer Management, Planning and Boundary Agreement* adopted by the Towns of Carrboro, Chapel Hill, Hillsborough, and Orange County. In its present form, the Agreement constrains the purchase and sale of water across jurisdictional boundaries and could impede the timely access to OWASA's Jordan Lake water supply allocation, which represents an important "insurance policy" for times of special need. We will work with the signatories to make any modifications to the Agreement that may be needed to resolve unnecessary constraints on our access to Jordan Lake.

It is recommended that OWASA engage in discussions with neighboring utilities about agreements that will secure the permanent ability to purchase water under appropriate conditions of supply and demand. Continued participation in the Jordan Lake Partnership will offer valuable opportunities to explore such arrangements.

It is further recommended that OWASA continue to keep the Carrboro, Chapel Hill, and Orange County elected boards up to date about these efforts and to work cooperatively in amending the 2001 Water and Sewer Management, Planning, and Boundary Agreement as may be needed to ensure a reliable and sustainable future water supply.

Option 7: Expand Cane Creek Reservoir by Constructing a New Dam

This option would provide 5 mgd of additional yield by raising the Cane Creek Reservoir's existing water level by 20 feet, thereby doubling its usable storage from 3 BG to 6 BG. The project would require expanding the capacity of the Cane Creek pumping station and constructing approximately 11 miles of new pipeline parallel to the existing RW line from Cane Creek to the Jones Ferry Road WTP. OWASA's 2001 *Comprehensive Water and Sewer Master Plan* considered accomplishing this with modifications to the existing Cane Creek dam and spillway, but Hazen and Sawyer's recent review determined that "it is unlikely that the existing dam and gated spillway could be modified to increase its structural height by the required amount" **(Appendix XIII)**.

This option would therefore require the construction of a new dam directly downstream of the existing Cane Creek dam and would cost nearly $130 million, or $25 million per mgd. It would involve formidable regulatory, environmental, and political obstacles, including the acquisition of approximately 450 acres of privately owned land, relocation of at least one public road, preparation of an EIS, and NC Environmental Management Commission approval of the right to condemn private property for water supply purposes. This proposal would likely face vigorous public opposition similar to or greater than what was encountered for the original Cane Creek Reservoir project.

No new evidence has been found to improve the low preferential ranking that this option received in OWASA's 2001 Comprehensive Water and Sewer Master Plan. It is recommended that the expansion of the Cane Creek Reservoir not be considered in any future reviews of OWASA water supply options.

Option 8: Expand University Lake by Constructing a New Dam



Existing University Lake and Dam

This option would provide 4.7 mgd of additional yield by increasing University Lake's usable storage from its current capacity of 0.45 BG to 3 BG with a new dam constructed about 400 feet downstream of the existing dam. This would raise University Lake's water level by 22 feet and would require the construction of new intake and pumping facilities to replace existing facilities, which would be inundated upon project completion.

Although it would provide additional yield, expanding University Lake would not add substantial flexibility or redundancy to the overall system if another major supply source or transmission component were out of service. As with the Option 7 (expansion of the Cane Creek Reservoir), implementation would involve formidable regulatory, environmental, and political obstacles, including:

- Permission by UNC and the NC Council of State;
- Permanent inundation of approximately 260 additional acres of land;
- Acquisition of at least 270 additional acres of land, including all or portions of 60 private lots, at least 5 residential buildings, and the UNC Francis Owen animal research facility;
- Five road relocations;
- Relocation of all existing OWASA recreational facilities;
- Impacts to wetlands and other environmentally sensitive lands; and
- Preparation of a detailed Environmental Impact Statement (EIS) as required by the National Environmental Policy Act (NEPA).

Estimated capital costs would exceed $100 million, or $23 million per mgd and are among the highest of the options evaluated. Additional information is available in **Appendix XIV.**

No new evidence has been found to improve the low preferential ranking that this option received in OWASA's 2001 Comprehensive Water and Sewer Master Plan. It is recommended that the expansion of University Lake not be considered in any future reviews of OWASA water supply options.

Option 9: Dredge Sediment from University Lake

This option would involve the removal, stockpiling, dewatering, and relocation of approximately 700,000 cubic yards of accumulated sediment (equivalent to ~125 MG of storage capacity) from University Lake. This would provide an additional yield of only 0.2 mgd and would cost an estimated $37 million, or $180 million per mgd of additional yield. Site constraints and the overall scale of this project would likely require phasing over a period of two or more years and would involve heavy and sustained vehicular traffic for the relocation of sediment removed from the lake. Additional details are available in **Appendix XV**.

Due to the negligible water supply benefits and extremely high financial and environmental costs, it is recommended that dredging accumulated sediment from University Lake not be considered in any future reviews of OWASA water supply options.

Option 10: Construct A Dam and Reservoir on Sevenmile Creek

A reservoir on Sevenmile Creek south of I-85 and west of Hillsborough could provide 4 mgd of additional yield to OWASA, but the financial, legal, regulatory, and political hurdles of this option render it unsuitable for further consideration.

Capital costs would likely exceed $115 million, or $29 million per mgd, including substantial costs for responding to regulatory and legal challenges. A large portion of the 1,000+ acres of land needed for this project is currently owned by Orange County and has been designated for eventual use as a nature preserve. In addition to strenuous public opposition, this project would face a major regulatory hurdle in obtaining interbasin transfer certification from the NC Environmental Management Commission due to the transfer of water from the Neuse to the Cape Fear River Basin. Additional details are available in **Appendix XVI**.

The creation of an OWASA reservoir on Sevenmile Creek is not feasible and should not be considered in any future review of potential water supply options.

Summary of Options

Table 2 presents a qualitative overview of the options, including the benefits and costs over the 50-year planning period as follows:

Adequate Yield: Does the option, by itself, provide enough water to meet customer needs during hypothetical drought conditions under the "Expected" or "High Demand" projections?

Flexibility, Redundancy: Does the option add significant flexibility or reliability to OWASA's overall supply system if another major supply source or transmission component were temporarily out of service?

Financial Cost: Net present cost per thousand gallons needed to satisfy projected water supply deficits during drought conditions. High, Medium, and Low indicate relative values among the options. Estimated dollar values are presented in Table 3, as derived in the OWASA staff spreadsheets included in corresponding Appendices.

Environmental and **Regulatory/Political Challenges:** The High, Medium, and Low indicators reflect the relative overall challenges described in the preceding narrative summaries.

Table 2. Summary of Options, Benefits, and Costs

Option		Adequate Yield?		More Flexibility, Redundancy?	Costs and Challenges		
		Expected Demand	High Demand		Financial	Environ-mental	Regulatory /Political
1A	Expanded Quarry Reservoir (Shallow)	Yes	No	No	Low	Low	Low
1B	Expanded Quarry Reservoir (Deep)	Yes	Yes	No	Medium	Low	Low
2	Jordan Lake (OWASA only)	Yes	Yes	Yes	High	Medium	High
3	Jordan Lake (Partnership)	Yes	Yes	Yes	Medium	Medium	High
4	Haw River to Cane Creek Reservoir	Yes	Yes	Yes	Medium	Medium	High
5A	Expand RCW System	No	No	No	Medium	Medium	Low
5B	Additional Conservation Investments	No	No	No	Medium	Low	Low
5C	Temporary Water Shortage Restrictions	No	No	No	Low	Low	Low
6	Purchase Water from Neighboring Suppliers	Yes	Yes	Yes	Low	Low	Medium
7	Expand Cane Creek Reservoir	Yes	Yes	No	High	High	High
8	Expand University Lake	Yes	Yes	No	High	High	High
9	Dredge University Lake	No	No	No	High	High	High
10	Sevenmile Creek, New Dam and Reservoir	Yes	Yes	Yes	High	High	High

Table 3 on the next page summarizes the net present life-cycle costs of each option over the 50-year planning period assuming the High Demand projections. Additional information is provided in **Appendix I**, with more technical and financial detail in **Appendices V** through **XVI**.

The net present cost of each option is based on the total volume of water needed to meet projected water supply deficits during the 50-year planning period. Costs are expressed as 2009 dollars per 1,000 gallons supplied during the planning period. Water supply deficits were calculated by applying the following assumptions to the "High" future water demand scenarios:

- Existing supply system in place through 2035,
  Yield = 9.8 mgd, with 30% storage reserve
- Expanded quarry (shallow version) in service in 2036
  Yield = 11.8 mgd, with 30% storage reserve
- 2001-02 drought of record recurs 9 times (at 5-year intervals) from 2015-2055
- Customer demands through 2060 follow the High projection scenario
- Deficits represent the difference between projected demands and the system's total operational yield in a given time period

These extremely conservative assumptions represent a precautionary approach for assessing OWASA's long-range water supply options. If the high demands, extreme drought conditions, and associated deficits assumed for this analysis do not occur, then the need for future water supply supplements can be further deferred.

Table 3. Net Present Cost Comparisons (for High Demand and Extreme Drought Scenarios)

Cost Comparison of Supplemental Water Supply Options During Three Projected Phases of System Development						*For "High" Demand Projections*
Supply & Demand Scenario:	**Existing System Operational Yield = 9.8 mgd with a 30% storage reserve. Supply scenarios assume extreme conditions in which the 2001-02 drought recurs 9 times from 2015-2055. Deficits occur in 2030 (0.62 mgd), 2035 (0.92 mgd), 2045 (0.65 mgd), 2050 (1.5 mgd), and 2055 (2.4 mgd), when average annual demands ("High" projection scenario) exceed either the 9.8 mgd operational yield of the Existing System or the 11.8 mgd yield with the Expanded Quarry (shallow version) in service.**					
Supplemental Supply or Demand Reduction Option			**Addi-tional Yield (mgd)**	**PHASE I 2009 - 2035**	**PHASE II 2036 - 2045**	**PHASE III 2046 - 2060**
				Range of Estimated Annual Deficits:		
				0.06 to 0.92 MGD	**0 to 0.65 MGD**	**1.5 to 2.4 MGD**
				Cumulative Need:		
				360 MG	**236 MG**	**1,420 MG**
				Net Present Cost per 1,000 gallons supplied (2009) [A] (Costs highlighted in red indicate that the supply option by itself cannot fully meet the estimated maximum annual deficit.)		
1A	Expanded Stone Quarry (Shallow)	1.5 BG (tot vol)	2.1	***QUARRY OPTIONS NOT AVAILABLE UNTIL 2036***	**$4**	***1.5 BG SHALLOW QUARRY IN SERVICE***
		2.1 BG (tot vol)	2.9		$5	
1B	Expanded Stone Quarry (Deep)	2.2 BG (tot vol)	3.4		$90	$15
		2.6 BG (tot vol)	4.0		$90	$15
		3.0 BG (tot vol)	4.6		$90	$15
2	Jordan Lake, OWASA Only (10 mgd design capacity)	RW to JFR WTP	6.2	$107	$105	$9
		FW to Dist Syst		$299	$325	$26
3	Jordan Lake, Joint Venture (OWASA share: 10 mgd of 65 mgd total design capacity)	RW to JFR WTP	6.2	$78	$76	$7
		FW to Dist Syst		$154	$155	$14
4	Haw River to Cane Creek Reservoir		7.7	$116	$114	$10
5A	Expand Reclaimed Water System 0.25 mgd permanent demand reduction		2.2	**$73**	**$35**	**$16**
5B	Additional Conservation Investments ($75 and $150 fixture rebates) 0.50 mgd permanent demand reduction		0.5	**$17 (w/$75 rebate)**	**$25**	**$15**
				$28 (w/$150 rebate)	**$37**	**$19**
5C	Temporary Water Shortage Restrictions 1.0 mgd temporary demand reduction		1.0	$14	**$18**	**$20**
6	Purchase Water from Neighboring Systems		N/A	$5	$6	$7
7	Expand Cane Creek Reservoir to 6 BG		5.0	$236	$241	$22
8	Expand University Lake to 3 BG		4.7	$200	$204	$18
9	Dredge Sediment from University Lake		0.20	**$248**	**$221**	**$60**
10	New Dam and Reservoir on Sevenmile Creek		4.0	$222	$221	$20

[A] Includes all life-cycle costs (capital, operating, maintenance, replacement, salvage value, etc.) through 2060 per 1,000 gallons pumped if each option were operated to satisfy specified demand conditions. For Option 5C (Stage 1 Water Supply Shortage Restrictions), NPC also includes (unbudgeted) water and sewer net revenue losses incurred during drought years.

SUPPLEMENTAL SUPPLY AND DEMAND REDUCTION OPTIONS

Key Findings

- The significant gains in conservation and water efficiency achieved by OWASA customers since 2002 are equivalent to a 20 to 25 percent "addition" to the community's available water supply. A fundamental assumption of this Plan is that those gains will be sustained in the future.

- Between 1.3 and 1.9 BG of expanded Quarry Reservoir storage will be available by 2035. This will provide between 2.1 and 2.9 mgd of additional yield for an estimated capital investment of less than $2 million, because it will be accessible with existing OWASA pumping facilities. This "shallow quarry" configuration is the most cost-effective of the supplemental supply or demand reductions options evaluated. All anticipated configurations of the expanded Quarry Reservoir could be refilled from the Cane Creek Reservoir through existing pumping and conveyance facilities.

- It is essential that OWASA retain its Jordan Lake allocation. With allowable peak day withdrawals of 10 mgd, our 5 percent storage allocation could provide up to 6.2 mgd of additional yield, due to the optimization benefits of our multiple reservoir system. Access to Jordan Lake as a source of either raw or finished water would add significant flexibility and redundancy to OWASA's overall system if another major supply source or transmission component were out of service.

- Temporary purchases of treated water from neighboring utilities offer the most cost-effective way to supplement our local supply during the infrequent times of severe drought, equipment failure, natural or manmade disaster, or other unforeseen circumstance. Purchasing water also reduces (unbudgeted) revenue losses that result from a declared Water Shortage and would have to be recovered through subsequent rate increases; avoids higher customer bills from surcharges imposed during a declared Water Shortage; and lessens the hardship and inconvenience to customers who have already reduced their water use through diligent and ongoing conservation efforts.

- Permanent facilities to supplement the Cane Creek Reservoir from the Haw River would provide 7.7 mgd of additional yield, but would face daunting regulatory/political, and public acceptance challenges. Rather than a permanent supply source, the Haw River provides a worst-case contingency option; i.e., for the temporary installation of withdrawal, pumping, and above-ground pipeline facilities to the Cane Creek Reservoir if storage in OWASA's existing reservoir/quarry system declined to 20 percent or less during extreme conditions and no other emergency options were available.

- Investing in an expansion of OWASA's RCW system or establishing financial incentive programs, such as plumbing fixture rebates, to promote additional water conservation is less cost-effective than certain other options with additional benefits, and is not recommended at this time.

- The following supply options were found to be unsuitable or non-feasible and should not be considered in future reviews of OWASA's Long-Range Water Supply Plan: dredging sediment from University Lake; expanding either University Lake or Cane Creek Reservoir by building higher dams; and constructing a dam and reservoir on Sevenmile Creek.

Key Actions

- Continue to promote water conservation and efficiency through customer awareness and education, targeted technical assistance, conservation pricing, and support for increased water efficiency standards in new and renovated buildings. The reliability of our local water supply sources assumes that recent gains in water use efficiency will be sustained during the next 50 years.

- Pursue with the University any opportunities for expanding the reclaimed water system that are determined to be cost-effective and beneficial.

- Develop a detailed plan of work for implementing the Expanded Quarry Reservoir option.

- Continue to participate in the Jordan Lake Partnership in order to retain OWASA's water supply storage allocation and ensure cost-effective access to that allocation through secure and permanent agreements with nearby utilities.

- Develop agreements with neighboring utilities to secure the permanent ability to purchase water under appropriate conditions of supply and demand. The Jordan Lake Partnership provides an important opportunity for achieving this. Provide periodic updates to the elected boards of Carrboro, Chapel Hill, and Orange County regarding these activities.

- Work cooperatively with the elected boards of Carrboro, Chapel Hill, and Orange County to modify as necessary the 2001 *Water and Sewer Management, Planning, and Boundary Agreement* to better reflect the important role that water purchases may play in ensuring the long-term reliability and sustainability of our water supply.

SECTION 5
SUMMARY AND RECOMMENDATIONS

This report presents a very positive long-range outlook for OWASA's water supply. Future generations living and working in our community will be able to enjoy a reliable supply of high quality drinking water with far less capital investment than predicted 10 years ago. This report indicates that OWASA has essentially achieved its Water Conservation Goal adopted in 2005:

> *To develop, fund, and implement a cost-effective water conservation and demand management program that will meet our community's long-term water supply needs (through 2050) by making the highest and best use of our local water resources and eliminating the need for costly new water supply sources and facilities.*

This has been accomplished through the combined efforts of OWASA, its customers, and the elected, business, and UNC leadership of the community. Supply and demand management milestones during the past nine years have included:

- Approval of the Quarry Reservoir expansion project
- Year-round conservation standards
- Seasonal and tiered customer water rates
- Water Treatment Plant process water recycling
- OWASA/UNC reclaimed water system
- Increasing use of non-potable and advanced water use efficiency technologies in new development
- Proactive customer education

Primary Recommendations:

1. Continue to promote water conservation and efficiency through customer awareness and education, targeted technical assistance, conservation pricing, and support for increased water efficiency standards in new and renovated buildings. The reliability of our local water supply sources assumes that recent gains in water use efficiency will be sustained during the next 50 years.

2. Continue to pursue the Quarry Reservoir expansion (shallow version) as the most cost-effective, long-term option for a supplemental supply source. This will ensure full local control of a substantial increment of supply with minimal additional capital investment.

3. Continue to participate in the Jordan Lake Partnership in order to retain OWASA's water supply storage allocation and to ensure cost-effective access to Jordan Lake through secure and permanent agreements with nearby utilities. It is essential that OWASA retain its and acquire access to its allocation.

4. Develop water purchase/sale agreements with neighboring utilities that will secure the permanent ability to cost-effectively purchase water under appropriate conditions of supply and demand consistent with the long-term performance objectives of (1) avoiding

future Water Shortage declarations, and (2) providing additional flexibility and redundancy in the event of critical facility failure, extreme weather, or other unforeseen/ emergency circumstances.

5. Work cooperatively with the elected boards of Carrboro, Chapel Hill, and Orange County to amend the 2001 *Water and Sewer Management, Planning, and Boundary Agreement* – only as needed – to resolve any unnecessary constraints on access to OWASA's Jordan Lake allocation and to ensure a reliable and sustainable water supply for the future.

Additional Recommendations:

A. OWASA should purchase water, provided that it is available, before declaring a Water Supply Shortage in the future in order to minimize the inconvenience and potential hardship of additional use restrictions and rate surcharges on customers and to minimize the de-stabilizing financial effects on OWASA. Decisions to purchase water and/or declare Water Supply Shortage restrictions should be based on clearly defined trigger conditions. *OWASA will notify its customers and local elected boards in advance of an impending need to purchase water if reservoir storage and demand conditions do not improve in the near future.*

B. Recognize that OWASA's reservoirs were intended to be drawn down during periods of low inflow. Continue to follow a risk-based approach to drought management and use the "critical drawdown" graphs to develop more detailed triggers for drought management decisions, such as when to purchase water, when to declare a Water Supply Shortage, etc.

C. Retain the option of supplementing Cane Creek with water pumped from the Haw River as a worst-case (temporary) contingency plan for emergency drought conditions, rather than as a permanent supply source.

D. Conduct no further evaluations of the following options, which are not considered to be viable: (a) expansion of University Lake or Cane Creek Reservoir; (b) sediment removal from University Lake; and (c) new dam and reservoir on Sevenmile Creek.

E. Recognize and pursue opportunities for expanding the reclaimed water system that are determined to be cost-effective and beneficial. Evaluate future requests for RCW service on a case-by-case basis, and require that extensions of the RCW system be paid for by benefiting parties in accordance with OWASA's contractual obligations to UNC.

F. Continue to monitor long-term trends in customer demand patterns, reservoir inflows, annual production rates of the active stone quarry, and other information needed to refine demand projections and water supply yield estimates. Demand projections should be systematically reviewed and adjusted to reflect actual observed trends at intervals of approximately every five years.

G. Continue to keep OWASA customers and local elected boards informed of any changes to the assumptions, conditions, or information on which this Water Supply Plan is based.

Appendix II.

OWASA Technical Memorandum:

50-Year Demand Projections

TECHNICAL MEMORANDUM

50-Year Demand Projections

Orange Water and Sewer Authority

Prepared By: OWASA Staff

Date: June 21, 2009 (Final)

Purpose and Scope

Fifty-year demand projections have been prepared to support OWASA's Long-Range Water Supply Plan Update.

The overall purpose of the update is to determine the optimum mix of strategies to ensure a reliable, cost-effective, and sustainable water supply that will meet the needs of Carrboro, Chapel Hill, and the University of North Carolina at Chapel Hill through 2060. During the nine years since OWASA's 50-year supply plan was last revised, two historic droughts have occurred; OWASA's rate structure has been significantly modified; a major reclaimed water (RCW) system has begun operation; and customer consumption patterns have changed markedly. The current comprehensive update recognizes the need to revisit the underlying assumptions – especially in view of such substantially changed conditions – and the relative benefits and costs of supply and demand management options that were evaluated previously, as well as others that have not yet been considered.

The demand projections described below comprise a range of population growth, density, development, and water consumption scenarios that may occur during the 50-year planning period. Key assumptions, methods, and results of these analyses are presented in Attachments 1 through 4.

Attachment 1: Supply and Demand Graph

The supply and demand graph depicts the following:

- Average annual water withdrawals from OWASA's reservoirs from 1980 through 2008 (black diamonds).
- Operational yield of OWASA's existing University Lake/Cane Creek/Quarry Reservoir system, plus additional yield available with 1.3 billion gallons (BG) of expanded Quarry Reservoir storage (1.3 BG = additional water volume available after 2030 per American Stone Company's minimum production commitment. This water would be accessible via existing OWASA pumping facilities; i.e., with little or no need for major capital improvements). The upper margin of each green rectangle represents total system yield calculated from 82 years of historic streamflow data. The lower margins are calculated on the basis of a 30% reduction of actual streamflow in order to approximate future hydrologic

conditions that might result from major changes in weather patterns and/or watershed land use/land cover. All yield calculations assume that 20% of usable reservoir storage is held in reserve for extreme (emergency) drought conditions. It is important to note that these yield estimates differ from those in previous OWASA studies, which based reservoir yields on total system storage, rather than a 20% reserve.

The yield estimates depicted in Attachment 1 and elsewhere in the Water Supply Plan Update represent the reservoir withdrawal rate that could be sustained ***while still maintaining a 20% emergency storage reserve*** *during the 2001-2002 drought, which was the worst drought of record for the OWASA system. Previous yield estimates were based on complete reservoir drawdown; that is, with no water left in reserve.*

- High, Low, and "Expected" demands through 2060 (red banded area) are described in the Assumptions listed in **Attachment 1a**.

Long-term demand projections are substantially less than reported in previous OWASA analyses, due primarily to system-wide increases in water use efficiency among existing customers. This trend has been reinforced by the historic droughts of 2001-2002 and 2007-2008, fundamental changes in OWASA's rate structure, and initiation of the OWASA/UNC RCW system. The change in customer behavior appears to be permanent and is expected to continue. The supply and demand graph in Attachment 1 indicates that OWASA's current supply system may meet the community's water supply needs substantially further into the future than previously thought.

Attachment 2: Sensitivity of Demand Projections to Different Assumptions

Attachment 2 summarizes the "what-if" effects of modifying selected assumptions about community growth and water use on long-term (2060) demands.

Attachment 3: Previous OWASA Demand Projections

The graph in Attachment 3 compares actual reservoir withdrawals (1965–2008) to historical OWASA demand projections from as far back as 1969, which comprised a range of projection methods. The 1969 and 1982 estimates simply multiplied undocumented *per capita* consumption rates (typically in the range of 90—125 gallons per person per day) by state or locally-generated service area population projections. The 1987 and 1998 projections extrapolated existing trends by fixed or varying percentages, which yielded growth curves that expanded exponentially. The 2000, 2008, and 2009 projections employed increasingly sophisticated *disaggregated demand* methods, which attempt to project water use by specific customer sectors, such as residential, commercial, University, etc. The current (2009) projections described in this report, are based on this approach.

Attachment 4: Table of Demand Projections

Attachment 4 is simply a tabular presentation of the High, Low, and Expected demand projections shown graphically in Attachment 1, as well as the corresponding RCW inputs and billed water projections calculated by the spreadsheet. The "WWTP" column includes projected wastewater treatment plant flows associated with the respective water demands. These were calculated from the consistent ratio (92.4%) observed between annual WWTP flow and water treatment plant (WTP) production as adjusted for RCW demands.

General Observations and Comments

1. Raw water demand projections for **2060** range from 10.8—15.0 mgd, which is substantially less than the 14.6—16.6 mgd previously estimated for **2050**.

2. This reduction is due primarily to the observed 20—25% increase in water efficiency that has been sustained among all sectors of OWASA water use in recent years. For example:

   - Total UNC building space increased by 26% from 1998-2008, but total water use decreased by 6% during the same period.

   - Average single family water use, which remained stable at 6,000 ± 200 gallons per month through 2001, declined to 4,700 gallons per month during the four-year period of 2004—2007, and declined further to 4,200 gallons per month in 2008.

   These changes appear to be permanent and should be considered to represent "new normal" consumption patterns.

3. Except as noted in the specific Assumptions listed in Attachment 1, it is assumed that overall community growth will return to pre-recessionary development rates and patterns within the next few years and continue in a linear fashion through 2060. Although this likely represents an unrealistically aggressive long-term growth scenario, no long-term forecasts are available beyond 2035 in the adopted plans of Carrboro, Chapel Hill, or Orange County.

Implications for OWASA's Long-Term Water Supply Plan

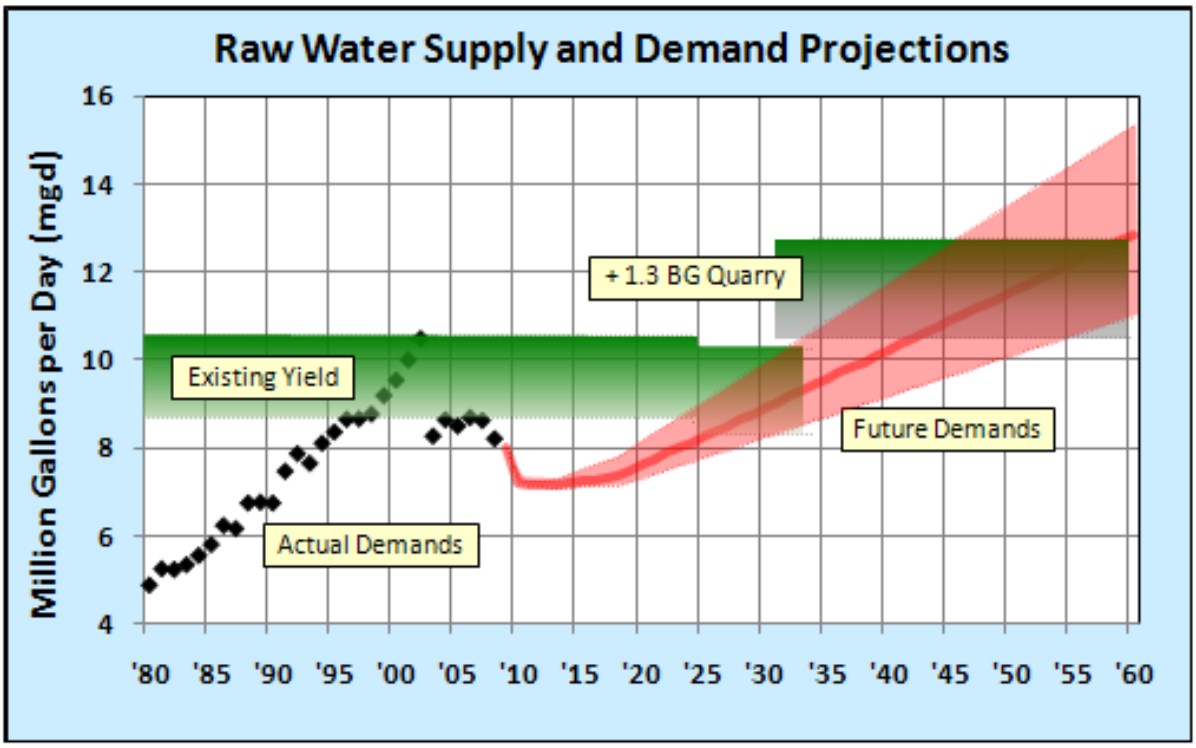
Demand projections presented in this report suggest that OWASA's current University Lake/Cane Creek/Quarry Reservoir supply system may meet the community's water supply needs substantially further into the future than previously thought. Supplementary needs that could arise may not require major capital projects, but might be met through less costly arrangements, such as inter-local water purchases or other partnership opportunities with neighboring communities.

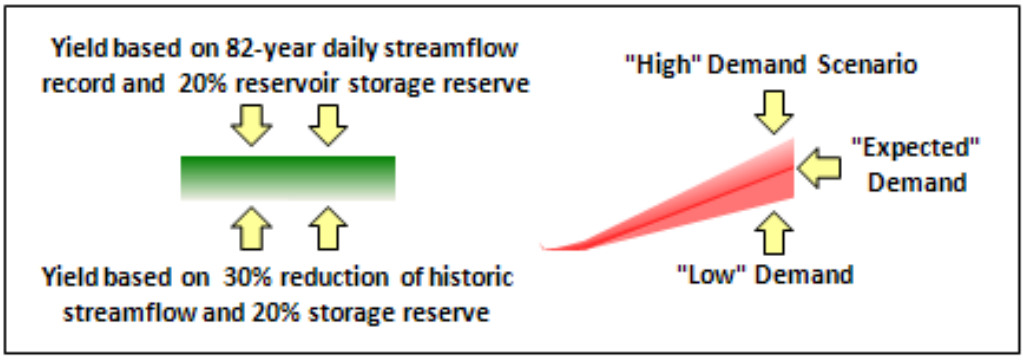
The economic and technical details of a full array of supply and demand management options are being evaluated in other individual Technical

Memoranda and will be incorporated into the overall recommendations of OWASA's Long-Range Water Supply Plan Update.

Attachment 1.

(Includes November 17, 2009 Adjustments)





Attachment 1a. Basis of Demand Projections

Key Assumptions for "Expected" Demand Scenario

1. No changes to OWASA service area boundaries.
2. Pace and rate of local development will return to 20-year average of ~563 new meter equivalents (MEs) per yr within the next 2 to 3 years and will continue at this linear rate through 2060.
3. Development profile will resemble recent and future trends as follows:

	% of 2008 Demand	New MEs per year [A]	gal/mo/ME	% of New Demand
Single Family Detached	34%	120	4,700	14%
Townhouse/Condos	5%	240	3,400	20%
Multifamily (master metered)	19%	68	14,400	24%
Non-Residential (non-UNC)	13%	61	9,900	15%
UNC	24%	35	21,500	7%
UNC Hospitals	5%	8	37,900	3%
Carolina North	0%	31	23,300	17%
Totals	100%	563	8,400	100%

[A] *563 new MEs (meter equivalents) each year through 2028 (UNC Central Campus buildout); 521 new MEs per year* from 2029 through 2060.

This profile represents an average annual increase in billed water demand of 0.155 mgd/yr.

4. Future consumption rates (gal/mo/ME) based on averages observed from 2004-2007, when annual demand remained stable (consumption during 2008 was actually 11% less than the preceeding 4 years)
5. Raw Water demand = 114% x Billed Water ≈ Water Treatment Plant (WTP) Production
6. WTP process water (6-8% of total production) will continue to be recycled.
7. UNC Central Campus and UNC Hospitals building space will increase by 4.6 million gross square feet (GSF), per the *2006 UNC Campus Master Plan* , at a constant rate of ~0.16 million GSF/yr through 2028. No further increase after 2028.
8. Carolina North will build out to a total of 8 million GSF at a constant rate of 0.17 million GSF/yr from 2014 through 2060. Until more detailed information is available, demand projections assume the same mix of uses and water consumption (gallons per day per gross square foot = gpd/GSF) as observed on the Main Campus/Hospitals. "Expected" projection assumes that 25% of total water needs will be met with reclaimed (RCW) or other non-potable water (NPW).
9. RCW demand by UNC/UNC Hospitals will increase through 2028 per previous UNC Energy Services estimates.

"Lower" Demand Assumptions

Same assumptions as "Expected" demand scenario, except for the following:

1. 15% less residential development through 2060.
2. 15% less non-residential (non-UNC) development.
3. 50% (rather than 25%) RCW/NPW use at Carolina North.
4. Additional conservation occurs at the following rates:

	Additional Demand Reduction per Year	**Total Reduction Through 2060**
All existing non-UNC customers	0.10%	5%
All new non-UNC customers	0.25%	12%
All UNC (Central Campus + Hospitals) users	0.15%	7%

"Higher" Demand Assumptions

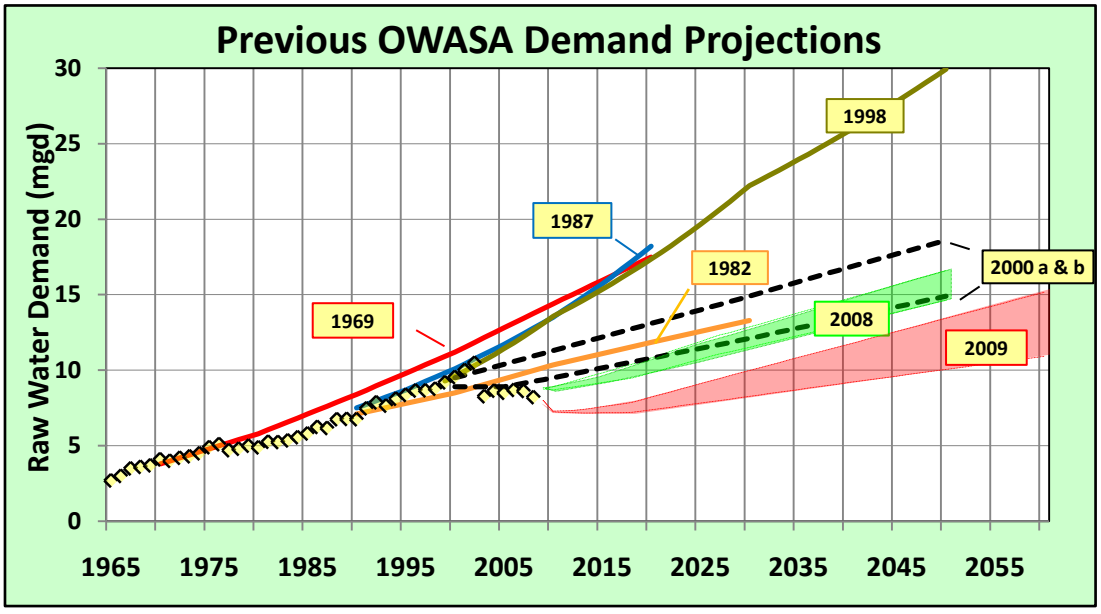
Same assumptions as "Expected" demand scenario, except for the following:

1. One new high density mixed use development (East 54 scale, demand equivalent to ~150 new SF homes) added each year through 2060.
2. Additional non-residential (non-UNC) development through 2060 is 25% greater than expected.
3. UNC Central Campus and UNC Hospitals growth is 25% greater than expected through 2028.
4. Carolina North water demand is 25% higher than expected; no use of RCW or NPW.

Attachment 2. Sensitivity of Demand Projections to Different Growth Assumptions

"What-If" Adjustments to Expected Demand Scenario	Effect on 2060 Raw Water Demand
Change consumption by 1,000 gal/mo/meter equivalent (ME) for all new single family detached units	±0.22 mgd
Change annual growth rate for all new single and multifamily residential development by 10%	±0.42 mgd
Add 1 new "East 54"-scale mixed use development project each year. Assume 6 2-in meters (6x8 = 48 MEs @ 15,000 gal/mo/ME) *[equivalent to ~150 new single homes/yr]*	+1.33 mgd
Add 1 new "Greenbridge"-scale condo project each year. One 4-inch meter = 25 MEs @ 13,000 gal/mo/ME	+0.60 mgd
Change Carolina North consumption by 50% (± 11,700 gal/mo/ME)	±0.47 mgd
Meet a portion of Carolina North potable demand with reclaimed (RCW) or other non-potable (NPW) water	-0.13 mgd per each 10% increment of demand met with RCW/NPW
Change UNC & UNC Hospital Growth Rate by 10%	±0.08 mgd per 10% increment
Additional conservation by existing (pre-2010) customers:	-0.40 mgd per each 0.10% annual increment
Additional conservation by new (2010-2060) customers:	-0.15 mgd per each 0.10% annual increment

Attachment 3. Comparison to Previous Projections



1969: *Report to the the University of North Carolina on Water Supply for Chapel Hill, North Carolina,* Hazen and Sawyer Engineers, August 1969.

1982: *Final Cane Creek EIS,* NC Department of Natural Resources and Community Development, Division of Environmental Management, September 1982.

1987: *Main Report, Eno River Capacity Use Investigation,* NC Department of Natural Resources and Community Development, December 1987.

1998: *UPDATE 98.xls,* unpublished Microsoft Excel spreadsheet file, Orange Water and Sewer Authority, 1998, Ed Holland, Planning Director.

2000a: "Baseline" projection, from *Comprehensive Water and Sewer Master Plan, Technical Memorandum 3.3, Long-Term Demand Forecasts with Conservation,* CH2MHill, January 21, 2000.

2000b: "Active Conservation" projection, from *Comprehensive Water and Sewer Master Plan, Technical Memorandum 3.3, Long-Term Demand Forecasts with Conservation,* CH2MHill, January 21, 2000.

2008: "Higher Conservation, Greater Reuse" and "Less Conservation, Less Reuse" demand projections, from *Annual Review and Update of Strategic Trends and Master Plan Issues,* staff memorandum to OWASA Board of Directors, October 17, 2008.

2009: OWASA Projections, May 29, 2009, staff draft.

Attachment 4. Raw Water Demand and Wastewater Flow Projections

(Includes November 17, 2009 Adjustments to FY '10 Billed and Unbilled Water Assumptions)

"Expected" Demands, Average Day (mgd)				
FY	Raw Water	Billed	RCW	WWTP
2010	7.18	6.30	0.66	7.33
2011	7.16	6.28	0.77	7.43
2012	7.15	6.27	0.89	7.54
2013	7.16	6.28	1.00	7.67
2014	7.20	6.32	1.12	7.83
2015	7.24	6.35	1.24	7.99
2016	7.28	6.39	1.36	8.16
2017	7.32	6.42	1.48	8.32
2018	7.37	6.46	1.60	8.49
2019	7.49	6.57	1.64	8.65
2020	7.62	6.69	1.68	8.82
2021	7.75	6.80	1.73	8.98
2022	7.88	6.91	1.77	9.14
2023	8.01	7.02	1.81	9.31
2024	8.13	7.14	1.86	9.47
2025	8.26	7.25	1.90	9.64
2026	8.39	7.36	1.94	9.80
2027	8.52	7.47	1.99	9.96
2028	8.65	7.58	2.03	10.13
2029	8.78	7.70	2.04	10.26
2030	8.91	7.82	2.04	10.39
2031	9.04	7.93	2.05	10.51
2032	9.17	8.05	2.06	10.64
2033	9.31	8.16	2.06	10.77
2034	9.44	8.28	2.07	10.90
2035	9.57	8.40	2.07	11.03
2036	9.70	8.51	2.08	11.16
2037	9.83	8.63	2.09	11.28
2038	9.97	8.74	2.09	11.41
2039	10.10	8.86	2.10	11.54
2040	10.23	8.97	2.10	11.67
2041	10.36	9.09	2.11	11.80
2042	10.49	9.21	2.12	11.93
2043	10.63	9.32	2.12	12.06
2044	10.76	9.44	2.13	12.18
2045	10.89	9.55	2.14	12.31
2046	11.02	9.67	2.14	12.44
2047	11.15	9.79	2.15	12.57
2048	11.29	9.90	2.15	12.70
2049	11.42	10.02	2.16	12.83
2050	11.55	10.13	2.17	12.95
2051	11.68	10.25	2.17	13.08
2052	11.82	10.36	2.18	13.21
2053	11.95	10.48	2.18	13.34
2054	12.08	10.60	2.19	13.47
2055	12.21	10.71	2.20	13.60
2056	12.34	10.83	2.20	13.73
2057	12.48	10.94	2.21	13.85
2058	12.61	11.06	2.21	13.98
2059	12.74	11.17	2.22	14.11
2060	12.87	11.29	2.23	14.24

"Lower" Demand Projections (mgd)				
FY	Raw Water	Billed	RCW	WWTP
2010	7.17	6.29	0.66	7.32
2011	7.13	6.26	0.77	7.40
2012	7.10	6.23	0.89	7.50
2013	7.08	6.21	1.00	7.60
2014	7.09	6.22	1.12	7.74
2015	7.10	6.23	1.25	7.88
2016	7.11	6.24	1.37	8.02
2017	7.12	6.24	1.50	8.15
2018	7.12	6.25	1.63	8.29
2019	7.22	6.33	1.67	8.43
2020	7.31	6.41	1.72	8.57
2021	7.40	6.49	1.77	8.71
2022	7.49	6.57	1.82	8.84
2023	7.59	6.65	1.87	8.98
2024	7.68	6.73	1.92	9.12
2025	7.77	6.81	1.97	9.26
2026	7.86	6.89	2.02	9.39
2027	7.95	6.97	2.07	9.53
2028	8.04	7.05	2.12	9.66
2029	8.14	7.14	2.13	9.76
2030	8.23	7.22	2.14	9.87
2031	8.33	7.30	2.16	9.97
2032	8.42	7.39	2.17	10.07
2033	8.52	7.47	2.18	10.17
2034	8.61	7.55	2.19	10.27
2035	8.70	7.64	2.21	10.37
2036	8.80	7.72	2.22	10.47
2037	8.89	7.80	2.23	10.57
2038	8.99	7.88	2.24	10.66
2039	9.08	7.96	2.25	10.76
2040	9.17	8.04	2.27	10.86
2041	9.26	8.13	2.28	10.96
2042	9.36	8.21	2.29	11.06
2043	9.45	8.29	2.30	11.16
2044	9.54	8.37	2.32	11.26
2045	9.63	8.45	2.33	11.35
2046	9.72	8.53	2.34	11.45
2047	9.82	8.61	2.35	11.55
2048	9.91	8.69	2.37	11.65
2049	10.00	8.77	2.38	11.74
2050	10.09	8.85	2.39	11.84
2051	10.18	8.93	2.40	11.94
2052	10.27	9.01	2.41	12.03
2053	10.36	9.09	2.43	12.13
2054	10.45	9.17	2.44	12.23
2055	10.54	9.25	2.45	12.32
2056	10.63	9.32	2.46	12.42
2057	10.72	9.40	2.48	12.51
2058	10.81	9.48	2.49	12.61
2059	10.90	9.56	2.50	12.70
2060	10.99	9.64	2.51	12.80

"Higher" Demand Projections (mgd)				
FY	Raw Water	Billed	RCW	WWTP
2010	7.21	6.32	0.66	7.36
2011	7.23	6.34	0.77	7.49
2012	7.26	6.37	0.89	7.64
2013	7.30	6.41	1.00	7.80
2014	7.40	6.49	1.11	8.01
2015	7.50	6.58	1.23	8.22
2016	7.60	6.66	1.34	8.43
2017	7.69	6.75	1.45	8.64
2018	7.79	6.84	1.57	8.85
2019	7.98	7.00	1.60	9.06
2020	8.16	7.16	1.64	9.27
2021	8.34	7.32	1.68	9.48
2022	8.53	7.48	1.72	9.69
2023	8.71	7.64	1.75	9.90
2024	8.90	7.80	1.79	10.11
2025	9.08	7.97	1.83	10.32
2026	9.27	8.13	1.87	10.53
2027	9.45	8.29	1.90	10.74
2028	9.63	8.45	1.94	10.95
2029	9.81	8.61	1.94	11.11
2030	9.99	8.76	1.94	11.28
2031	10.17	8.92	1.94	11.44
2032	10.35	9.08	1.94	11.61
2033	10.53	9.23	1.94	11.77
2034	10.71	9.39	1.94	11.94
2035	10.88	9.55	1.94	12.10
2036	11.06	9.70	1.94	12.27
2037	11.24	9.86	1.94	12.43
2038	11.42	10.02	1.94	12.60
2039	11.60	10.17	1.94	12.76
2040	11.78	10.33	1.94	12.93
2041	11.96	10.49	1.94	13.09
2042	12.13	10.64	1.94	13.26
2043	12.31	10.80	1.94	13.42
2044	12.49	10.96	1.94	13.59
2045	12.67	11.11	1.94	13.75
2046	12.85	11.27	1.94	13.92
2047	13.03	11.43	1.94	14.08
2048	13.21	11.58	1.94	14.25
2049	13.38	11.74	1.94	14.41
2050	13.56	11.90	1.94	14.58
2051	13.74	12.05	1.94	14.74
2052	13.92	12.21	1.94	14.91
2053	14.10	12.37	1.94	15.07
2054	14.28	12.52	1.94	15.24
2055	14.46	12.68	1.94	15.40
2056	14.64	12.84	1.94	15.57
2057	14.81	12.99	1.94	15.73
2058	14.99	13.15	1.94	15.90
2059	15.17	13.31	1.94	16.06
2060	15.35	13.46	1.94	16.23

ORANGE WATER AND SEWER AUTHORITY
Quality Service Since 1977

April 9, 2009

Mr. Roger Stancil
Chapel Hill Town Manager
405 Martin Luther King Jr. Blvd.
Chapel Hill, NC 27514

SUBJECT: POTENTIAL IMPACTS OF CAROLINA NORTH ON OWASA'S FUTURE OPERATIONS

Dear Roger:

Per your request, this letter summarizes our current estimate of the impacts that the Carolina North development may have on future OWASA operations. Substantial aspects of Carolina North planning are still at the concept level; therefore, our analysis is preliminary and subject to change. We will remain engaged with University and Town representatives as plans are refined and formalized in order to ensure OWASA's ability to meet its commitments to the University and all other customers.

Legal Requirements – There is no question about our legal requirement to provide service to Carolina North, as OWASA's original Sale and Purchase Agreement with the University clearly outlines our overall responsibility to "furnish, for as long as the University shall desire, potable water in such quantity as may be required to meet its needs."

Developer Requirements – Carolina North will be subject to the same cost-of-service policies, procedures, and fees that apply to any development project seeking OWASA services. Consistent with the principle that "growth pays for growth," the University (or perhaps other third party developers) will pay for the extension or expansion of certain OWASA facilities needed to serve the project. For all new connections to the water and/or sewer system, the University will be required to pay all applicable availability fees at the rates in effect at the time of the connection, just as any other party is required to do.

Anticipated Utility Needs – The University envisions meeting the needs of Carolina North through the integrated use of all available water resources, including rainwater recovered from building rooftops, parking lots, and other impervious surfaces; on-site wastewater treatment to provide a source of reclaimed water, perhaps supplemented by domestic wastewater "mined" from a nearby OWASA sewer line and treated for non-potable use. These waters would be used conjunctively for irrigation, toilet flushing, cooling-tower makeup water, and other non-potable purposes at Carolina North. We expect that certain innovative or non-traditional water supply/wastewater facilities would ultimately be owned and operated by OWASA, but the details of such arrangements have not been developed.

Water Demands – OWASA incorporated the potential needs of Carolina North in its 2001 *Comprehensive Water and Sewer Master Plan,* which included a "placeholder" demand of 1.6 million gallons per day (mgd) of potable water when the site is fully developed. This estimate assumed that all water needs, including non-potable uses, would be met with treated drinking water; that is, the placeholder estimate assumed no demand reduction through the use of reclaimed or other alternative water sources described above.

Additionally, the 1.6 mgd estimate assumed that demand patterns at Carolina North would be similar and proportional to those on the main UNC campus. However, as required by Session Law 2008-203 and as encouraged by our conservation pricing structure, new buildings at Carolina North will certainly be much more water-efficient than older buildings on the main campus. (SL 2008-203 requires new State buildings to be at least 20% more efficient with indoor water use than the 2006 Plumbing Code requires, and at least 50% more efficient with potable water use for landscaping.)

Recent analyses indicate that water use by the University and UNC Hospitals during 2008 *was the same as it was in 1998, despite a 26 percent increase in total building space that has occurred during that period.* This represents a remarkable improvement in water efficiency. Applying the 2008 UNC/UNC Hospitals consumption rate of 0.123 gallons per day (gpd) per gross square foot (GSF) of total building space to the 8 million GSF anticipated for Carolina North yields an estimated average day water demand of 1.0 mgd, which is substantially less than our 1.6 mgd placeholder. *This estimate still includes no assumptions about further reductions that may be achieved with captured rainwater, reclaimed wastewater, and so forth.* The ultimate effects of Carolina North demands on OWASA's water supply will be driven by a number of additional factors which have not yet been determined, but it appears that the impact will be substantially less than our placeholder estimate of 1.6 mgd.

Wastewater – It is unlikely that wastewater generated at Carolina North will have significant hydraulic effects on OWASA's Mason Farm Wastewater Treatment Plant, especially if the innovative technologies envisioned for the site are implemented; however, additional analysis and evaluation will be needed as Carolina North's utility plans are more fully developed. All sewer connections will be subject to OWASA's sewer use ordinance and pre-treatment requirements.

Reclaimed Water – The recently completed OWASA/UNC reclaimed water (RCW) system will provide highly treated wastewater from the Mason Farm Wastewater Treatment Plant for major non-potable uses on the University's main campus. Preliminary analyses have indicated that RCW at Carolina North may be best provided through a separate satellite RCW system. We are pleased that the concept plans for Carolina North incorporate a site for a potential RCW treatment facility. OWASA and the University will work together in determining the most appropriate solution for meeting Carolina North's RCW needs.

Future Actions – OWASA will continue to work closely with the University in determining:

- Ultimate demand projections for Carolina North to ensure that OWASA's long term master planning addresses any capital investments that may be needed before additional capability is required;
- Potential impacts of innovative technologies at Carolina North on OWASA's Mason Farm Wastewater Treatment Plant; e.g., nutrient and other operational effects of "sewer mining" on the flow of wastewater solids returned to OWASA's sewer system; and
- Ownership and maintenance responsibilities for any satellite wastewater or other alternative/innovative treatment systems that may be employed at Carolina North.

Summary – Carolina North will present unique but manageable opportunities and challenges to OWASA, which we expect to manage largely through existing policies and procedures. Based on UNC's demonstrated improvements in water efficiency and its visionary plans for integrated water management, Carolina North's water supply needs are well within OWASA's long range planning projections. New arrangements may be needed for the operation and management of certain innovative/alternative water-related technologies, but we do not foresee any undesired water and/or sewer utility impacts of Carolina North on OWASA's existing or future customer base.

We look forward to our continued partnership with the University and other community stakeholders as we develop and implement the necessary plans for meeting the water, wastewater and reclaimed water needs of Carolina North in a sustainable manner.

If you or your staff have any questions or would like any additional information, please let me know.

Sincerely,

Ed Kerwin

Ed Kerwin
Executive Director

c: Ms. Laura Blackmon, Orange County Manager
Mr. Steve Stewart, Carrboro Town Manager
Ms. Carolyn Elfland, Associate Vice-Chancellor for Campus Services, UNC-Chapel Hill
Mr. Jack Evans, Executive Director, UNC Carolina North Project
OWASA Board of Directors

Orange Water and Sewer Authority (OWASA)
Mason Farm WWTP
Hydraulic Profile

		Flow Condition			
		Average Daily Flow	Max Month Flow	Peak WW Flow (3Q)	Peak Flow One Unit Out
Plant Influent Flow	**mgd**	**10.35**	**14.5**	**Varies**	**Varies**
Return Activated Sludge (RAS) Flow	**mgd**	**10.35**	**14.5**	**14.5**	**14.5**

SEGMENT A: UV DISINFECTION TO OUTFALL

Morgan Creek WSEL (Assume Creek Level at Top of Pipe)	**249.00**	**249.00**	**249.00**	**249.00**

Outfall Pipe - Gravity Flow

Outfall Pipe
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	60	in		Q (mgd) =	10.35	14.50	101.00
Length =	138	ft		V (fps) =	0.82	1.14	7.96
Hazen & Will. C =	120			Hv (ft) =	0.01	0.02	0.98
Minor Loss K's :	Entr (1) / Exit (1)	1.5		Hf (ft) =	0.01	0.01	0.42
	Flap Gate	0.5		Hm (ft) =	0.02	0.05	2.36
	45 Bends	0.4		HL (ft) =	0.03	0.06	2.78
	Sum K =	2.4					
	HGL at end of final effluent channel				**249.03**	**249.06**	**251.78**

Final Effluent Channel
Module OCR *Short Rectangular Open Channel*

Invert El =	246.00		Q (mgd) =	10.35	14.5	101.00
Width =	8.00	ft	Backwater (ft) =	3.03	3.06	5.78
Manning n =	0.015		Yc (ft) =	0.50	0.63	2.28
Length =	62	ft	Yn (ft) =	1.17	1.50	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A
K-ent =			Area (sf) =	9.39	11.97	12.33
Downstream Conditions			Actual Depth (ft) =	3.03	3.06	5.78
			Area (sf) =	24.25	24.48	46.25
			V (fps) =	0.66	0.92	3.38
			Sf (ft/ft) =	0.0000	0.0000	0.0004
Upstream Conditions			Upstrm Depth (ft) =	3.03	3.06	5.81
			Iterate 0 >>>	0.000	0.000	0.000
			Check >>>	ok	ok	ok
			Area (sf) =	24.26	24.50	46.45
			V (fps) =	0.66	0.92	3.36
			Sf (ft/ft)=	0.0000	0.0000	0.0004
			Hm (ft) =	0.00	0.00	0.00
HGL downstream of post aeration tank port				**249.04**	**249.08**	**251.98**
Final Effluent Weir Elev 252.00				**ok**	**ok**	**ok**

Outfall Pipe - Pumped Flow

				Pump Capacity (each) =			17.5	17.5
				No. of Pumps Operating =			4	3
				Effluent Flow (mgd) =			70.00	52.50
	100-Yr Flood Level in Morgan Creek						**262.00**	**262.00**
Outfall Pipe								
Module PIPH	*Piping by Hazen-Williams Equation*							
Diameter =	60	in		Q (mgd) =			70.00	52.50
Length =	138	ft		V (fps) =			5.52	4.14
Hazen & Will. C =	120			Hv (ft) =			0.47	0.27
Minor Loss K's :	Entr (1) / Exit (1)	1.5		Hf (ft) =			0.21	0.12
	Flap Gate	0.5		Hm (ft) =			1.13	0.64
	45 Bends	0.4		HL (ft) =			1.35	0.76
	Sum K =	2.4						
	HGL at end of final effluent channel						**263.35**	**262.76**
Effluent Pump Discharge Channel Drop Box								
Module CCR	*Rectangular Closed Conduit by Mannings Equation*							
Width =	8	ft		Q (mgd) =			70.00	52.50
Height =	8	ft		V (fps) =			1.69	1.27
Length =	12.00	ft		Hv (ft) =			0.04	0.03
Manning n =	0.015			Hf (ft) =			0.00	0.00
SumK =	1.5			Hm (ft) =			0.07	0.04
				HL (ft) =			0.07	0.04
	HGL in Filter Sump Trench						**263.42**	**262.80**
Effluent Pump Discharge Channel								
Module OCLI	*Rectangular Lateral Inflow Channel*							
Invert El =	260	ft		Q (mgd) =			70.00	52.50
Width =	8	ft		Backwater (ft) =			3.42	2.80
Length =	52.00	ft		Yc (ft) =			1.78	1.47
Slope =	0.0000	ft/ft		Actual Depth (ft) =			3.42	2.80
Manning n =	0.015			V-end (fps) =			3.96	3.62
				Hf (ft) ~			0.01	0.01
				Yu (ft) =			3.88	3.19
	Max HGL in Pump Discharge Channel						**263.88**	**263.19**
				Segment Flow =	10.35	14.5	70.00	52.50
Final Effluent Weir								
Module WS	*Sharp Crested Weir*							
Weir Elev =	252.00	ft		Q (mgd) =	10.35	14.5	70	52.5
Length =	20	ft		H_w (ft) =	0.39	0.49	1.39	1.15
C_w =	3.3			Weir Submergence (H_2)=	0.00	0.00	0.00	0.00
				<If Subm> H_1 (ft) =	1.00	1.00	1.39	1.00
				<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
				Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
	HGL upstream of final effluent weir				**252.39**	**252.49**	**253.39**	**253.15**

Channel from Post Aeration Tank #2 to Final Effluent Channel

Module OCR *Short Rectangular Open Channel*

Invert El =	232.00		Q (mgd) =	5.175	7.25	35	52.5
Width =	7.00	ft	Backwater (ft) =	20.39	20.49	21.39	21.15
Manning n =	0.015		Yc (ft) =	0.34	0.43	1.23	1.61
Length =	12.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	8.22	10.48	10.79	10.79
			Actual Depth (ft) =	20.39	20.49	21.39	21.15
Downstream Conditions			Area (sf) =	142.72	143.41	149.74	148.04
			V (fps) =	0.06	0.08	0.36	0.55
			Sf (ft/ft) =	0.0000	0.0000	0.0000	0.0000
Upstream Conditions			Upstrm Depth (ft) =	20.39	20.49	21.39	21.15
			Iterate 0 >>>	0.000	0.000	-596.264	0.000
			Check >>>	ok	ok	Solve	ok
			Area (sf) =	142.72	143.41	149.75	148.04
			V (fps) =	0.06	0.08	0.36	0.55
			Sf (ft/ft)=	0.0000	0.0000	0.0000	0.0000
			Hm (ft) =	0.00	0.00	0.00	0.00
HGL downstream of post aeration tank port				**252.39**	**252.49**	**253.40**	**253.16**

Post Aeration

# Post Aeration Tanks in Service =	**2**	**2**	**2**	**1**
Flow per Tank (mgd) =	5.175	7.25	35	52.5

Post Aeration Tank Effluent Port

Module SPR *Submerged Rectangular Port*

Heighth =	4.5	ft	Q (mgd) =	5.175	7.25	35	52.5
Width =	4.5	ft	V (fps) =	0.40	0.55	2.67	4.01
Length =	18	in	HL (ft) =	0.01	0.01	0.27	0.60
No. Suppressed Cont =	0	(0 - 3)					
L/P =	0.083						
C_d =	0.65						
HGL at post aeration tank				**252.39**	**252.50**	**253.66**	**253.75**

UV Disinfection

UV Effluent Channel Ports (if Unsubmerged)

Module FPR *Free Surface Rectangular Port*

Invert El =	248 ft	Q (mgd) =	5.175	7.25	35	52.5
Width=	6 ft	Backwater (ft) =	4.39	4.50	5.66	5.75
Length =	18 in	Yc (ft) =	0.38	0.48	1.36	1.78
No. Suppressed Cont =	1 (0 - 3)	Act. Depth (ft) =	4.39	4.50	5.66	5.75
		L/P =	0.10	0.10	0.09	0.09
		C_d =	0.67	0.67	0.66	0.66
		V (fps) =	0.30	0.42	1.59	2.35
		HL (ft) =	0.00	0.01	0.09	0.20
	HGL at UV effluent channel (if Free Surface)		**252.40**	**252.50**	**253.75**	**253.95**

UV Effluent Channel Port (If Submerged)

Module SPR *Submerged Rectangular Port*

Heighth =	5 ft	Q (mgd) =	5.175	7.25	35	52.5
Width =	6 ft	V (fps) =	0.27	0.37	1.80	2.71
Length =	18 in	HL (ft) =	0.00	0.01	0.12	0.27
No. Suppressed Cont =	1 (0 - 3)					
L/P =	0.068					
C_d =	0.65					
	HGL at UV effluent channel (if Submerged)		**252.40**	**252.50**	**253.78**	**254.02**
	Actaul HGL in UV Effluent Channel		**252.40**	**252.50**	**253.78**	**254.02**

UV Effluent Weir Gate

Module WS *Sharp Crested Weir*

		# UV Channels in Service =	**1**	**1**	**3**	**2**
Gate Width =	6.75 ft	Q (mgd) =	10.35	14.5	23.33	26.25
C_w =	3.3	Top of Gate Elev =	255.83	255.83	255.09	255.00
		H_w (ft) =	0.80	1.00	1.38	1.49
		Weir Submergence (H_2)=	0.00	0.00	0.00	0.00
		<If Subm> H_1 (ft) =	1.00	1.00	1.00	2.23
		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
		Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
	HGL upstream of final effluent weir		**256.63**	**256.83**	**256.47**	**256.49**

HGL's FROM B & C Profile				
HGL at UV effluent channel		**252.51**	**253.17**	
HGL upstream of modulating UV channel slide gate		**256.90**	**256.90**	
HGL at beginning of UV channel		**256.94**	**256.94**	
HGL in UV influent channel	**257.11**	**257.11**	***257.12***	***257.12***

SEGMENT B: FILTER INFLUENT FLUME TO UV INFLUENT CHANNEL

Filters

Segment Flow =	10.35	14.5	**52.00**	**43.50**
# Filters in Service =	**5**	**5**	**5**	**4**
Flow per Filter (mgd) =	2.07	2.90	10.40	10.88

B1 Primary Flow Path: Through Filters

Filter Effluent Header from Filter 2 to UV Influent Channel

Module PIPH *Piping by Hazen-Williams Equation*

			# Filters Contributing to Flow =	**5**	**5**	**5**	**4**
Diameter =	72	in	Q (mgd) =	10.35	14.50	52.00	43.5
Length =	60	ft	V (fps) =	0.57	0.79	2.85	2.38
Hazen & Will. C =	120		Hv (ft) =	0.00	0.01	0.13	0.09
Minor Loss K's :	Exit (1)	1	Hf (ft) =	0.00	0.00	0.02	0.02
	90 Bends (2)	0.6	Hm (ft) =	0.01	0.02	0.20	0.14
			HL (ft) =	0.01	0.02	0.22	0.16
	Sum K =	1.6					
	HGL at filter 2 effluent and header junction			**257.12**	**257.13**	**257.34**	**257.28**

Filter Effluent Header from Filter 3 to Filter 2

Module PIPH *Piping by Hazen-Williams Equation*

			# Filters Contributing to Flow =	**4**	**4**	**4**	**4**
Diameter =	72	in	Q (mgd) =	8.28	11.60	41.60	43.5
Length =	13.5	ft	V (fps) =	0.45	0.64	2.28	2.38
Hazen & Will. C =	120		Hv (ft) =	0.00	0.01	0.08	0.09
Minor Loss K's :	Run of Tee (1)	0.1	Hf (ft) =	0.00	0.00	0.00	0.00
			Hm (ft) =	0.00	0.00	0.01	0.01
			HL (ft) =	0.00	0.00	0.01	0.01
	Sum K =	0.1					
	HGL at filter 3 effluent and header junction			**257.12**	**257.13**	**257.35**	**257.29**

Filter Effluent Header from Filter 4 to Filter 3

Module PIPH *Piping by Hazen-Williams Equation*

			# Filters Contributing to Flow =	**3**	**3**	**3**	**3**
Diameter =	72	in	Q (mgd) =	6.21	8.7	31.2	32.625
Length =	13.5	ft	V (fps) =	0.34	0.48	1.71	1.79
Hazen & Will. C =	120		Hv (ft) =	0.00	0.00	0.05	0.05
Minor Loss K's :	Run of Tee (1)	0.1	Hf (ft) =	0.00	0.00	0.00	0.00
			Hm (ft) =	0.00	0.00	0.00	0.00
			HL (ft) =	0.00	0.00	0.01	0.01
	Sum K =	0.1					
	HGL at filter 4 effluent and header junction			**257.12**	**257.13**	**257.36**	**257.30**

Filter Effluent Header from Filter 5 to Filter 4

Module PIPH *Piping by Hazen-Williams Equation*

		# Filters Contributing to Flow =	**2**	**2**	**2**	**2**
Diameter =	72 in	Q (mgd) =	4.14	5.80	20.80	21.75
Length =	13.5 ft	V (fps) =	0.23	0.32	1.14	1.19
Hazen & Will. C =	120	Hv (ft) =	0.00	0.00	0.02	0.02
Minor Loss K's :	Run of Tee (1) 0.1	Hf (ft) =	0.00	0.00	0.00	0.00
		Hm (ft) =	0.00	0.00	0.00	0.00
		HL (ft) =	0.00	0.00	0.00	0.00
	Sum K = 0.1					
	HGL at filter 5 effluent and header junction		**257.12**	**257.13**	**257.36**	**257.30**

Filter Effluent Header from Filter 6 to Filter 5

Module PIPH *Piping by Hazen-Williams Equation*

		# Filters Contributing to Flow =	**1**	**1**	**1**	**1**
Diameter =	72 in	Q (mgd) =	2.07	2.90	10.40	10.875
Length =	13.5 ft	V (fps) =	0.11	0.16	0.57	0.60
Hazen & Will. C =	120	Hv (ft) =	0.00	0.00	0.01	0.01
Minor Loss K's :	Entrance (1) 0.5	Hf (ft) =	0.00	0.00	0.00	0.00
	Run of Tee (1) 0.1	Hm (ft) =	0.00	0.00	0.00	0.00
		HL (ft) =	0.00	0.00	0.00	0.00
	Sum K = 0.6					
	HGL at filter 6 effluent and header junction		**257.12**	**257.13**	**257.37**	**257.30**

Filter #6 Effluent Pipe

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24 in	Q (mgd) =	2.07	2.90	10.40	10.88
Length =	18 ft	V (fps) =	1.02	1.43	5.12	5.36
Hazen & Will. C =	100	Hv (ft) =	0.02	0.03	0.41	0.45
Minor Loss K's :	Entr (1) / Exit (1) 1.5	Hf (ft) =	0.00	0.01	0.10	0.11
	BFV (2) 1.14	Hm (ft) =	0.05	0.10	1.28	1.40
	Run of Tee (2) 0.2	HL (ft) =	0.06	0.11	1.38	1.51
	90 Bend (1) 0.3					
	Sum K = 3.14					
	HGL at end of filtered water collection conduit		**257.18**	**257.24**	**258.75**	**258.81**

Filtered Water Collection Conduit

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	3 ft	Q (mgd) =	2.07	2.90	10.40	10.88
Height =	3 ft	V (fps) =	0.36	0.50	1.79	1.87
Length =	22.67 ft	Hv (ft) =	0.00	0.00	0.05	0.05
Manning n =	0.015	Hf (ft) =	0.00	0.00	0.01	0.01
SumK =		Hm (ft) =	0.00	0.00	0.00	0.00
		HL (ft) =	0.00	0.00	0.01	0.01
	HGL in Filter Sump Trench		**257.18**	**257.24**	**258.76**	**258.82**

Headloss Through Clean Filter (ft)		Q (mgd) =	2.07	2.90	10.40	10.88
HL info from Tetra		Media Depth (ft) =	6.00	6.00	6.00	6.00
		HLR (gpm/sf) =	1.76	2.47	8.86	9.26
		Clean Media HL (ft) =	0.18	0.27	1.42	1.50
	HGL in Filter Cell		**257.35**	**257.51**	**260.18**	**260.33**

Filtering Head Available

Elevation of Influent Weir =	264.25	264.25	264.25	264.25
Filtering Head Available =	**6.40**	**6.24**	**3.57**	**3.42**

Filter Influent Weir

Module WS *Sharp Crested Weir*

Weir Elev =	264.25 ft	Q (mgd) =	2.07	2.90	10.40	10.88
Length =	136 ft	H_w (ft) =	0.04	0.05	0.11	0.11
C_w =	3.3	Weir Submergence (H_2)=	0.00	0.00	0.00	0.00
		<If Subm> H_1 (ft) =	1.00	1.00	1.00	1.00
		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
		Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
	HGL at end of filter influent trough		**264.29**	**264.30**	**264.36**	**264.36**

Filter Influent Trough

Module OCR *Short Rectangular Open Channel*

Invert El =	261.25	Q (mgd) =	1.035	1.45	5.2	5.4375
Width =	1.25 ft	Backwater (ft) =	3.04	3.05	3.11	3.11
Manning n =	0.015	Yc (ft) =	0.37	0.46	1.09	1.12
Length =	70.00 ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000 ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50	Area (sf) =	1.47	1.87	1.93	1.93
Downstream Conditions		Actual Depth (ft) =	3.04	3.05	3.11	3.11
		Area (sf) =	3.80	3.81	3.89	3.89
		V (fps) =	0.42	0.59	2.07	2.16
		Sf (ft/ft) =	0.0000	0.0001	0.0010	0.0011
Upstream Conditions		Upstrm Depth (ft) =	3.04	3.05	3.18	3.19
		Iterate 0 >>>	0.000	0.000	0.000	0.000
		Check >>>	ok	ok	ok	ok
		Area (sf) =	3.80	3.82	3.98	3.99
		V (fps) =	0.42	0.59	2.02	2.11
		Sf (ft/ft)=	0.0000	0.0001	0.0010	0.0011
		Hm (ft) =	0.00	0.00	0.03	0.03
	HGL at beginning of filter influent trough		**264.29**	**264.31**	**264.53**	**264.55**

Filter Influent Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	263.75 ft	Q (mgd) =	2.07	2.90	10.40	10.88
Throat Width (ft) =	0.75 ft	Unsubmerged Ha (ft) =	1.10	1.35	2.97	3.06
Length =	6.75 ft	Submergence Hb (ft) =	0.54	0.56	0.78	0.80
Coefficient K1 =	3.68	Submergence (%) =	49%	41%	26%	26%
Exponent n1 =	1.62	Submerged Ha (ft) =	1.00	1.66	2.15	2.15
Coefficient K2 =	1.96	<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.39	Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
Transitional Subm =	76%	Actual Ha =	1.10	1.35	2.97	3.06
k ent =	0.5	Actual Subm (ft) =	49%	41%	26%	26%
Width at Mth (ft) =	2.25 ft	Va (fps) =	1.66	1.89	3.09	3.14
		EGLa (ft) =	264.89	265.16	266.87	266.96
		H at Mouth (ft) =	1.12	1.38	3.04	3.13
		Iterate 0 >>>	0.00	0.00	0.00	0.00
		Check >>>	ok	ok	ok	ok
		V at Mouth (fps) =	1.27	1.45	2.35	2.39
		EGLm (ft) =	264.89	265.16	266.88	266.96
		Entrance Loss (ft) =	0.01	0.02	0.04	0.04
	HGL upstream of flume		**264.91**	**265.18**	**266.92**	**267.01**

B2 ALTERNATE FLOW PATH - FILTER BYPASS

Segment Flow = **82.00**

HGL in UV influent channel **257.12**

Submerged Rectangular Port

Module SPR *Submerged Rectangular Port*

Heighth =	6	ft	Q (mgd) =	82.00
Width =	4	ft	V (fps) =	5.29
Length =	18	in	HL (ft) =	0.99
No. Suppressed Cont =	2	(0 - 3)		
L/P =	0.075			
C_d =	0.66			

HGL at bottom of filter bypass dropbox **258.11**

Filter Bypass Drop Box

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	8	ft	Q (mgd) =	82.00
Height =	4	ft	V (fps) =	3.96
Length =	13.25	ft	Hv (ft) =	0.24
Manning n =	0.015		Hf (ft) =	0.01
SumK =	1.5		Hm (ft) =	0.37
			HL (ft) =	0.38

HGL downstream of bypass weir **258.49**

Downward Opening Weir Gate

Module WS *Sharp Crested Weir*

Weir Elev =	263.5	ft	Q (mgd) =	82.00
Length =	5	ft	H_w (ft) =	3.90
C_w =	3.3		Weir Submergence (H_2)=	0.00
			<If Subm> H_1 (ft) =	1.00
			<If Subm> Iterate 0 >>>	0.000
			Check >>>	Not Subm

HGL at filter influent channel (via Bypass) **267.40**
HGL at filter influent channel (via Filters) **267.01**

SEGMENT C: SECONDARY EFFLUENT WEIRS TO FILTER INFLUENT FLUMES

FILTER INFLUENT CHANNEL AND SECONDARY EFFLUENT PIPING

Segment Flow =	10.35	14.5	**46.00**	**33.00**
# Filters in Service =	**5**	**5**	**5**	**4**
Flow per Filter (mgd) =	2.07	2.90	9.20	8.25

Filter Influent Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	263.75	ft	Q (mgd) =	2.07	2.90	9.20	8.25
Throat Width (ft) =	0.75	ft	Unsubmerged Ha (ft) =	1.10	1.35	2.76	2.58
Length =	6.75	ft	Submergence Hb (ft) =	0.00	0.00	0.00	0.00
Coefficient K1 =	3.68		Submergence (%) =	0%	0%	0%	0%
Exponent n1 =	1.62		Submerged Ha (ft) =	1.00	1.66	2.15	2.15
Coefficient K2 =	1.96		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.39		Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
Transitional Subm =	76%		Actual Ha =	1.10	1.35	2.76	2.58
k ent =	0.5		Actual Subm (ft) =	0%	0%	0%	0%
Width at Mth (ft) =	2.25	ft	Va (fps) =	1.66	1.89	2.95	2.83
			EGLa (ft) =	264.89	265.16	266.64	266.45
			H at Mouth (ft) =	1.12	1.38	2.82	2.63
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V at Mouth (fps) =	1.27	1.45	2.24	2.15
			EGLm (ft) =	264.89	265.16	266.65	266.46
			Entrance Loss (ft) =	0.01	0.02	0.04	0.04
	HGL upstream of flume			**264.91**	**265.18**	**266.69**	**266.49**

Filter Influent Channel from Filter 5 to Filter 6

Module OCR *Short Rectangular Open Channel*

			# Filters Contributing to Flow =	**1**	**1**	**1**	**1**
Invert El =	261.25		Q (mgd) =	2.07	2.90	9.20	8.25
Width =	8.00	ft	Backwater (ft) =	3.66	3.93	5.44	5.24
Manning n =	0.015		Yc (ft) =	0.17	0.21	0.46	0.43
Length =	14.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =			Area (sf) =	9.39	11.97	12.33	12.33
			Actual Depth (ft) =	3.66	3.93	5.44	5.24
Downstream Conditions			Area (sf) =	29.26	31.42	43.48	41.94
			V (fps) =	0.11	0.14	0.33	0.30
			Sf (ft/ft) =	0.0000	0.0000	0.0000	0.0000
Upstream Conditions			Upstrm Depth (ft) =	3.66	3.93	5.44	5.24
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	29.26	31.42	43.48	41.94
			V (fps) =	0.11	0.14	0.33	0.30
			Sf (ft/ft)=	0.0000	0.0000	0.0000	0.0000
			Hm (ft) =	0.00	0.00	0.00	0.00
	EGL in filter influent channel at filter 5			**264.91**	**265.18**	**266.69**	**266.49**

EGL to HGL

Module OCREH *Rectangular Channel EGL to HGL Conversion*

Invert =	261.25	ft	Q (mgd) =	2.07	2.9	9.2	8.25
Width =	8.00	ft	Yc (ft) =	0.17	0.21	0.46	0.43
			E (ft) =	3.66	3.93	5.44	5.24
			Depth (ft) =	3.66	3.93	5.44	5.24
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V (fps) =	0.11	0.14	0.33	0.30
	HGL in filter influent channel at filter 5			**264.91**	**265.18**	**266.69**	**266.49**

Filter Influent Channel from Filter 4 to Filter 5

Module OCR *Short Rectangular Open Channel*

			# Filters Contributing to Flow =	**2**	**2**	**2**	**2**
Invert El =	261.25		Q (mgd) =	4.14	5.8	18.4	16.5
Width =	8.00	ft	Backwater (ft) =	3.66	3.93	5.44	5.24
Manning n =	0.015		Yc (ft) =	0.27	0.34	0.73	0.68
Length =	14.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =			Area (sf) =	9.39	11.97	12.33	12.33
			Actual Depth (ft) =	3.66	3.93	5.44	5.24
Downstream Conditions			Area (sf) =	29.26	31.42	43.48	41.94
			V (fps) =	0.22	0.29	0.65	0.61
			Sf (ft/ft) =	0.0000	0.0000	0.0000	0.0000
			Upstrm Depth (ft) =	3.66	3.93	5.44	5.24
Upstream Conditions			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	29.26	31.42	43.48	41.94
			V (fps) =	0.22	0.29	0.65	0.61
			Sf (ft/ft)=	0.0000	0.0000	0.0000	0.0000
			Hm (ft) =	0.00	0.00	0.00	0.00
	EGL in filter influent channel at filter 4			**264.91**	**265.18**	**266.69**	**266.50**

EGL to HGL

Module OCREH *Rectangular Channel EGL to HGL Conversion*

Invert =	261.25	ft	Q (mgd) =	4.14	5.8	18.4	16.5
Width =	8.00	ft	Yc (ft) =	0.27	0.34	0.73	0.68
			E (ft) =	3.66	3.93	5.44	5.25
			Depth (ft) =	3.66	3.93	5.44	5.24
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V (fps) =	0.22	0.29	0.65	0.61
	HGL in filter influent channel at filter 4			**264.91**	**265.18**	**266.69**	**266.49**

Filter Influent Channel from Influent Pipe to Filter 4

Module OCR *Short Rectangular Open Channel*

		# Filters Contributing to Flow =	**3**	**3**	**3**	**3**
Invert El =	261.25		Q (mgd) =	6.21	8.7	27.6	24.75
Width =	8.00 ft		Backwater (ft) =	3.66	3.93	5.44	5.24
Manning n =	0.015		Yc (ft) =	0.36	0.44	0.96	0.89
Length =	7.00 ft		Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000 ft/ft		Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	9.39	11.97	12.33	12.33
		Downstream Conditions	Actual Depth (ft) =	3.66	3.93	5.44	5.24
			Area (sf) =	29.26	31.42	43.48	41.94
			V (fps) =	0.33	0.43	0.98	0.91
			Sf (ft/ft) =	0.0000	0.0000	0.0000	0.0000
		Upstream Conditions	Upstrm Depth (ft) =	3.66	3.93	5.44	5.24
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	29.26	31.42	43.49	41.94
			V (fps) =	0.33	0.43	0.98	0.91
			Sf (ft/ft)=	0.0000	0.0000	0.0000	0.0000
			Hm (ft) =	0.00	0.00	0.01	0.01
		HGL in filter influent channel at influent pipe		**264.91**	**265.18**	**266.71**	**266.51**
		Compare to HGL in influent channel (via Bypass		**0.00**	**0.00**	**0.00**	**267.40**

SE Piping from SE Collection Header 72" Tee to Effluent Filter Complex

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	72 in	Q (mgd) =	10.35	14.50	46.00	33.00
Length =	178 ft	V (fps) =	0.57	0.79	2.52	1.81
Hazen & Will. C =	120	Hv (ft) =	0.00	0.01	0.10	0.05
Minor Loss K's :	Entr (1) / Exit (1) 1.5	Hf (ft) =	0.00	0.01	0.05	0.03
	90 Bend (2) 0.6	Hm (ft) =	0.01	0.02	0.21	0.11
	Sum K = 2.1	HL (ft) =	0.01	0.03	0.26	0.13
	HGL at 72" Tee in SE Collection Header		**264.92**	**265.21**	**266.97**	**266.65**

Secondary Clarifiers

		Total Secondary Effluent (SE) Flow =	**10.4**	**14.5**	**46.0**	**33.0**
		Total RAS Flow =	**10.4**	**14.5**	**14.5**	**14.5**
		Total Mixed Liquor (ML) Flow =	**20.7**	**29.0**	**60.5**	**47.5**
SC 1 Flow Proportion =	2	SC 1 in Service	1	1	1	1
SC 2 Flow Proportion =	1	SC 2 in Service	1	1	1	1
SC 3 Flow Proportion =	1	SC 3 in Service	1	1	1	1
SC 4 Flow Proportion =	2	SC 4 in Service	1	1	1	1
SC 5 Flow Proportion =	3	SC 5 in Service	0	1	1	0
		Flow Units =	6	9	9	6
		SC 1 % Flow =	**33.3%**	**22.2%**	**22.2%**	**33.3%**
		SC 2 % Flow =	**16.7%**	**11.1%**	**11.1%**	**16.7%**
		SC 3 % Flow =	**16.7%**	**11.1%**	**11.1%**	**16.7%**
		SC 4 % Flow =	**33.3%**	**22.2%**	**22.2%**	**33.3%**
		SC 5 % Flow =	**0.0%**	**33.3%**	**33.3%**	**0.0%**

Secondary Effluent Collection Header to SC 4 & 5

SE Header Piping - From SC 4&5 Jct to 72" Tee
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	72 in		Q (mgd) =	3.45	8.06	25.56	11.00
Length =	0 ft		V (fps) =	0.19	0.44	1.40	0.60
Hazen & Will. C =	120		Hv (ft) =	0.00	0.00	0.03	0.01
Minor Loss K's :	Exit @ 90 Tee	1	Hf (ft) =	0.00	0.00	0.00	0.00
			Hm (ft) =	0.00	0.00	0.03	0.01
			HL (ft) =	0.00	0.00	0.03	0.01
	Sum K =	1					
	HGL at SE #4 & 5 header upstream of 90 tee			**264.92**	**265.21**	**267.00**	**266.65**

SE Header Piping - From SC 4&5 Jct to 72" Tee
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	48 in		Q (mgd) =	3.45	8.06	25.56	11.00
Length =	8 ft		V (fps) =	0.42	0.99	3.15	1.35
Hazen & Will. C =	120		Hv (ft) =	0.00	0.02	0.15	0.03
Minor Loss K's :	Increaser 48x72	0.1	Hf (ft) =	0.00	0.00	0.01	0.00
	Run of 48" Tee	0.5	Hm (ft) =	0.00	0.01	0.09	0.02
			HL (ft) =	0.00	0.01	0.10	0.02
	Sum K =	0.6					
	HGL at SC #4&5 Jct			**264.93**	**265.22**	**267.10**	**266.67**

Secondary Effluent Collection Header to SC 1-3

SE Header Piping - From SC 1-3 to 72" Tee
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	72 in		Q (mgd) =	6.90	6.44	20.44	22.00
Length =	ft		V (fps) =	0.38	0.35	1.12	1.20
Hazen & Will. C =	120		Hv (ft) =	0.00	0.00	0.02	0.02
Minor Loss K's :	Exit @ 90 Tee	1	Hf (ft) =	0.00	0.00	0.00	0.00
			Hm (ft) =	0.00	0.00	0.02	0.02
			HL (ft) =	0.00	0.00	0.02	0.02
	Sum K =	1					
	HGL upstream of SC #1,2,&3 header tee branch			**264.93**	**265.21**	**266.99**	**266.67**

SE Header Piping - From Future Clarifier Jct to 72" Tee
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	60 in		Q (mgd) =	6.90	6.44	20.44	22.00
Length =	110 ft		V (fps) =	0.54	0.51	1.61	1.73
Hazen & Will. C =	120		Hv (ft) =	0.00	0.00	0.04	0.05
Minor Loss K's :	Increaser 60x72	0.1	Hf (ft) =	0.00	0.00	0.02	0.02
	Run of Tee (1)	0.1	Hm (ft) =	0.00	0.00	0.01	0.01
			HL (ft) =	0.00	0.00	0.03	0.03
	Sum K =	0.2					
	HGL at Future Clarifier Jct			**264.93**	**265.21**	**267.01**	**266.70**

SE Header Piping - From SC #3 Jct to Future Clarifier Jct

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	48 in		Q (mgd) =	6.90	6.44	20.44	22.00
Length =	22 ft		V (fps) =	0.85	0.79	2.52	2.71
Hazen & Will. C =	120		Hv (ft) =	0.01	0.01	0.10	0.11
Minor Loss K's :	Increaser 48x60	0.1	Hf (ft) =	0.00	0.00	0.01	0.01
	Entrance (1)	0.5	Hm (ft) =	0.01	0.01	0.06	0.07
			HL (ft) =	0.01	0.01	0.07	0.08
	Sum K =	0.6					
	HGL at SC#3 Jct			**264.94**	**265.22**	**267.08**	**266.78**

SE Header Piping - From SC2 Jct to SC3 Jct

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	48 in		Q (mgd) =	5.18	4.83	15.33	16.50
Length =	100 ft		V (fps) =	0.64	0.60	1.89	2.03
Hazen & Will. C =	120		Hv (ft) =	0.01	0.01	0.06	0.06
Minor Loss K's :	Run of Tee (1)	0.1	Hf (ft) =	0.00	0.00	0.03	0.03
			Hm (ft) =	0.00	0.00	0.01	0.01
			HL (ft) =	0.00	0.00	0.03	0.04
	Sum K =	0.1					
	HGL at SC#2 Jct			**264.94**	**265.22**	**267.11**	**266.82**

SE Header Piping - From SC1/Future SC Jct to SC2 Jct

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	48 in		Q (mgd) =	3.45	3.22	10.22	11.00
Length =	25 ft		V (fps) =	0.42	0.40	1.26	1.35
Hazen & Will. C =	120		Hv (ft) =	0.00	0.00	0.02	0.03
Minor Loss K's :	Run of Cross (1)	0.1	Hf (ft) =	0.00	0.00	0.00	0.00
	BFV	0.57	Hm (ft) =	0.00	0.00	0.02	0.02
	45 Bend (1)	0.2	HL (ft) =	0.00	0.00	0.02	0.03
	Sum K =	0.87					
	HGL at SC #1 Jct			**264.94**	**265.23**	**267.14**	**266.84**

Summary of HGLs in SE Collection Header

HGL at SC1 Jct	**264.94**	**265.23**	**267.14**	**266.84**
HGL at SC2 Jct	**264.94**	**265.22**	**267.11**	**266.82**
HGL at SC3 Jct	**264.94**	**265.22**	**267.08**	**266.78**
HGL at SC4 Jct	**264.93**	**265.22**	**267.10**	**266.67**
HGL at SC5 Jct	**264.93**	**265.22**	**267.10**	**266.67**

C1 Secondary Clarifier #1

SC #1 Effluent Piping

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30 in		Q (mgd) =	3.45	3.22	10.22	11.00
Length =	200 ft		V (fps) =	1.09	1.02	3.22	3.47
Hazen & Will. C =	100		Hv (ft) =	0.02	0.02	0.16	0.19
Minor Loss K's :	Entrance (1)	0.5	Hf (ft) =	0.05	0.04	0.36	0.41
	Increaser 30x48	0.1	Hm (ft) =	0.03	0.02	0.23	0.26
	90 Bend (2)	0.6	HL (ft) =	0.07	0.06	0.59	0.67
	11.5 Bend (2)	0.2					
	Sum K =	1.4					
	HGL at bottom of SC #1 effluent box			**265.02**	**265.29**	**267.72**	**267.52**

SC #1 Effluent Box

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	5 ft	Q (mgd) =	3.45	3.22	10.22	11.00
Height =	3 ft	V (fps) =	0.36	0.33	1.05	1.13
Length =	11 ft	Hv (ft) =	0.00	0.00	0.02	0.02
Manning n =	0.015	Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5	Hm (ft) =	0.00	0.00	0.03	0.03
		HL (ft) =	0.00	0.00	0.03	0.03
	HGL at SC #1 effluent box		**265.02**	**265.29**	**267.75**	**267.55**

SC #1 Effluent Launder

Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	265.5 ft	Q (mgd) =	1.73	1.61	5.11	5.50
Width =	3 ft	Backwater (ft) =	-0.48	-0.21	2.25	2.05
Length =	193.11 ft	Yc (ft) =	0.29	0.28	0.60	0.63
Slope =	0.0000 ft/ft	Actual Depth (ft) =	0.29	0.28	2.25	2.05
Manning n =	0.015	V-end (fps) =	3.06	2.99	1.17	1.38
		Hf (ft) ~	0.40	0.40	0.01	0.02
		Yu (ft) =	0.91	0.89	2.30	2.12
	HGL at Upstream End of SC #1 effluent launder		**266.41**	**266.39**	**267.80**	**267.62**
	Effluent Weir Elevation	267.79	**ok**	**ok**	**submerged**	**ok**

C2 Secondary Clarifier #2

SC #2 Effluent Piping

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24 in		Q (mgd) =	1.73	1.61	5.11	5.50
Length =	65 ft		V (fps) =	0.85	0.79	2.52	2.71
Hazen & Will. C =	100		Hv (ft) =	0.01	0.01	0.10	0.11
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.01	0.01	0.10	0.11
	22.5 Bend (2)	0.3	Hm (ft) =	0.03	0.02	0.23	0.27
	BFV (1)	0.57	HL (ft) =	0.04	0.03	0.33	0.38
	Sum K =	2.37					
	HGL at bottom of SC #2 effluent box			**264.98**	**265.26**	**267.44**	**267.20**

SC #2 Effluent Box

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	5 ft	Q (mgd) =	1.73	1.61	5.11	5.50
Height =	3 ft	V (fps) =	0.18	0.17	0.53	0.57
Length =	10 ft	Hv (ft) =	0.00	0.00	0.00	0.00
Manning n =	0.015	Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5	Hm (ft) =	0.00	0.00	0.01	0.01
		HL (ft) =	0.00	0.00	0.01	0.01
	HGL at SC #2 effluent box		**264.98**	**265.26**	**267.45**	**267.20**

SC #2 Effluent Launder
Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	265.5 ft	Q (mgd) =	0.86	0.81	2.56	2.75
Width =	3 ft	Backwater (ft) =	-0.52	-0.24	1.95	1.70
Length =	138.16 ft	Yc (ft) =	0.18	0.18	0.38	0.40
Slope =	0.0000 ft/ft	Actual Depth (ft) =	0.18	0.18	1.95	1.70
Manning n =	0.015	V-end (fps) =	2.42	2.37	0.68	0.83
		Hf (ft) ~	0.31	0.31	0.00	0.00
		Yu (ft) =	0.63	0.61	1.97	1.73
	HGL downstream of SC #2 effluent weir		**266.13**	**266.11**	**267.47**	**267.23**
	Effluent Weir Elevation	268	**ok**	**ok**	**ok**	**ok**

C3 Secondary Clarifier #3

SC #3 Effluent Piping
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24 in	Q (mgd) =	1.73	1.61	5.11	5.50
Length =	65 ft	V (fps) =	0.85	0.79	2.52	2.71
Hazen & Will. C =	100	Hv (ft) =	0.01	0.01	0.10	0.11
Minor Loss K's :	Entr (1) / Exit (1) 1.5	Hf (ft) =	0.01	0.01	0.10	0.11
	22.5 Bend (2) 0.3	Hm (ft) =	0.03	0.02	0.23	0.27
	BFV (1) 0.57	HL (ft) =	0.04	0.03	0.33	0.38
	Sum K = 2.37					
	HGL at bottom of SC #3 effluent box		**264.98**	**265.26**	**267.41**	**267.16**

SC #3 Effluent Box
Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	5 ft	Q (mgd) =	1.73	1.61	5.11	5.50
Height =	3 ft	V (fps) =	0.18	0.17	0.53	0.57
Length =	10 ft	Hv (ft) =	0.00	0.00	0.00	0.00
Manning n =	0.015	Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5	Hm (ft) =	0.00	0.00	0.01	0.01
		HL (ft) =	0.00	0.00	0.01	0.01
	HGL at SC #3 effluent box		**264.98**	**265.26**	**267.42**	**267.17**

SC #3 Effluent Launder
Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	265.5 ft	Q (mgd) =	0.86	0.81	2.56	2.75
Width =	3 ft	Backwater (ft) =	-0.52	-0.24	1.92	1.67
Length =	138.16 ft	Yc (ft) =	0.18	0.18	0.38	0.40
Slope =	0.0000 ft/ft	Actual Depth (ft) =	0.18	0.18	1.92	1.67
Manning n =	0.015	V-end (fps) =	2.42	2.37	0.69	0.85
		Hf (ft) ~	0.31	0.31	0.00	0.00
		Yu (ft) =	0.63	0.61	1.93	1.69
	HGL downstream of SC #3 effluent weir		**266.13**	**266.11**	**267.43**	**267.19**
	Effluent Weir Elevation	268	**ok**	**ok**	**ok**	**ok**

C4 Secondary Clarifier #4

SC #4 Effluent Piping

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30	in	Q (mgd) =	3.45	3.22	10.22	11.00
Length =	45	ft	V (fps) =	1.09	1.02	3.22	3.47
Hazen & Will. C =	100		Hv (ft) =	0.02	0.02	0.16	0.19
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.01	0.01	0.08	0.09
	90 @ Tee (1)	1.5	Hm (ft) =	0.07	0.06	0.57	0.66
	BFV (1)	0.54	HL (ft) =	0.08	0.07	0.65	0.75
	Sum K =	3.54					
	HGL at bottom of SC #4 effluent box			**265.00**	**265.29**	**267.75**	**267.42**

SC #4 Effluent Box

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	6	ft	Q (mgd) =	3.45	3.22	10.22	11.00
Height =	4.5	ft	V (fps) =	0.20	0.18	0.59	0.63
Length =	15	ft	Hv (ft) =	0.00	0.00	0.01	0.01
Manning n =	0.015		Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5		Hm (ft) =	0.00	0.00	0.01	0.01
			HL (ft) =	0.00	0.00	0.01	0.01
	HGL at SC #4 effluent box			**265.00**	**265.29**	**267.76**	**267.43**

SC #4 Effluent Trough

Module OCR *Short Rectangular Open Channel*

Invert El =	265.00		Q (mgd) =	3.45	3.22	10.22	11.00
Width =	4.50	ft	Backwater (ft) =	0.00	0.29	2.76	2.43
Manning n =	0.015		Yc (ft) =	0.35	0.34	0.73	0.76
Length =	14.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	5.28	6.74	6.94	6.94
			Actual Depth (ft) =	0.35	0.34	2.76	2.43
Downstream Conditions			Area (sf) =	1.59	1.52	12.40	10.95
			V (fps) =	3.36	3.29	1.28	1.55
			Sf (ft/ft) =	0.0056	0.0057	0.0001	0.0002
			Upstrm Depth (ft) =	0.49	0.47	2.73	2.53
Upstream Conditions			Iterate 0 >>>	0.000	0.001	209.878	-497.428
			Check >>>	ok	ok	Solve	Solve
			Area (sf) =	2.21	2.13	12.29	11.40
			V (fps) =	2.42	2.34	1.29	1.49
			Sf (ft/ft)=	0.0020	0.0020	0.0001	0.0002
			Hm (ft) =	0.05	0.04	0.01	0.02
	HGL at upstream end of trough			**265.63**	**265.60**	**267.77**	**267.59**

SC #4 Effluent Launder

Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	265	ft	Q (mgd) =	1.73	1.61	5.11	5.50
Width =	2	ft	Backwater (ft) =	0.63	0.60	2.77	2.59
Length =	128.74	ft	Yc (ft) =	0.38	0.36	0.79	0.83
Slope =	0.0000	ft/ft	Actual Depth (ft) =	0.63	0.60	2.77	2.59
Manning n =	0.015		V-end (fps) =	2.13	2.07	1.43	1.65
			Hf (ft) ~	0.07	0.07	0.01	0.02
			Yu (ft) =	0.83	0.79	2.85	2.69
	HGL downstream of SC #4 effluent weir			**265.83**	**265.79**	**267.85**	**267.69**
	Effluent Weir Elevation	**268**		**ok**	**ok**	**ok**	**ok**

C5 Secondary Clarifier #5

SC #5 Effluent Piping

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	42	in	Q (mgd) =	0.00	4.83	15.33	0.00
Length =	215	ft	V (fps) =	0.00	0.78	2.47	0.00
Hazen & Will. C =	120		Hv (ft) =	0.00	0.01	0.09	0.00
Minor Loss K's :	Entrance (1)	0.5	Hf (ft) =	0.00	0.01	0.11	0.00
	Increaser 42x48	0.1	Hm (ft) =	0.00	0.02	0.19	0.00
	45 Bend (3)	0.6	HL (ft) =	0.00	0.03	0.30	0.00
	90 Bend	0.3					
	BFV (1)	0.52					
	Sum K =	2.02					

HGL at bottom of SC #5 effluent box	**264.93**	**265.25**	**267.40**	**266.67**

SC #5 Effluent Box

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	6	ft	Q (mgd) =	0.00	4.83	15.33	0.00
Height =	4.5	ft	V (fps) =	0.00	0.28	0.88	0.00
Length =	14	ft	Hv (ft) =	0.00	0.00	0.01	0.00
Manning n =	0.015		Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5		Hm (ft) =	0.00	0.00	0.02	0.00
			HL (ft) =	0.00	0.00	0.02	0.00

HGL at SC #5 effluent box	**264.93**	**265.26**	**267.42**	**266.67**

SC #5 Effluent Launder

Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	264.09	ft	Q (mgd) =	0.00	2.42	7.67	0.00
Width =	3	ft	Backwater (ft) =	0.84	1.16	3.31	2.58
Length =	215.09	ft	Yc (ft) =	0.00	0.36	0.79	0.00
Slope =	0.0000	ft/ft	Actual Depth (ft) =	0.84	1.16	3.31	2.58
Manning n =	0.015		V-end (fps) =	0.00	1.07	1.19	0.00
			Hf (ft) ~	0.00	0.01	0.01	0.00
			Yu (ft) =	0.84	1.21	3.36	2.58

HGL at upstream end of SC #5 effluent launder	**264.93**	**265.30**	**267.45**	**266.67**
Effluent Weir Elevation 267.79	**ok**	**ok**	**ok**	**ok**

Summary of HGLs in SC Effluent Launders

HGL in SC #1 Launder	**266.41**	**266.39**	**267.80**	**267.62**
HGL in SC #2 Launder	**266.41**	**266.39**	**267.80**	**267.23**
HGL in SC #3 Launder	**266.13**	**266.11**	**267.43**	**267.19**
HGL in SC #4 Launder	**265.83**	**265.79**	**267.85**	**267.69**
HGL in SC #5 Launder	**264.93**	**265.30**	**267.45**	**266.67**

SEGMENT D: MIXED LIQUOR DISTRIBUTION FLUMES TO SECONDARY EFFLUENT WEIRS

Total Secondary Effluent (SE) Flow =	**10.4**	**14.5**	**43.5**	**23.0**
Total RAS Flow =	**10.4**	**14.5**	**14.5**	**14.5**
Total Mixed Liquor (ML) Flow =	**20.7**	**29.0**	**58.0**	**37.5**
SC 1 % Flow =	**33.3%**	**22.2%**	**22.0%**	**34.0%**
SC 2 % Flow =	**16.7%**	**11.1%**	**12.0%**	**18.8%**
SC 3 % Flow =	**16.7%**	**11.1%**	**9.0%**	**14.2%**
SC 4 % Flow =	**33.3%**	**22.2%**	**21.0%**	**33.0%**
SC 5 % Flow =	**0.0%**	**33.3%**	**36.0%**	**0.0%**
			100.0%	**100.0%**

D1 Secondary Clarifier #1: ML Flume to SC Effluent Weir

SC #1 Weir Notches

Module FPR *Free Surface Rectangular Port*

Invert El =	267.79 ft	Q/notch (mgd) =	0.0076	0.0071	0.0212	0.0173
Width=	0.15 ft	Backwater (ft) =	-1.38	-1.40	0.01	-0.17
Length =	12 in	Yc (ft) =	0.06	0.06	0.12	0.10
No. Suppressed Cont =	2 (0 - 3)	Act. Depth (ft) =	0.06	0.06	0.12	0.10
# Notches =	452	L/P =	3.79	3.87	2.64	2.86
		C_d =	0.94	0.94	0.89	0.90
		V (fps) =	1.37	1.34	1.93	1.80
		HL (ft) =	0.03	0.03	0.07	0.06
HGL upstream of SC #1 effluent weir			**267.88**	**267.88**	**267.98**	**267.95**

SC #1 Influent Well Diffuser Gates

Module FPR *Free Surface Rectangular Port (Ports closed to 6")*

Invert El =	265 ft	Q/gate (mgd) =	0.86	0.81	1.60	1.59
Width=	0.5 ft	Backwater (ft) =	2.88	2.88	2.98	2.95
Length =	0.25 in	Yc (ft) =	0.61	0.58	0.91	0.91
No. Suppressed Cont =	2 (0 - 3)	Act. Depth (ft) =	2.88	2.88	2.98	2.95
# Gates =	8	L/P =	0.00	0.00	0.00	0.00
		C_d =	0.62	0.62	0.62	0.62
		V (fps) =	0.93	0.87	1.66	1.67
		HL (ft) =	0.03	0.03	0.11	0.11
HGL upstream of SC #1 diffuser gates			**267.92**	**267.91**	**268.09**	**268.07**

SC #1 ML Inlet Ports

Module FPR *Free Surface Rectangular Port (Ports closed to 6")*

Invert El =	265 ft	Q/gate (mgd) =	1.73	1.61	3.19	3.19
Width=	0.5 ft	Backwater (ft) =	2.92	2.91	3.09	3.07
Length =	0.25 in	Yc (ft) =	0.96	0.92	1.45	1.45
No. Suppressed Cont =	2 (0 - 3)	Act. Depth (ft) =	2.92	2.91	3.09	3.07
# Gates =	4	L/P =	0.00	0.00	0.00	0.00
		C_d =	0.62	0.62	0.62	0.62
		V (fps) =	1.83	1.71	3.19	3.22
		HL (ft) =	0.13	0.12	0.41	0.42
HGL upstream of SC #1 ML inlet ports			**268.05**	**268.03**	**268.50**	**268.48**

SC #1 Pier

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	41.5	in	Q (mgd) =	6.90	6.44	12.76	12.75
Length =	15	ft	V (fps) =	1.14	1.06	2.10	2.10
Hazen & Will. C =	110		Hv (ft) =	0.02	0.02	0.07	0.07
Minor Loss K's :	Entrance/Exit	1.5	Hf (ft) =	0.00	0.00	0.01	0.01
			Hm (ft) =	0.03	0.03	0.10	0.10
			HL (ft) =	0.03	0.03	0.11	0.11
	Sum K =	1.5					
	HGL above RAS connection			**268.08**	**268.05**	**268.61**	**268.59**

SC #1 Pier @ RAS Connection

Module Pier *Clarifer Pier with concentric RAS pipe*

Pier ID =	41.5	in	Q (mgd) =	6.90	6.44	12.76	12.75
RAS Pipe OD =	18.5	in					
Length =	5	ft	V (fps) =	1.42	1.32	2.62	2.62
Manning n =	0.015		Hv (ft) =	0.03	0.03	0.11	0.11
SumK =	1.5		Hf (ft) =	0.00	0.00	0.01	0.01
Flow Area (sf) =	7.53	sf	Hm (ft) =	0.05	0.04	0.16	0.16
Rh =	0.479166667	ft	HL (ft) =	0.05	0.04	0.17	0.17
	HGL at bottom of SC #1 Pier			**268.10**	**268.07**	**268.67**	**268.65**

North AB Splitter Box to SC #1

Module PIPH *Piping by Hazen-Williams Equation*

d

Diameter =	30	in	Q (mgd) =	6.90	6.44	12.76	12.75
Length =	200	ft	V (fps) =	2.18	2.03	4.02	4.02
Hazen & Will. C =	100		Hv (ft) =	0.07	0.06	0.25	0.25
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.17	0.15	0.54	0.54
	90 Bend (4)	1.2	Hm (ft) =	0.23	0.20	0.78	0.78
	45 Bend (2)	0.4	HL (ft) =	0.40	0.35	1.32	1.32
	Sum K =	3.1					
	HGL at north AB splitter box			**268.50**	**268.42**	**269.99**	**269.97**

SC #1 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	268.59	ft	Q (mgd) =	6.90	6.44	12.76	12.75
Throat Width (ft) =	4.00	ft	Unsubmerged Ha (ft) =	0.82	0.79	1.22	1.22
Length =	9.0	ft	Submergence Hb (ft) =	0.00	0.00	1.40	1.38
Coefficient K1 =	3.50		Submergence (%) =	0%	0%	115%	113%
Exponent n1 =	1.56		Submerged Ha (ft) =	1.00	1.66	1.44	1.42
Coefficient K2 =	1.70		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.38		Check >>>	Not Subm	Not Subm	ok	ok
Transitional Subm =	80%		Actual Ha =	0.82	0.79	1.44	1.42
k ent =	0.5		Actual Subm (ft) =	0%	0%	98%	97%
Width at Mth (ft) =	6.00	ft	Va (fps) =	2.43	2.38	2.58	2.61
			EGLa (ft) =	269.50	269.46	270.13	270.11
			H at Mouth (ft) =	0.85	0.81	1.46	1.45
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V at Mouth (fps) =	2.10	2.05	2.25	2.27
			EGLm (ft) =	269.51	269.47	270.13	270.12
			Entrance Loss (ft) =	0.03	0.03	0.04	0.04
	HGL Upstream SC 1 Flume			**269.54**	**269.50**	**270.17**	**270.16**

D2 Secondary Clarifier #2: ML Flume to SC Effluent Weir

SC #2 Effluent Weir

Module WV *V-Notch Weir*

Weir El =	268 ft	Q/notch (mgd) =	0.005	0.005	0.016	0.014
# Notches =	320	H_w (ft) =	0.10	0.10	0.16	0.15
Depth of Notch =	3 in	Weir Submerged?	No	No	No	No
		Notch Overtopped?	No	No	No	No
	HGL upstream of SC #2		**268.10**	**268.10**	**268.16**	**268.15**

SC #2 ML Inlet Ports

Module FPR *Free Surface Rectangular Port*

					5.8875	5.8875
					0.490625	0.490625
Invert El =	265 ft	Q/gate (mgd) =	0.43	0.40	0.87	0.88
Width=	0.45 ft	Backwater (ft) =	3.10	3.10	3.16	3.15
Length =	1.5 in	Yc (ft) =	0.41	0.39	0.65	0.66
No. Suppressed Cont =	0 (0 - 3)	Act. Depth (ft) =	3.10	3.10	3.16	3.15
# Ports =	8	L/P =	0.02	0.02	0.02	0.02
		C_d =	0.61	0.61	0.61	0.61
		V (fps) =	0.48	0.45	0.95	0.96
		HL (ft) =	0.01	0.01	0.04	0.04
	HGL upstream of SC 2 ML inlet ports		**268.11**	**268.11**	**268.20**	**268.19**

SC #2 Pier

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	31 in	Q (mgd) =	3.45	3.22	6.96	7.05
Length =	15 ft	V (fps) =	1.02	0.95	2.06	2.08
Hazen & Will. C =	100	Hv (ft) =	0.02	0.01	0.07	0.07
Minor Loss K's :	Entrance/Exit 1.5	Hf (ft) =	0.00	0.00	0.01	0.01
		Hm (ft) =	0.02	0.02	0.10	0.10
		HL (ft) =	0.03	0.02	0.11	0.11
	Sum K = 1.5					
	HGL above RAS connection		**268.14**	**268.13**	**268.31**	**268.30**

SC #2 Pier @ RAS Connection

Module Pier *Clarifer Pier with concentric RAS pipe*

Pier ID =	31 in	Q (mgd) =	3.45	3.22	6.96	7.05
RAS Pipe OD =	12.5 in					
Length =	5 ft	V (fps) =	1.22	1.14	2.45	2.48
Manning n =	0.015	Hv (ft) =	0.02	0.02	0.09	0.10
SumK =	1.5	Hf (ft) =	0.00	0.00	0.01	0.01
Flow Area (sf) =	4.39 sf	Hm (ft) =	0.03	0.03	0.14	0.14
Rh =	0.385416667 ft	HL (ft) =	0.04	0.03	0.15	0.16
	HGL at bottom of SC #2 Pier		**268.15**	**268.14**	**268.35**	**268.34**

North AB Splitter Box to SC #2

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	20 in		Q (mgd) =	3.45	3.22	6.96	7.05
Length =	120 ft		V (fps) =	2.45	2.29	4.94	5.00
Hazen & Will. C =	100		Hv (ft) =	0.09	0.08	0.38	0.39
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.21	0.18	0.76	0.78
	90 Bends (3)	0.9	Hm (ft) =	0.22	0.19	0.91	0.93
			HL (ft) =	0.43	0.38	1.67	1.71
	Sum K =	2.4					
	HGL at north AB splitter box			**268.58**	**268.52**	**270.02**	**270.06**

SC #2 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	268.59 ft		Q (mgd) =	3.45	3.22	6.96	7.05
Throat Width (ft) =	2.00 ft		Unsubmerged Ha (ft) =	0.83	0.80	1.30	1.31
Length =	9.0 ft		Submergence Hb (ft) =	0.00	0.00	1.43	1.47
Coefficient K1 =	3.50		Submergence (%) =	0%	0%	110%	111%
Exponent n1 =	1.56		Submerged Ha (ft) =	1.00	1.66	1.48	1.51
Coefficient K2 =	1.70		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.38		Check >>>	Not Subm	Not Subm	ok	ok
Transitional Subm =	80%		Actual Ha =	0.83	0.80	1.48	1.51
k ent =	0.5		Actual Subm (ft) =	0%	0%	97%	97%
Width at Mth (ft) =	4.00 ft		Va (fps) =	1.93	1.88	2.18	2.16
			EGLa (ft) =	269.48	269.44	270.14	270.17
			H at Mouth (ft) =	0.85	0.82	1.51	1.54
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V at Mouth (fps) =	1.57	1.53	1.79	1.77
			EGLm (ft) =	269.48	269.44	270.15	270.18
			Entrance Loss (ft) =	0.02	0.02	0.02	0.02
	HGL Upstream SC 2 Flume			**269.50**	**269.46**	**270.17**	**270.20**

D3 Secondary Clarifier #3: ML Flume to SC Effluent Weir

SC #3 Effluent Weir

Module WV *V-Notch Weir*

Weir El =	268 ft		Q/notch (mgd) =	0.005	0.005	0.012	0.010
# Notches =	320		H_w (ft) =	0.10	0.10	0.14	0.13
Depth of Notch =	3 in		Weir Submerged?	No	No	No	No
			Notch Overtopped?	No	No	No	No
	HGL upstream of SC #3 effluent weir			**268.10**	**268.10**	**268.14**	**268.13**

SC #3 ML Inlet Ports

Module FPR *Free Surface Rectangular Port*

						5.8875	5.8875
						0.490625	0.490625
Invert El =	265 ft		Q/gate (mgd) =	0.43	0.40	0.65	0.67
Width=	0.45 ft		Backwater (ft) =	3.10	3.10	3.14	3.13
Length =	1.5 in		Yc (ft) =	0.41	0.39	0.54	0.55
No. Suppressed Cont =	0 (0 - 3)		Act. Depth (ft) =	3.10	3.10	3.14	3.13
# Ports =	8		L/P =	0.02	0.02	0.02	0.02
			C_d =	0.61	0.61	0.61	0.61
			V (fps) =	0.48	0.45	0.71	0.73
			HL (ft) =	0.01	0.01	0.02	0.02
	HGL upstream of SC 3 ML inlet ports			**268.11**	**268.11**	**268.16**	**268.15**

SC #3 Pier

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	31	in	Q (mgd) =	3.45	3.22	5.22	5.33
Length =	15	ft	V (fps) =	1.02	0.95	1.54	1.57
Hazen & Will. C =	100		Hv (ft) =	0.02	0.01	0.04	0.04
Minor Loss K's :	Entrance/Exit	1.5	Hf (ft) =	0.00	0.00	0.01	0.01
			Hm (ft) =	0.02	0.02	0.06	0.06
			HL (ft) =	0.03	0.02	0.06	0.06
	Sum K =	1.5					
	HGL above RAS connection			**268.14**	**268.13**	**268.23**	**268.22**

SC #3 Pier @ RAS Connection

Module Pier *Clarifer Pier with concentric RAS pipe*

Pier ID =	31	in	Q (mgd) =	3.45	3.22	5.22	5.33
RAS Pipe OD =	12.5	in					
Length =	5	ft	V (fps) =	1.22	1.14	1.84	1.88
Manning n =	0.015		Hv (ft) =	0.02	0.02	0.05	0.05
SumK =	1.5		Hf (ft) =	0.00	0.00	0.01	0.01
Flow Area (sf) =	4.39	sf	Hm (ft) =	0.03	0.03	0.08	0.08
Rh =	0.385416667	ft	HL (ft) =	0.04	0.03	0.08	0.09
	HGL at bottom of SC #3 Pier			**268.15**	**268.14**	**268.25**	**268.24**

North AB Splitter Box to SC #3

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	20	in	Q (mgd) =	3.45	3.22	5.22	5.33
Length =	260	ft	V (fps) =	2.45	2.29	3.70	3.78
Hazen & Will. C =	100		Hv (ft) =	0.09	0.08	0.21	0.22
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.45	0.40	0.97	1.01
	90 @ Tee (1)	1.5	Hm (ft) =	0.33	0.29	0.77	0.80
	90 Bends (2)	0.6	HL (ft) =	0.79	0.69	1.74	1.81
	Sum K =	3.6					
	HGL at north AB splitter box			**268.94**	**268.83**	**269.99**	**270.05**

SC #3 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	268.59	ft	Q (mgd) =	3.45	3.22	5.22	5.33
Throat Width (ft) =	2.00	ft	Unsubmerged Ha (ft) =	0.83	0.80	1.08	1.10
Length =	9.0	ft	Submergence Hb (ft) =	0.35	0.24	1.40	1.46
Coefficient K1 =	3.50		Submergence (%) =	42%	30%	129%	133%
Exponent n1 =	1.56		Submerged Ha (ft) =	1.00	1.66	1.41	1.47
Coefficient K2 =	1.70		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.38		Check >>>	Not Subm	Not Subm	ok	ok
Transitional Subm =	80%		Actual Ha =	0.83	0.80	1.41	1.47
k ent =	0.5		Actual Subm (ft) =	42%	30%	99%	99%
Width at Mth (ft) =	4.00	ft	Va (fps) =	1.93	1.88	1.72	1.68
			EGLa (ft) =	269.48	269.44	270.05	270.10
			H at Mouth (ft) =	0.85	0.82	1.43	1.48
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V at Mouth (fps) =	1.57	1.53	1.42	1.39
			EGLm (ft) =	269.48	269.44	270.05	270.10
			Entrance Loss (ft) =	0.02	0.02	0.02	0.01
	HGL Upstream SC 3 Flume			**269.50**	**269.46**	**270.06**	**270.12**

D4 Secondary Clarifier #4: ML Flume to SC Effluent Weir

SC #4 Weir Notches

Module FPR *Free Surface Rectangular Port*

Invert El =	268 ft	Q/notch (mgd) =	0.0056	0.0052	0.0148	0.0123
Width=	0.15 ft	Backwater (ft) =	-2.17	-2.21	-0.15	-0.31
Length =	0 in	Yc (ft) =	0.05	0.05	0.09	0.08
No. Suppressed Cont =	0 (0 - 3)	Act. Depth (ft) =	0.05	0.05	0.09	0.08
# Notches =	618	L/P =	0.00	0.00	0.00	0.00
		C_d =	0.60	0.60	0.60	0.60
		V (fps) =	1.24	1.21	1.71	1.61
		HL (ft) =	0.07	0.06	0.13	0.11
	HGL upstream of SC #4 effluent weir		**268.11**	**268.11**	**268.22**	**268.19**

Influent Well Diffuser Gates

Module FPR *Free Surface Rectangular Port (Gate Closed to ~ 6")*

Invert El =	264.61 ft	Q/gate (mgd) =	0.43	0.40	0.76	0.77
Width=	0.5 ft	Backwater (ft) =	3.50	3.50	3.61	3.58
Length =	0.25 in	Yc (ft) =	0.38	0.36	0.56	0.56
No. Suppressed Cont =	2 (0 - 3)	Act. Depth (ft) =	3.50	3.50	3.61	3.58
# Gates =	16	L/P =	0.00	0.00	0.00	0.00
		C_d =	0.62	0.62	0.62	0.62
		V (fps) =	0.38	0.36	0.65	0.67
		HL (ft) =	0.01	0.01	0.02	0.02
	HGL upstream of diffuser gates		**268.12**	**268.11**	**268.24**	**268.21**

ML Inlet Ports

Module SPR *Submerged Rectangular Port*

Heighth =	2.5 ft	Q (mgd) =	1.73	1.61	3.05	3.09
Width =	0.67 ft	V (fps) =	1.60	1.50	2.83	2.87
Length =	0.25 in	HL (ft) =	0.11	0.10	0.34	0.35
No. Suppressed Cont =	0 (0 - 3)					
# Ports =	4					
L/P =	0.003					
C_d =	0.60					
	HGL upstream of ML inlet ports		**268.23**	**268.21**	**268.58**	**268.56**

SC #4 Pier

Module Pier *Clarifer Pier with concentric RAS pipe*

Pier ID =	36 in	Q (mgd) =	6.90	6.44	12.18	12.38
RAS Pipe OD =	18 in					
Length =	20 ft	V (fps) =	2.01	1.88	3.55	3.61
Manning n =	0.015	Hv (ft) =	0.06	0.05	0.20	0.20
SumK =	1.5	Hf (ft) =	0.03	0.03	0.10	0.10
Flow Area (sf) =	5.30 sf	Hm (ft) =	0.09	0.08	0.29	0.30
Rh =	0.375 ft	HL (ft) =	0.12	0.11	0.39	0.40
	HGL at bottom of influent pier		**268.35**	**268.32**	**268.97**	**268.97**

South AB Splitter Box to SC #4

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30 in		Q (mgd) =	6.90	6.44	12.18	12.38
Length =	160 ft		V (fps) =	2.18	2.03	3.84	3.90
Hazen & Will. C =	100		Hv (ft) =	0.07	0.06	0.23	0.24
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.14	0.12	0.40	0.41
	90 Bends (3)	1.2	Hm (ft) =	0.20	0.17	0.62	0.64
			HL (ft) =	0.34	0.30	1.02	1.05
	Sum K =	2.7					
	HGL at south AB splitter box			**268.69**	**268.61**	**269.98**	**270.01**

SC #4 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	268.59 ft	Q (mgd) =	6.90	6.44	12.18	12.38
Throat Width (ft) =	4.00 ft	Unsubmerged Ha (ft) =	0.82	0.79	1.18	1.20
Length =	9.0 ft	Submergence Hb (ft) =	0.10	0.02	1.39	1.42
Coefficient K1 =	3.50	Submergence (%) =	12%	3%	118%	119%
Exponent n1 =	1.56	Submerged Ha (ft) =	1.00	1.66	1.42	1.45
Coefficient K2 =	1.70	<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.38	Check >>>	Not Subm	Not Subm	ok	ok
Transitional Subm =	80%	Actual Ha =	0.82	0.79	1.42	1.45
k ent =	0.5	Actual Subm (ft) =	12%	3%	98%	98%
Width at Mth (ft) =	6.00 ft	Va (fps) =	2.43	2.38	2.48	2.47
		EGLa (ft) =	269.50	269.46	270.11	270.14
		H at Mouth (ft) =	0.85	0.81	1.45	1.47
		Iterate 0 >>>	0.00	0.00	0.00	0.00
		Check >>>	ok	ok	ok	ok
		V at Mouth (fps) =	2.10	2.05	2.17	2.16
		EGLm (ft) =	269.51	269.47	270.11	270.14
		Entrance Loss (ft) =	0.03	0.03	0.04	0.04
	HGL Upstream SC 4 Flume		**269.54**	**269.50**	**270.15**	**270.17**

D5 Secondary Clarifier #5: ML Flume to SC Effluent Weir

SC #5 Weir Notches

Module FPR *Free Surface Rectangular Port*

Invert El =	267.79 ft	Q/notch (mgd) =	0.0000	0.0096	0.0310	0.0000
Width=	0.15 ft	Backwater (ft) =	-2.86	-2.49	-0.34	-1.12
Length =	0 in	Yc (ft) =	0.00	0.07	0.15	0.00
No. Suppressed Cont =	0 (0 - 3)	Act. Depth (ft) =	0.00	0.07	0.15	0.00
# Notches =	505	L/P =	0.00	0.00	0.00	0.00
		C_d =	0.60	0.60	0.60	0.60
		V (fps) =	#DIV/0!	1.48	2.19	#DIV/0!
		HL (ft) =	#DIV/0!	0.09	0.21	#DIV/0!
	HGL upstream of SC #5 effluent weir		**267.79**	**267.95**	**268.15**	**267.79**

Influent Well Diffuser Gates

Module FPR *Free Surface Rectangular Port (Gates Closed to ~ 12")*

Invert El =	265.19	ft	Q/gate (mgd) =	0.00	1.21	2.61	0.00
Width=	1	ft	Backwater (ft) =	2.60	2.76	2.96	2.60
Length =	0.25	in	Yc (ft) =	0.00	0.48	0.80	0.00
No. Suppressed Cont =	2	(0 - 3)	Act. Depth (ft) =	2.60	2.76	2.96	2.60
# Gates =	8		L/P =	0.00	0.00	0.00	0.00
			C_d =	0.62	0.62	0.62	0.62
			V (fps) =	0.00	0.68	1.37	0.00
			HL (ft) =	0.00	0.02	0.07	0.00
HGL upstream of diffuser ports				**267.79**	**267.97**	**268.22**	**267.79**

ML Inlet Ports

Module SPR *Submerged Rectangular Port*

Heighth =	2	ft	Q (mgd) =	0.00	2.42	5.22	0.00
Width =	1.50	ft	V (fps) =	0.00	1.25	2.69	0.00
Length =	0	in	HL (ft) =	0.00	0.07	0.31	0.00
No. Suppressed Cont =	0	(0 - 3)					
# Ports =	4						
L/P =	0.000						
C_d =	0.60						
HGL upstream of ML inlet ports				**267.79**	**268.04**	**268.53**	**267.79**

SC #5 Pier

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	48	in	Q (mgd) =	0.00	9.67	20.88	0.00
Length =	20	ft	V (fps) =	0.00	1.19	2.57	0.00
Hazen & Will. C =	120		Hv (ft) =	0.00	0.02	0.10	0.00
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.00	0.00	0.01	0.00
			Hm (ft) =	0.00	0.03	0.15	0.00
			HL (ft) =	0.00	0.04	0.16	0.00
	Sum K =	1.5					
HGL at bottom of SC #5 Pier				**267.79**	**268.07**	**268.70**	**267.79**

South AB Splitter Box to SC #5

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	42	in	Q (mgd) =	0.00	9.67	20.88	0.00
Length =	215	ft	V (fps) =	0.00	1.56	3.36	0.00
Hazen & Will. C =	120		Hv (ft) =	0.00	0.04	0.18	0.00
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.00	0.05	0.20	0.00
	90 Bends (3)	0.9	Hm (ft) =	0.00	0.10	0.46	0.00
	45 Bends (1)	0.2	HL (ft) =	0.00	0.15	0.66	0.00
	Sum K =	2.6					
HGL at south AB splitter box				**267.79**	**268.22**	**269.35**	**267.79**

SC #5 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	268.59 ft	Q (mgd) =	0.00	9.67	20.88	0.00
Throat Width (ft) =	6.00 ft	Unsubmerged Ha (ft) =	0.00	0.78	1.28	0.00
Length =	9.0 ft	Submergence Hb (ft) =	0.00	0.00	0.76	0.00
Coefficient K1 =	3.50	Submergence (%) =	#DIV/0!	0%	60%	#DIV/0!
Exponent n1 =	1.56	Submerged Ha (ft) =	1.00	1.66	1.94	1.75
Coefficient K2 =	1.70	<If Subm> Iterate 0 >>>	#DIV/0!	0.000	0.000	#DIV/0!
Exponent n2 =	1.38	Check >>>	#DIV/0!	Not Subm	Not Subm	#DIV/0!
Transitional Subm =	80%	Actual Ha =	#DIV/0!	0.78	1.28	#DIV/0!
k ent =	0.5	Actual Subm (ft) =	#DIV/0!	0%	59.74%	#DIV/0!
Width at Mth (ft) =	8.00 ft	Va (fps) =	#DIV/0!	2.61	3.44	#DIV/0!
		EGLa (ft) =	#DIV/0!	269.48	270.05	#DIV/0!
		H at Mouth (ft) =	0.85	0.81	1.32	0.85
		Iterate 0 >>>	#DIV/0!	0.00	0.00	#DIV/0!
		Check >>>	#DIV/0!	ok	ok	#DIV/0!
		V at Mouth (fps) =	0.00	2.32	3.06	0.00
		EGLm (ft) =	269.44	269.48	270.06	269.44
		Entrance Loss (ft) =	0.00	0.04	0.07	0.00
HGL Upstream SC 5 Flume			**267.79**	**269.52**	**270.13**	**267.79**
Summary of HGLs in Mixed Liquor Distribution Boxes						
North AB Splitter Box Upstream of SC #1						
		HGL Downstream of Flume	**268.50**	**268.42**	**269.99**	**269.97**
		HGL Upstream of Flume	**269.54**	**269.50**	**270.17**	**270.16**
North AB Splitter Box Upstream of SC #2						
		HGL Downstream of Flume	**268.58**	**268.52**	**270.02**	**270.06**
		HGL Upstream of Flume	**269.50**	**269.46**	**270.17**	**270.20**
North AB Splitter Box Upstream of SC #3						
		HGL Downstream of Flume	**268.94**	**268.83**	**269.99**	**270.05**
		HGL Upstream of Flume	**269.50**	**269.46**	**270.06**	**270.12**
South AB Splitter Box Upstream of SC #4						
		HGL Downstream of Flume	**268.69**	**268.61**	**269.98**	**270.01**
		HGL Upstream of Flume	**269.54**	**269.50**	**270.15**	**270.17**
South AB Splitter Box Upstream of SC #5						
		HGL Downstream of Flume	**267.79**	**268.22**	**269.35**	**267.79**
		HGL Upstream of Flume	**267.79**	**269.52**	**270.13**	**267.79**
HGL Upstream Distribution Flumes			**269.54**	**269.52**	**270.17**	**270.20**

SEGMENT E: AERATION BASIN INFLUENT CHANNEL TO MIXED LIQUOR DISTRIBUTION FLUMES

Aeration Basins in 3:4 Configuration

Total Segment ML Flow =	**20.7**	**29**	**66**	**44**
PE Flow =	**10.35**	**14.5**	**51.5**	**29.5**
RAS Flow =	**10.35**	**14.5**	**14.5**	**14.5**
Total ML Flow to North Distn Box =	**13.80**	**12.89**	**29.33**	**29.33**
Total ML Flow to South Distn Box =	**6.90**	**16.11**	**36.67**	**14.67**
# of Flow Trains =	**3**	**3**	**3**	**2**
Flow per Train =	**6.90**	**9.67**	**22.00**	**22.00**
HGL Upstream Distribution Flumes	**269.54**	**269.52**	**270.17**	**270.20**

Effluent Channel #2

Module OCR *Short Rectangular Open Channel*

Invert El =	261.00		Q (mgd) =	6.90	16.11	36.67	14.67
Width =	7.00	ft	Backwater (ft) =	8.54	8.52	9.17	9.20
Manning n =	0.015		Yc (ft) =	0.42	0.73	1.27	0.69
Length =	110	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	8.22	10.48	10.79	10.79
			Actual Depth (ft) =	8.54	8.52	9.17	9.20
Downstream Conditions			Area (sf) =	59.78	59.64	64.20	64.40
			V (fps) =	0.18	0.42	0.88	0.35
			Sf (ft/ft) =	0.0000	0.0000	0.0000	0.0000
			Upstrm Depth (ft) =	8.54	8.52	9.17	9.20
Upstream Conditions			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	59.78	59.65	64.21	64.40
			V (fps) =	0.18	0.42	0.88	0.35
			Sf (ft/ft)=	0.0000	0.0000	0.0000	0.0000
			Hm (ft) =	0.00	0.00	0.01	0.00
HGL after weir gate to Effluent Channel 2				**269.54**	**269.53**	**270.19**	**270.20**

Downward Opening Weir Gate from Effluent Channel 5B to Effluent Channel 2

Module FPR *Free Surface Rectangular Port*

			Q/gate (mgd) =	10.35	14.5	33	22
Invert El =	266.66	ft	Backwater (ft) =	2.88	2.87	3.53	3.54
Width=	7	ft	Yc (ft) =	0.55	0.68	1.18	0.90
Length =	12	in	Act. Depth (ft) =	2.88	2.87	3.53	3.54
No. Suppressed Cont =	0	(0 - 3)	L/P =	0.08	0.08	0.07	0.07
			C_d =	0.64	0.64	0.64	0.64
			V (fps) =	0.79	1.12	2.07	1.37
			HL (ft) =	0.02	0.05	0.16	0.07
HGL before weir gate to Effluent Channel 2				**269.56**	**269.57**	**270.35**	**270.27**

Downward Opening Weir Gate from Effluent Channel 1 to Effluent Channel 5A or 5B

Module FPR *Free Surface Rectangular Port*

		Q/gate (mgd) =	10.35	14.5	33	22
Invert El =	266.66 ft	Backwater (ft) =	2.90	2.91	3.69	3.61
Width=	7 ft	Yc (ft) =	0.55	0.68	1.18	0.90
Length =	12 in	Act. Depth (ft) =	2.90	2.91	3.69	3.61
No. Suppressed Cont =	0 (0 - 3)	L/P =	0.08	0.08	0.07	0.07
		C_d =	0.64	0.64	0.64	0.64
		V (fps) =	0.79	1.10	1.97	1.35
		HL (ft) =	0.02	0.05	0.15	0.07
	HGL before weir gate to Cell 5B		**269.59**	**269.62**	**270.50**	**270.34**

Upward Opening Weir Gate from Cell 4B to Effluent Channel 1

Module FPR *Free Surface Rectangular Port*

No. Gates =	2	Q/gate (mgd) =	3.45	4.83	11.00	11.00
Invert El =	269.17 ft	Backwater (ft) =	0.42	0.45	1.33	1.17
Width=	3 ft	Yc (ft) =	0.46	0.58	1.00	1.00
Length =	12 in	Act. Depth (ft) =	0.46	0.58	1.33	1.17
No. Suppressed Cont =	0 (0 - 3)	L/P =	0.25	0.24	0.18	0.19
		C_d =	0.74	0.74	0.70	0.71
		V (fps) =	3.85	4.31	4.26	4.83
		HL (ft) =	0.42	0.53	0.58	0.73
	HGL at Cell 4B		**270.05**	**270.28**	**271.08**	**271.07**

Weir Gate from Cell 3C to Cell 4B

Module FPR *Free Surface Rectangular Port*

No. Gates =	2	Q/gate (mgd) =	3.45	4.83	11.00	11.00
Invert El =	269.5 ft	Backwater (ft) =	0.55	0.78	1.58	1.57
Width=	3 ft	Yc (ft) =	0.46	0.58	1.00	1.00
Length =	12 in	Act. Depth (ft) =	0.55	0.78	1.58	1.57
No. Suppressed Cont =	0 (0 - 3)	L/P =	0.24	0.22	0.16	0.16
		C_d =	0.74	0.72	0.69	0.69
		V (fps) =	3.25	3.19	3.60	3.61
		HL (ft) =	0.30	0.30	0.42	0.42
	HGL at Cell 3C		**270.35**	**270.58**	**271.50**	**271.49**

Weir Gate from Cell 3D to 3C

Module FPR *Free Surface Rectangular Port*

No. Gates =	2	Q/gate (mgd) =	3.45	4.83	11.00	11.00
Invert El =	269.5 ft	Backwater (ft) =	0.85	1.08	2.00	1.99
Width=	3 ft	Yc (ft) =	0.46	0.58	1.00	1.00
Length =	12 in	Act. Depth (ft) =	0.85	1.08	2.00	1.99
No. Suppressed Cont =	0 (0 - 3)	L/P =	0.21	0.19	0.14	0.14
		C_d =	0.72	0.71	0.68	0.68
		V (fps) =	2.10	2.30	2.84	2.85
		HL (ft) =	0.13	0.16	0.27	0.27
	HGL at Cell 3D		**270.48**	**270.75**	**271.77**	**271.76**

Weir Gate from Cell 4A to 3D

Module FPR *Free Surface Rectangular Port*

No. Gates =	2	Q/gate (mgd) =	3.45	4.83	11.00	11.00
Invert El =	269.5 ft	Backwater (ft) =	0.98	1.25	2.27	2.26
Width=	3 ft	Yc (ft) =	0.46	0.58	1.00	1.00
Length =	12 in	Act. Depth (ft) =	0.98	1.25	2.27	2.26
No. Suppressed Cont =	0 (0 - 3)	L/P =	0.20	0.18	0.13	0.13
		C_d =	0.71	0.70	0.68	0.68
		V (fps) =	1.82	2.00	2.50	2.50
		HL (ft) =	0.10	0.13	0.21	0.21
	HGL at Cell 4A		**270.58**	**270.87**	**271.98**	**271.98**

Weir Gate from Influent Channel to Cell 4A

Module FPR *Free Surface Rectangular Port*

No. Gates =	2	Q/gate (mgd) =	3.45	4.83	11.00	11.00
Invert El =	271.17 ft	Backwater (ft) =	-0.59	-0.30	0.81	0.81
Width=	3 ft	Yc (ft) =	0.46	0.58	1.00	1.00
Length =	12 in	Act. Depth (ft) =	0.46	0.58	1.00	1.00
No. Suppressed Cont =	0 (0 - 3)	L/P =	0.25	0.24	0.20	0.20
		C_d =	0.74	0.74	0.71	0.71
		V (fps) =	3.85	4.31	5.67	5.67
		HL (ft) =	0.42	0.53	0.98	0.98
	HGL at Influent Channel		**272.05**	**272.28**	**273.15**	**273.15**

Aeration Basin Influent Channel

Module OCR *Short Rectangular Open Channel*

Invert El =	261.00	Q (mgd) =	6.90	9.67	22.00	22.00
Width =	7.00 ft	Backwater (ft) =	11.05	11.28	12.15	12.15
Manning n =	0.015	Yc (ft) =	0.42	0.52	0.90	0.90
Length =	80.00 ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000 ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =		Area (sf) =	8.22	10.48	10.79	10.79
	Downstream Conditions	Actual Depth (ft) =	11.05	11.28	12.15	12.15
		Area (sf) =	77.33	78.96	85.06	85.06
		V (fps) =	0.14	0.19	0.40	0.40
		Sf (ft/ft) =	0.0000	0.0000	0.0000	0.0000
	Upstream Conditions	Upstrm Depth (ft) =	11.05	11.28	12.15	12.15
		Iterate 0 >>>	0.000	0.000	0.000	0.000
		Check >>>	ok	ok	ok	ok
		Area (sf) =	77.33	78.96	85.07	85.07
		V (fps) =	0.14	0.19	0.40	0.40
		Sf (ft/ft)=	0.0000	0.0000	0.0000	0.0000
		Hm (ft) =	0.00	0.00	0.00	0.00
	EGL in influent channel		**272.05**	**272.28**	**273.15**	**273.15**

EGL to HGL

Module OCREH *Rectangular Channel EGL to HGL Conversion*

Invert =	261.00 ft	Q (mgd) =	13.8	19.33	44.00	44
Width =	7.00 ft	Yc (ft) =	0.66	0.83	1.43	1.43
		E (ft) =	11.05	11.28	12.15	12.15
		Depth (ft) =	11.05	11.28	12.14	12.14
		Iterate 0 >>>	0.00	0.00	0.00	0.00
		Check >>>	ok	ok	ok	ok
		V (fps) =	0.28	0.38	0.80	0.80
	HGL in influent channel		**272.05**	**272.28**	**273.14**	**273.14**
	Top of Wall 273.66	**Freeboard**	**1.61**	**1.38**	**0.52**	**0.52**

Check Aeration Basin Flow Distribution

HGL Variation in Influent Channel	0.001	0.001	0.007	0.007
Head Loss at Influent Gates	0.42	0.53	0.98	0.98
Flow Distribution	**100%**	**100%**	**100%**	**100%**

Intermediate PS

Intermediate Pump Station #2 Wetwell =	**249.00**	**249.00**	**249.00**	**249.00**
Max HGL in Aeration Basin Influent Channel =	**272.05**	**272.28**	**273.14**	**273.14**
Pump Station Static Head (ft) =	23.05	23.28	24.14	24.14

SEGMENT F: PRIMARY CLARIFIER EFFLUENT WEIRS TO INTERMEDIATE PUMP STATION

Total Segment Flow =	**10.35**	**14.5**	**43.5**	**43.5**

Splitter Box #2 to Intermediate PS #2 Wetwell
Module PIPH *Piping by Hazen-Williams Equation*

Bypass from Headworks (or thru TF) =			**10.0**	**16.0**

Diameter =	30 in		Q (mgd) =	10.35	14.5	33.5	27.5
Length =	310 ft		V (fps) =	3.26	4.57	10.56	8.67
Hazen & Will. C =	100		Hv (ft) =	0.17	0.32	1.73	1.17
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.57	1.06	5.01	3.48
	Bends	0.9	Hm (ft) =	0.56	1.10	5.89	3.97
		1	HL (ft) =	1.13	2.17	10.90	7.45
	Sum K =	3.4					

HGL at splitter box #2	**250.13**	**251.17**	**259.90**	**256.45**

Splitter Box #2 Effluent Weir
Module WS *Sharp Crested Weir*

Weir Elev =	259.6 ft		Q (mgd) =	10.35	14.5	33.5	27.5
Length =	18 ft		H_w (ft) =	0.42	0.52	0.96	0.80
C_w =	3.3		Weir Submergence (H_2)=	0.00	0.00	0.30	0.00
			<If Subm> H_1 (ft) =	1.00	1.00	0.96	0.97
			<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	Not Subm	Not Subm	ok	Not Subm

HGL upstream of splitter box #2 effluent weir	**260.02**	**260.12**	**260.56**	**260.40**
TOW 264.5 Freeboard	**4.48**	**4.38**	**3.94**	**4.10**

Primary Clarifiers

Total Primary Effluent (PE) Flow =	**10.4**	**14.5**	**33.50**	**27.50**
# Clarifiers in Service =	**3**	**3**	**3**	**2**
PE Flow per Clarifier =	**3.45**	**4.83**	**11.17**	**13.75**

F1 Primary Clarifier #1

PE #1 to Splittler Box #2
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30 in		Q (mgd) =	3.45	4.83	11.17	13.75
Length =	24 ft		V (fps) =	1.09	1.52	3.52	4.34
Hazen & Will. C =	100		Hv (ft) =	0.02	0.04	0.19	0.29
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.01	0.01	0.05	0.07
	45 Bend (1)	0.2	Hm (ft) =	0.04	0.07	0.39	0.58
	90 Bend (1)	0.3	HL (ft) =	0.04	0.08	0.44	0.66
	Sum K =	2					

HGL at bottom of PC #1 effluent box	**260.06**	**260.21**	**261.00**	**261.06**

PC #1 Effluent Launder
Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	258.63 ft		Q (mgd) =	1.73	2.42	5.58	6.88
Width =	1.5 ft		Backwater (ft) =	1.43	1.58	2.37	2.43
Length =	114.61 ft		Yc (ft) =	0.46	0.58	1.01	1.16
Slope =	0.0000 ft/ft		Actual Depth (ft) =	1.43	1.58	2.37	2.43
Manning n =	0.015		V-end (fps) =	1.24	1.58	2.43	2.92
			Hf (ft) ~	0.02	0.02	0.05	0.07
			Yu (ft) =	1.49	1.68	2.59	2.75

HGL at upstream end of PC #1 effluent launder	**260.12**	**260.31**	**261.22**	**261.38**

	Effluent Weir Elevation	261.44	**ok**	**ok**	**ok**	**ok**

F2 Primary Clarifier #2

PE #2 to Splittler Box #2

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30 in		Q (mgd) =	3.45	4.83	11.17	13.75
Length =	33 ft		V (fps) =	1.09	1.52	3.52	4.34
Hazen & Will. C =	100		Hv (ft) =	0.02	0.04	0.19	0.29
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.01	0.01	0.07	0.10
	45 Bend (1)	0.2	Hm (ft) =	0.04	0.07	0.39	0.58
	90 Bend	0.3	HL (ft) =	0.04	0.09	0.45	0.69
	Sum K =	2					

HGL at bottom of PC #2 effluent box	**260.06**	**260.21**	**261.01**	**261.09**

PC #2 Effluent Launder

Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	258.63 ft	Q (mgd) =	1.73	2.42	5.58	6.88
Width =	1.5 ft	Backwater (ft) =	1.43	1.58	2.38	2.46
Length =	114.61 ft	Yc (ft) =	0.46	0.58	1.01	1.16
Slope =	0.0000 ft/ft	Actual Depth (ft) =	1.43	1.58	2.38	2.46
Manning n =	0.015	V-end (fps) =	1.24	1.58	2.42	2.89
		Hf (ft) ~	0.02	0.02	0.05	0.07
		Yu (ft) =	1.49	1.68	2.61	2.77

HGL at upstream end of PC #2 effluent launder	**260.12**	**260.31**	**261.24**	**261.40**
Effluent Weir Elevation 261.44	**ok**	**ok**	**ok**	**ok**

F3 Primary Clarifier #3

PE #3 to Splittler Box #2

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30 in		Q (mgd) =	3.45	4.83	11.17	13.75
Length =	83 ft		V (fps) =	1.09	1.52	3.52	4.34
Hazen & Will. C =	100		Hv (ft) =	0.02	0.04	0.19	0.29
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.02	0.04	0.18	0.26
			Hm (ft) =	0.03	0.05	0.29	0.44
			HL (ft) =	0.05	0.09	0.46	0.70
	Sum K =	1.5					

HGL at bottom of PC #3 effluent box	**260.06**	**260.21**	**261.02**	**261.10**

PC #3 Effluent Box

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	6 ft	Q (mgd) =	3.45	4.83	11.17	13.75
Height =	4.67 ft	V (fps) =	0.19	0.27	0.62	0.76
Length =	2 ft	Hv (ft) =	0.00	0.00	0.01	0.01
Manning n =	0.015	Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5	Hm (ft) =	0.00	0.00	0.01	0.01
		HL (ft) =	0.00	0.00	0.01	0.01

HGL in PC #3 effluent box	**260.07**	**260.22**	**261.03**	**261.11**

PC #3 Effluent Launder

Module OCLI *Rectangular Lateral Inflow Channel*

Invert El =	258.12 ft	Q (mgd) =	1.73	2.42	5.58	6.88
Width =	2 ft	Backwater (ft) =	1.95	2.10	2.91	2.99
Length =	128.74 ft	Yc (ft) =	0.38	0.48	0.83	0.96
Slope =	0.0000 ft/ft	Actual Depth (ft) =	1.95	2.10	2.91	2.99
Manning n =	0.015	V-end (fps) =	0.69	0.89	1.48	1.78
		Hf (ft) ~	0.00	0.01	0.01	0.02
		Yu (ft) =	1.96	2.13	2.99	3.11
	HGL at upstream end of PC #3 effluent launder		**260.08**	**260.25**	**261.11**	**261.23**
	Effluent Weir Elevation	261.44	**ok**	**ok**	**ok**	**ok**

SEGMENT G: PRIMARY CLARIFIER INFLUENT DISTRIBUTION FLUMES TO PC EFFLUENT WEIRS

Total Segment Flow =	**10.35**	**14.5**	**43.5**	**43.5**
Bypass from Headworks =				**17.0**
Total Primary Effluent (PE) Flow =	**10.4**	**14.5**	**43.5**	**26.5**
# Clarifiers in Service =	**3**	**3**	**3**	**2**
PE Flow per Clarifier =	**3.45**	**4.83**	**14.50**	**13.25**
PC 1 % Flow =			**34.0%**	**50.0%**
PC 2 % Flow =			**34.0%**	**50.0%**
PC 3 % Flow =			**32.0%**	**0.0%**
			100.0%	**100.0%**

G1 Primary Clarifier #1: Influent Flume to Effluent Weir

PC #1 Effluent Weir
Module WV *V-Notch Weir*

Weir El =	261.44 ft	Q (mgd) =	3.45	4.83	14.79	13.25
# Notches =	376.8	H_w (ft) =	0.13	0.14	0.23	0.22
Depth of Notch =	3 in	Weir Submerged?	No	No	No	No
		Notch Overtopped?	No	No	No	No
HGL upstream of PC #1 effluent weir			**261.57**	**261.58**	**261.67**	**261.66**

PC #1 Pier
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	36 in	Q (mgd) =	3.45	4.83	14.79	13.25
Length =	20 ft	V (fps) =	0.76	1.06	3.24	2.90
Hazen & Will. C =	120	Hv (ft) =	0.01	0.02	0.16	0.13
Minor Loss K's : Ent	0.5	Hf (ft) =	0.00	0.00	0.02	0.02
Exit	1.25	Hm (ft) =	0.02	0.03	0.28	0.23
Sum K =	1.75	HL (ft) =	0.02	0.03	0.31	0.25
HGL at bottom of PC #1 Pier			**261.58**	**261.62**	**261.97**	**261.90**

Splitter Box #1 to PC #1
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24 in	Q (mgd) =	3.45	4.83	14.79	13.25
Length =	45 ft	V (fps) =	1.70	2.38	7.29	6.53
Hazen & Will. C =	100	Hv (ft) =	0.04	0.09	0.82	0.66
Minor Loss K's : Ent/Exit	1	Hf (ft) =	0.03	0.06	0.47	0.39
90 Bends	0.3	Hm (ft) =	0.06	0.12	1.15	0.93
Reducer	0.1	HL (ft) =	0.09	0.18	1.63	1.31
Sum K =	1.4					
HGL at Reducer			**261.68**	**261.80**	**263.60**	**263.22**

Splitter Box #1 to PC #1
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30 in	Q (mgd) =	3.45	4.83	14.79	13.25
Length =	90 ft	V (fps) =	1.09	1.52	4.66	4.18
Hazen & Will. C =	100	Hv (ft) =	0.02	0.04	0.34	0.27
Minor Loss K's : Ent/Exit	0.5	Hf (ft) =	0.02	0.04	0.32	0.26
90 Bends	0.3	Hm (ft) =	0.02	0.04	0.41	0.33
45 Bends (1)	0.4	HL (ft) =	0.04	0.08	0.73	0.59
Sum K =	1.2					
HGL at Splitter Box #1			**261.72**	**261.88**	**264.33**	**263.80**

PC #1 Cutthroat Flume

Module CFLU *Cutthroat Flume*

			Q (mgd) =	3.45	4.83	14.79	13.25
Crest El=	261.5	ft	Unsubmerged Ha (ft) =	1.48	1.83	3.76	3.50
Throat Width (ft) =	0.83	ft	Submergence Hb (ft) =	0.22	0.38	2.83	2.30
Length =	9.0	ft	Submergence (%) =	15%	21%	75%	66%
Coefficient K1 =	3.50		Submerged Ha (ft) =	1.00	1.66	1.22	1.75
Exponent n1 =	1.56		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Coefficient K2 =	1.70		Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
Exponent n2 =	1.38		Actual Ha =	1.48	1.83	3.76	3.50
Transitional Subm =	80%		Actual Subm (ft) =	15%	21%	75%	66%
k ent =	0.5		Va (fps) =	1.67	1.88	2.81	2.70
Width at Mth (ft) =	2.83	ft	EGLa (ft) =	263.02	263.39	265.38	265.11
			H at Mouth (ft) =	1.50	1.86	3.81	3.55
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V at Mouth (fps) =	1.26	1.42	2.12	2.04
			EGLm (ft) =	263.02	263.39	265.38	265.12
			Entrance Loss (ft) =	0.01	0.02	0.03	0.03
	HGL Upstream PC1 Flume			**263.03**	**263.41**	**265.42**	**265.15**

G2 Primary Clarifier #2: Influent Flume to Effluent Weir

PC #2 Effluent Weir

Module WV *V-Notch Weir*

			Q (mgd) =	3.45	4.83	14.79	13.25
Weir El =	261.44	ft	H_w (ft) =	0.13	0.14	0.23	0.22
# Notches =	376.8		Weir Submerged?	No	No	No	No
Depth of Notch =	3	in	Notch Overtopped?	No	No	No	No
	HGL upstream of PC #2 effluent weir			**261.57**	**261.58**	**261.67**	**261.66**

PC #2 Pier

Module PIPH *Piping by Hazen-Williams Equation*

			Q (mgd) =	3.45	4.83	14.79	13.25
Diameter =	36	in	V (fps) =	0.76	1.06	3.24	2.90
Length =	20	ft	Hv (ft) =	0.01	0.02	0.16	0.13
Hazen & Will. C =	120		Hf (ft) =	0.00	0.00	0.02	0.02
Minor Loss K's :	Ent	0.5	Hm (ft) =	0.02	0.03	0.28	0.23
	Exit	1.25	HL (ft) =	0.02	0.03	0.31	0.25
	Sum K =	1.75					
	HGL at bottom of PC #1 Pier			**261.58**	**261.62**	**261.97**	**261.90**

Splitter Box #1 to PC #2

Module PIPH *Piping by Hazen-Williams Equation*

			Q (mgd) =	3.45	4.83	14.79	13.25
Diameter =	30	in	V (fps) =	1.09	1.52	4.66	4.18
Length =	130	ft	Hv (ft) =	0.02	0.04	0.34	0.27
Hazen & Will. C =	100		Hf (ft) =	0.03	0.06	0.46	0.38
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hm (ft) =	0.05	0.09	0.84	0.68
	90 Bends (2)	0.6	HL (ft) =	0.08	0.15	1.31	1.06
	45 Bends (1)	0.4					
	Sum K =	2.5					
	HGL at Splitter Box #1			**261.66**	**261.77**	**263.28**	**262.96**

PC #2 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	261.5 ft	Q (mgd) =	3.45	4.83	14.79	13.25
Throat Width (ft) =	0.83 ft	Unsubmerged Ha (ft) =	1.48	1.83	3.76	3.50
Length =	9.0 ft	Submergence Hb (ft) =	0.16	0.27	1.78	1.46
Coefficient K1 =	3.50	Submergence (%) =	11%	15%	47%	42%
Exponent n1 =	1.56	Submerged Ha (ft) =	1.00	1.66	1.22	1.75
Coefficient K2 =	1.70	<If Subm> Iterate 0 >>>	0.000	0.000	0.000	0.000
Exponent n2 =	1.38	Check >>>	Not Subm	Not Subm	Not Subm	Not Subm
Transitional Subm =	80%	Actual Ha =	1.48	1.83	3.76	3.50
k ent =	0.5	Actual Subm (ft) =	11%	15%	47%	42%
Width at Mth (ft) =	2.83 ft	Va (fps) =	1.67	1.88	2.81	2.70
		EGLa (ft) =	263.02	263.39	265.38	265.11
		H at Mouth (ft) =	1.50	1.86	3.81	3.55
		Iterate 0 >>>	0.00	0.00	0.00	0.00
		Check >>>	ok	ok	ok	ok
		V at Mouth (fps) =	1.26	1.42	2.12	2.04
		EGLm (ft) =	263.02	263.39	265.38	265.12
		Entrance Loss (ft) =	0.01	0.02	0.03	0.03
	HGL Upstream PC2 Flume		**263.03**	**263.41**	**265.42**	**265.15**

G3 Primary Clarifier #3: Influent Flume to Effluent Weir

PC #3 Effluent Weir

Module WV *V-Notch Weir*

Weir El =	261.44 ft	Q (mgd) =	3.45	4.83	13.92	0.00
# Notches =	376.8	H_w (ft) =	0.13	0.14	0.22	0.00
Depth of Notch =	3 in	Weir Submerged?	No	No	No	No
		Notch Overtopped?	No	No	No	No
	HGL upstream of PC #3 effluent weir		**261.57**	**261.58**	**261.66**	**261.44**

Influent Well Diffuser Gates

Module FPR *Free Surface Rectangular Port (Assume Gate closed to 12" opng)*

Invert El =	259 ft	Q (mgd) =	0.58	0.81	2.32	0.00
Width=	1 ft	Backwater (ft) =	2.57	2.58	2.66	2.44
Length =	0.25 in	Yc (ft) =	0.29	0.36	0.74	0.00
No. Suppressed Cont =	2 (0 - 3)	Act. Depth (ft) =	2.57	2.58	2.66	2.44
No. Gates =	6	L/P =	0.00	0.00	0.00	0.00
		C_d =	0.62	0.62	0.62	0.62
		V (fps) =	0.35	0.48	1.35	0.00
		HL (ft) =	0.00	0.01	0.07	0.00
	HGL Upstream of Diffuser Ports		**261.57**	**261.59**	**261.73**	**261.44**

Inlet Ports (4 Ports)

Module SPR *Submerged Rectangular Port*

Heighth =	2.5 ft	Q (mgd) =	0.86	1.21	3.48	0.00
Width =	0.67 ft	V (fps) =	0.80	1.12	3.23	0.00
Length =	0.25 in	HL (ft) =	0.03	0.05	0.45	0.00
No. Suppressed Cont =	0 (0 - 3)					
L/P =	0.003					
C_d =	0.60					
No. Ports =	4					
	HGL upstream of ML inlet ports		**261.60**	**261.65**	**262.18**	**261.44**

PC #3 Pier

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30.00 in		Q (mgd) =	3.45	4.83	13.92	0.00
Length =	20 ft		V (fps) =	1.09	1.52	4.39	0.00
Hazen & Will. C =	100		Hv (ft) =	0.02	0.04	0.30	0.00
Minor Loss K's :	Ent	0.5	Hf (ft) =	0.00	0.01	0.06	0.00
	Exit	1.25	Hm (ft) =	0.03	0.06	0.52	0.00
			HL (ft) =	0.04	0.07	0.59	0.00
	Sum K =	1.75					
	HGL at bottom of PC #1 Pier			**261.64**	**261.72**	**262.77**	**261.44**

Splitter Box #1 to PC #3

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24 in		Q (mgd) =	3.45	4.83	13.92	0.00
Length =	60 ft		V (fps) =	1.70	2.38	6.86	0.00
Hazen & Will. C =	100		Hv (ft) =	0.04	0.09	0.73	0.00
Minor Loss K's :	Ent/Exit	1.5	Hf (ft) =	0.04	0.08	0.57	0.00
	90 Bends	0.3	Hm (ft) =	0.09	0.18	1.46	0.00
	45 Bends	0.2	HL (ft) =	0.13	0.26	2.03	0.00
	Sum K =	2					
	HGL at Splitter Box #1			**261.77**	**261.98**	**264.79**	**261.44**

PC #3 Cutthroat Flume

Module CFLU *Cutthroat Flume*

Crest El=	261.5 ft		Q (mgd) =	3.45	4.83	13.92	0.00
Throat Width (ft) =	0.83 ft		Unsubmerged Ha (ft) =	1.48	1.83	3.61	0.00
Length =	9.0 ft		Submergence Hb (ft) =	0.27	0.48	3.29	0.00
Coefficient K1 =	3.50		Submergence (%) =	18%	26%	91%	#DIV/0!
Exponent n1 =	1.56		Submerged Ha (ft) =	1.00	1.66	3.72	1.75
Coefficient K2 =	1.70		<If Subm> Iterate 0 >>>	0.000	0.000	0.000	#DIV/0!
Exponent n2 =	1.38		Check >>>	Not Subm	Not Subm	ok	#DIV/0!
Transitional Subm =	80%		Actual Ha =	1.48	1.83	3.72	0.00
k ent =	0.5		Actual Subm (ft) =	18%	26%	89%	#DIV/0!
Width at Mth (ft) =	2.83 ft		Va (fps) =	1.67	1.88	2.67	#DIV/0!
			EGLa (ft) =	263.02	263.39	265.33	#DIV/0!
			H at Mouth (ft) =	1.50	1.86	3.77	0.85
			Iterate 0 >>>	0.00	0.00	0.00	#DIV/0!
			Check >>>	ok	ok	ok	#DIV/0!
			V at Mouth (fps) =	1.26	1.42	2.02	0.00
			EGLm (ft) =	263.02	263.39	265.33	262.35
			Entrance Loss (ft) =	0.01	0.02	0.03	0.00
	HGL Upstream PC3 Flume			**263.03**	**263.41**	**265.36**	**261.44**

Summary of HGLs in Splitter Box 1

HGL upstream of PC #1 flume	**263.03**	**263.41**	**265.42**	**265.15**
HGL upstream of PC #2 flume	**263.03**	**263.41**	**265.42**	**265.15**
HGL upstream of PC #3 flume	**263.03**	**263.41**	**265.36**	**261.44**
HGL Upstream	**263.03**	**263.41**	**265.42**	**265.15**
TOW 265.98 **Freeboard**	**2.95**	**2.57**	**0.56**	**0.83**

SEGMENT H: INFLUENT PARSHALL FLUME TO SPLITTER BOX 1

Segment Flow =	**10.35**	**14.50**	**56.00**
HGL for Upstream Calculations	**263.03**	**263.41**	**265.50**

H1 Parallel Flow Path (Carries portion of flow from JB3 to SB1)

			14.00
Path H2 Flow (mgd) =	2.59	3.65	14.33
Path H1 Flow (mgd) =	7.76	10.85	41.67
	0.001	0.000	-0.004
RS Piping from JB3 to Splitter Box 1	0.333762887	0.33640553	0.343892489

Module PPIPH *Parallel Pipes by Hazen-Williams Equation*

Pipe 1

Diameter =	30	in	Pipe 1 Q (mgd) =	5.11	7.15	27.41
Length =	340	ft	Iterate 0 >>>	0.000	0.000	0.000
Hazen & Will. C =	110		Check >>>	ok	ok	ok
Minor Loss K's :	Entr (1) / Exit (1)	1.5	V (fps) =	1.61	2.25	8.64
	45 Bends (3)	0.6	Hv (ft) =	0.04	0.08	1.16
	22.5 Bends (1)	0.15	Hf (ft) =	0.14	0.26	3.18
	11.25 Bends (2)	0.2	Hm (ft) =	0.10	0.19	2.84
	Sum K =	2.45	HL (ft) =	0.24	0.46	6.02

Pipe 2

Diameter =	24	in	Pipe 2 Q (mgd) =	2.65	3.70	14.26
Length =	350	ft	V (fps) =	1.30	1.82	7.02
Hazen & Will. C =	100		Hv (ft) =	0.03	0.05	0.77
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.15	0.28	3.45
	90 at Tee (1)	1.5	Hm (ft) =	0.09	0.17	2.57
	22.5 Bends (1)	0.15	HL (ft) =	0.24	0.46	6.02
	11.25 Bends (2)	0.2				
	Sum K =	3.35				

HGL at Junction Box #2	**263.28**	**263.86**	**271.52**

RS Piping from Junction Box #3 to Junction Box #2

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	48	in	Q (mgd) =	2.59	3.65	14.33
Length =	180	ft	V (fps) =	0.32	0.45	1.77
Hazen & Will. C =	120		Hv (ft) =	0.00	0.00	0.05
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.00	0.00	0.04
	45 Bends (2)	0.4	Hm (ft) =	0.00	0.01	0.10
	22.5 Bends (1)	0.15	HL (ft) =	0.01	0.01	0.14
	Sum K =	2.05				

HGL at Junction Box #3	**263.28**	**263.87**	**271.66**

H2 Parallel Flow Path (Carries portion of flow from JB3 to SB1)

RS Piping from Junction Box #3 to Splitter Box #1

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24	in	Q (mgd) =	2.59	3.65	14.33
Length =	460	ft	V (fps) =	1.28	1.80	7.06
Hazen & Will. C =	100		Hv (ft) =	0.03	0.05	0.77
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.19	0.36	4.58
	45 Bends (2)	0.4	Hm (ft) =	0.05	0.10	1.59
	22.5 Bends (1)	0.15	HL (ft) =	0.25	0.47	6.17
	Sum K =	2.05				

HGL at Junction Box #3	**263.28**	**263.87**	**271.67**

End Parallel Flow Paths

Headworks to Junction Box #3

Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	60	in	Q (mgd) =	10.35	14.50	56.00
Length =	45	ft	V (fps) =	0.82	1.14	4.41
Hazen & Will. C =	120		Hv (ft) =	0.01	0.02	0.30
Minor Loss K's :	Entr (1) / Exit (1)	1.5	Hf (ft) =	0.00	0.00	0.05
			Hm (ft) =	0.02	0.03	0.45
			HL (ft) =	0.02	0.03	0.50
	Sum K =	1.5				
	HGL at Headworks effluent			**263.30**	**263.91**	**272.17**

Port from Common Effluent Box to Final Effluent Box

Module SPR *Submerged Rectangular Port*

Heighth =	4	ft	Q (mgd) =	10.35	14.50	56.00
Width =	7	ft	V (fps) =	0.57	0.80	3.09
Length =	12	in	HL (ft) =	0.01	0.02	0.36
No. Suppressed Cont =	2	(0 - 3)				
L/P =	0.045					
C_d =	0.65					
	HGL downstream of Flume			**263.31**	**263.93**	**272.52**

Parshall Flume

Module PFLU *Parshall Flume*

Crest El=	271	ft	Q (mgd) =	10.35	14.5	56.00
Throat Width (ft) =	5	ft	Ha (ft) =	0.87	1.07	2.52
Capacity (mgd) =	55.3	mgd	Submergence (ft) =	0.00	0.00	1.52
Width at Mth (ft) =	7.55	ft	Submergence (%) =	0%	0%	60%
k ent =	0		Va (fps) =	2.75	3.11	5.13
			EGLa (ft) =	271.99	272.23	273.93
			H at Mouth (ft) =	0.90	1.12	2.64
			Iterate 0 >>>	0.00	0.00	0.00
			Check >>>	ok	ok	ok
			V at Mouth (fps) =	2.35	2.66	4.35
			Entrance Loss (ft) =	0.00	0.00	0.00
	HGL upstream of parshall flume			**271.99**	**272.23**	**273.93**

SEGMENT I: HEADWORKS INFLUENT CHANNEL TO INFLUENT PARSHALL FLUME

Headworks

Segment Flow =	**10.35**	**14.50**	**69.90**	**46.60**

Parshall Flume — Assume Flume Unsubmerged

Module PFLU *Parshall Flume*

Crest El=	271 ft	Q (mgd) =	10.35	14.50	69.90	46.60
Throat Width (ft) =	5 ft	Ha (ft) =	0.87	1.07	2.90	2.24
Capacity (mgd) =	55.3 mgd	Submergence (ft) =	0.00	0.00	0.00	0.00
Width at Mth (ft) =	7.55 ft	Submergence (%) =	0%	0%	0%	0%
k ent =	0	Va (fps) =	2.75	3.11	5.57	4.80
		EGLa (ft) =	271.99	272.23	274.38	273.60
		H at Mouth (ft) =	0.90	1.12	3.04	2.35
		Iterate 0 >>>	0.00	0.00	0.00	0.00
		Check >>>	ok	ok	ok	ok
		V at Mouth (fps) =	2.35	2.66	4.71	4.07
		Entrance Loss (ft) =	0.00	0.00	0.00	0.00
HGL upstream of parshall flume			**271.99**	**272.23**	**274.38**	**273.60**

Channel Contraction to Parshall Flume

Module OCCX *Open Channel Contraction / Expansion*

Invert ds =	271 ft	Q (mgd) =	10.35	14.5	69.9	46.6
Width ds =	7.55 ft	Yc (ft) =	0.36	0.45	1.29	0.98
Invert us =	271 ft	Depth ds (ft) =	0.99	1.23	3.38	2.60
Width us =	13 ft	V ds (fps) =	2.15	2.42	4.24	3.67
k =	0.5	Depth us (ft) =	1.06	1.32	3.68	2.82
		Iterate 0 >>>	0.00	0.00	0.00	0.00
		Check >>>	ok	ok	ok	ok
		V us (fps) =	1.16	1.30	2.26	1.96
HGL upstream of contraction			**272.06**	**272.32**	**274.68**	**273.82**

Channel Upstream of Parshall Flume

Module OCR *Short Rectangular Open Channel*

Invert El =	270.25	Q (mgd) =	10.35	14.5	69.9	46.6
Width =	13.00 ft	Backwater (ft) =	1.81	2.07	4.43	3.57
Manning n =	0.015	Yc (ft) =	0.36	0.45	1.29	0.98
Length =	5.00 ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000 ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50	Area (sf) =	15.27	19.46	20.04	20.04
Downstream Conditions		Actual Depth (ft) =	1.81	2.07	4.43	3.57
		Area (sf) =	23.56	26.95	57.56	46.47
		V (fps) =	0.68	0.83	1.88	1.55
		Sf (ft/ft) =	0.0000	0.0000	0.0001	0.0001
Upstream Conditions		Upstrm Depth (ft) =	1.81	2.07	4.43	3.58
		Iterate 0 >>>	0.000	0.000	0.001	0.000
		Check >>>	ok	ok	ok	ok
		Area (sf) =	23.57	26.95	57.57	46.48
		V (fps) =	0.68	0.83	1.88	1.55
		Sf (ft/ft)=	0.0000	0.0000	0.0001	0.0001
		Hm (ft) =	0.00	0.01	0.03	0.02
HGL at end of vortex unit effluent channel			**272.07**	**272.34**	**274.76**	**273.88**

Conduit downstream of Baffle

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	13	ft	Q (mgd) =	10.35	14.50	69.90	46.60
Height =	3	ft	V (fps) =	0.41	0.58	2.77	1.85
Length =	8	ft	Hv (ft) =	0.00	0.01	0.12	0.05
Manning n =	0.015		Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5		Hm (ft) =	0.00	0.01	0.18	0.08
			HL (ft) =	0.00	0.01	0.18	0.08
				272.08	**272.35**	**274.94**	**273.96**

Submerged Port Below Baffle

Module SPR *Submerged Rectangular Port*

Heighth =	3	ft	Q (mgd) =	10.35	14.50	69.90	46.60
Width =	13	ft	V (fps) =	0.41	0.58	2.77	1.85
Length =	12	in	HL (ft) =	0.01	0.01	0.28	0.13
No. Suppressed Cont =	3	(0 - 3)					
L/P =	0.031						
C_d =	0.65						
EGL/HGL		at		**272.08**	**272.36**	**275.23**	**274.09**

Conduit upstream of Baffle

Module CCR *Rectangular Closed Conduit by Mannings Equation*

Width =	13	ft	Q (mgd) =	10.35	14.50	69.90	46.60
Height =	6	ft	V (fps) =	0.21	0.29	1.39	0.92
Length =	8	ft	Hv (ft) =	0.00	0.00	0.03	0.01
Manning n =	0.015		Hf (ft) =	0.00	0.00	0.00	0.00
SumK =	1.5		Hm (ft) =	0.00	0.00	0.04	0.02
			HL (ft) =	0.00	0.00	0.05	0.02
				272.08	**272.36**	**275.27**	**274.11**

Common Effluent Channel to Flow Diversion Box

Module OCR *Short Rectangular Open Channel*

Invert El =	270.25		Q (mgd) =	10.35	14.50	69.90	46.60
Width =	6.00	ft	Backwater (ft) =	1.83	2.11	5.02	3.86
Manning n =	0.015		Yc (ft) =	0.61	0.76	2.16	1.65
Length =	28.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	7.05	8.98	9.25	9.25
Downstream Conditions			Actual Depth (ft) =	1.83	2.11	5.02	3.86
			Area (sf) =	11.01	12.67	30.15	23.16
			V (fps) =	1.45	1.77	3.59	3.11
			Sf (ft/ft) =	0.0002	0.0002	0.0006	0.0005
Upstream Conditions			Upstrm Depth (ft) =	1.84	2.12	5.04	3.87
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	11.04	12.71	30.25	23.25
			V (fps) =	1.45	1.76	3.57	3.10
			Sf (ft/ft)=	0.0002	0.0002	0.0006	0.0005
			Hm (ft) =	0.02	0.02	0.10	0.07
HGL at beginning of common effluent channel				**272.14**	**272.44**	**275.59**	**274.35**

Grit Collectors

Plant Flow =	**10.35**	**14.5**	**69.9**	**46.6**
# Bar Screens in Service =	**1**	**1**	**2**	**1**
Flow per Grit Collector =	**10.35**	**14.50**	**34.95**	**46.60**

Grit Collector Effluent Channel to Common Effluent Channel

Module OCR *Short Rectangular Open Channel*

Invert El =	270.25		Q (mgd) =	10.35	14.50	34.95	46.60
Width =	4.50	ft	Backwater (ft) =	1.89	2.19	5.34	4.10
Manning n =	0.015		Yc (ft) =	0.73	0.92	1.65	2.00
Length =	6.50	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	5.28	6.74	6.94	6.94
			Downstream Conditions				
			Actual Depth (ft) =	1.89	2.19	5.34	4.10
			Area (sf) =	8.50	9.86	24.03	18.44
			V (fps) =	1.88	2.28	2.25	3.91
			Sf (ft/ft) =	0.0003	0.0005	0.0003	0.0009
			Upstream Conditions				
			Upstrm Depth (ft) =	1.89	2.19	5.34	4.11
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	8.51	9.87	24.03	18.47
			V (fps) =	1.88	2.27	2.25	3.90
			Sf (ft/ft)=	0.0003	0.0005	0.0003	0.0009
			Hm (ft) =	0.03	0.04	0.04	0.12
HGL at end of vortex unit effluent channel				**272.22**	**272.56**	**275.71**	**274.71**

Grit Collector Effluent Channel

Module OCR *Short Rectangular Open Channel*

Invert El =	270.25		Q (mgd) =	10.35	14.50	34.95	46.60
Width =	9.00	ft	Backwater (ft) =	1.97	2.31	5.46	4.46
Manning n =	0.015		Yc (ft) =	0.46	0.58	1.04	1.26
Length =	7.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	10.57	13.47	13.87	13.87
			Downstream Conditions				
			Actual Depth (ft) =	1.97	2.31	5.46	4.46
			Area (sf) =	17.76	20.83	49.13	40.14
			V (fps) =	0.90	1.08	1.10	1.80
			Sf (ft/ft) =	0.0001	0.0001	0.0000	0.0001
			Upstream Conditions				
			Upstrm Depth (ft) =	1.97	2.31	5.46	4.46
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	17.77	20.83	49.13	40.15
			V (fps) =	0.90	1.08	1.10	1.80
			Sf (ft/ft)=	0.0001	0.0001	0.0000	0.0001
			Hm (ft) =	0.01	0.01	0.01	0.03
HGL in Grit Collector				**272.24**	**272.59**	**275.74**	**274.79**

Grit Collector Losses

per mfr

HL (ft) =	0.12	0.12	0.12	0.12
HGL Upstream of Screens	**272.36**	**272.71**	**275.86**	**274.91**

Grit Collector Influent Channel

Module OCR *Short Rectangular Open Channel*

Invert El =	269.00		Q (mgd) =	10.35	14.50	34.95	46.60
Width =	4.50	ft	Backwater (ft) =	3.24	3.59	6.74	5.79
Manning n =	0.015		Yc (ft) =	0.73	0.92	1.65	2.00
Length =	5.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	5.28	6.74	6.94	6.94
Downstream Conditions			Actual Depth (ft) =	3.24	3.59	6.74	5.79
			Area (sf) =	14.59	16.16	30.32	26.04
			V (fps) =	1.10	1.39	1.78	2.77
			Sf (ft/ft) =	0.0001	0.0001	0.0002	0.0004
Upstream Conditions			Upstrm Depth (ft) =	3.24	3.59	6.74	5.79
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	14.60	16.17	30.32	26.05
			V (fps) =	1.10	1.39	1.78	2.77
			Sf (ft/ft)=	0.0001	0.0001	0.0002	0.0004
			Hm (ft) =	0.01	0.01	0.02	0.06
HGL at end of vortex unit effluent channel				**272.27**	**272.64**	**275.81**	**274.97**

Influent Screening

Plant Flow =	**10.4**	**14.5**	**69.9**	**46.6**
# Bar Screens in Service =	**1**	**1**	**3**	**2**
Flow per Screen =	**10.35**	**14.50**	**23.30**	**23.30**

Channel Between Screening and Grit

Module OCR *Short Rectangular Open Channel*

Invert El =	269.00		Q (mgd) =	10.35	14.50	34.95	46.60
Width =	5.00	ft	Backwater (ft) =	3.27	3.64	6.81	5.97
Manning n =	0.015		Yc (ft) =	0.68	0.86	1.54	1.86
Length =	16.50	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	5.87	7.48	7.71	7.71
Downstream Conditions			Actual Depth (ft) =	3.27	3.64	6.81	5.97
			Area (sf) =	16.36	18.19	34.06	29.83
			V (fps) =	0.98	1.23	1.59	2.42
			Sf (ft/ft) =	0.0001	0.0001	0.0001	0.0003
Upstream Conditions			Upstrm Depth (ft) =	3.27	3.64	6.81	5.97
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	16.36	18.19	34.07	29.86
			V (fps) =	0.98	1.23	1.59	2.41
			Sf (ft/ft)=	0.0001	0.0001	0.0001	0.0003
			Hm (ft) =	0.01	0.01	0.02	0.05
HGL in channel				**272.29**	**272.67**	**275.87**	**275.11**

Screening Channel (Downstream of Screen)

Module OCR *Short Rectangular Open Channel*

Invert El =	270.50		Q (mgd) =	10.35	14.50	23.30	23.30
Width =	4.00	ft	Backwater (ft) =	1.79	2.17	5.37	4.61
Manning n =	0.015		Yc (ft) =	0.79	0.99	1.36	1.36
Length =	12.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	4.70	5.99	6.17	6.17
Downstream Conditions			Actual Depth (ft) =	1.79	2.17	5.37	4.61
			Area (sf) =	7.18	8.70	21.49	18.43
			V (fps) =	2.23	2.58	1.68	1.96
			Sf (ft/ft) =	0.0005	0.0006	0.0002	0.0003
Upstream Conditions			Upstrm Depth (ft) =	1.80	2.18	5.37	4.61
			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	7.21	8.73	21.50	18.44
			V (fps) =	2.22	2.57	1.68	1.95
			Sf (ft/ft)=	0.0005	0.0006	0.0002	0.0002
			Hm (ft) =	0.04	0.05	0.02	0.03
EGL downstream of barscreen				**272.42**	**272.84**	**275.94**	**275.20**

EGL to HGL

Module OCREH *Rectangular Channel EGL to HGL Conversion*

Invert =	270.5	ft	Q (mgd) =	10.35	14.50	23.30	23.30
Width =	4	ft	Yc (ft) =	0.79	0.99	1.36	1.36
			E (ft) =	1.92	2.34	5.44	4.70
			Depth (ft) =	1.84	2.24	5.40	4.64
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V (fps) =	2.17	2.50	1.67	1.94
HGL				**272.34**	**272.74**	**275.90**	**275.14**

Bar Screen

			HL (ft) =	1.00	1.00	1.00	1.00
HGL Upstream of Screens				**273.34**	**273.74**	**276.90**	**276.14**

Screening Channel (Upstream of Screen)

Module OCR *Short Rectangular Open Channel*

Invert El =	270.50		Q (mgd) =	10.35	14.50	23.30	23.30
Width =	4.00	ft	Backwater (ft) =	2.84	3.24	6.40	5.64
Manning n =	0.015		Yc (ft) =	0.79	0.99	1.36	1.36
Length =	4.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.50		Area (sf) =	4.70	5.99	6.17	6.17
			Actual Depth (ft) =	2.84	3.24	6.40	5.64
Downstream Conditions			Area (sf) =	11.37	12.96	25.59	22.56
			V (fps) =	1.41	1.73	1.41	1.60
			Sf (ft/ft) =	0.0002	0.0002	0.0001	0.0002
			Upstrm Depth (ft) =	2.84	3.24	6.40	5.64
Upstream Conditions			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	11.38	12.96	25.59	22.57
			V (fps) =	1.41	1.73	1.41	1.60
			Sf (ft/ft)=	0.0002	0.0002	0.0001	0.0002
			Hm (ft) =	0.02	0.02	0.02	0.02
HGL in Screening Influent Channel				**273.39**	**273.81**	**276.94**	**276.20**

Common Influent Channel

Module OCR *Short Rectangular Open Channel*

Invert El =	270.50		Q (mgd) =	10.35	14.50	46.60	46.60
Width =	5.00	ft	Backwater (ft) =	2.89	3.31	6.44	5.70
Manning n =	0.015		Yc (ft) =	0.68	0.86	1.86	1.86
Length =	16.00	ft	Yn (ft) =	1.17	1.50	1.54	1.54
Slope =	0.0000	ft/ft	Iterate 0 >>>	N/A	N/A	N/A	N/A
K-ent =	0.00		Area (sf) =	5.87	7.48	7.71	7.71
			Actual Depth (ft) =	2.89	3.31	6.44	5.70
Downstream Conditions			Area (sf) =	14.45	16.55	32.22	28.50
			V (fps) =	1.11	1.36	2.24	2.53
			Sf (ft/ft) =	0.0001	0.0001	0.0002	0.0003
			Upstrm Depth (ft) =	2.89	3.31	6.45	5.71
Upstream Conditions			Iterate 0 >>>	0.000	0.000	0.000	0.000
			Check >>>	ok	ok	ok	ok
			Area (sf) =	14.46	16.56	32.24	28.53
			V (fps) =	1.11	1.35	2.24	2.53
			Sf (ft/ft)=	0.0001	0.0001	0.0002	0.0003
			Hm (ft) =	0.00	0.00	0.00	0.00
EGL at Influent Box				**273.41**	**273.84**	**277.03**	**276.31**

EGL to HGL

Module OCREH *Rectangular Channel EGL to HGL Conversion*

Invert =	270.5	ft	Q (mgd) =	10.35	14.50	46.60	46.60
Width =	5	ft	Yc (ft) =	0.68	0.86	1.86	1.86
			E (ft) =	2.91	3.34	6.53	5.81
			Depth (ft) =	2.89	3.31	6.45	5.71
			Iterate 0 >>>	0.00	0.00	0.00	0.00
			Check >>>	ok	ok	ok	ok
			V (fps) =	1.11	1.35	2.24	2.53
HGL				**273.39**	**273.81**	**276.95**	**276.21**

Port to Common Influent Channel

Module FPR *Free Surface Rectangular Port*

Invert El =	270.5	ft	Q (mgd) =	10.35	14.50	46.60	46.60
Width=	4	ft	Backwater (ft) =	2.89	3.31	6.45	5.71
Length =	12	in	Yc (ft) =	0.79	0.99	2.16	2.16
No. Suppressed Cont =	3	(0 - 3)	Act. Depth (ft) =	2.89	3.31	6.45	5.71
			L/P =	0.10	0.09	0.06	0.06
			C_d =	0.69	0.68	0.66	0.67
			V (fps) =	1.38	1.69	2.80	3.16
			HL (ft) =	0.06	0.10	0.28	0.35
HGL at Influent Box				**273.45**	**273.91**	**277.22**	**276.55**
TOW	**277.4**		**Freeboard**	**3.95**	**3.49**	**0.18**	**0.85**

Orange Water and Sewer Authority (OWASA)
Mason Farm WWTP
Hydraulic Profile

PRIMARY CLARIFIER BYPASS PIPING

HGL in Intermediate PS	**249.00**	**249.00**

Trickling Filter to Int PS
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30	in	Q (mgd) =	16.5	17.2
Length =	280	ft	V (fps) =	5.20	5.42
Hazen & Will. C =	100		Hv (ft) =	0.42	0.46
Minor Loss K's :	Ent/Exit	6	Hf (ft) =	1.22	1.32
	45 Bends	0.4	Hm (ft) =	2.69	2.92
			HL (ft) =	3.91	4.24
	Sum K =	6.4			

HGL at Trickling Filter	**252.91**	**253.24**

Headworks to Trickling Filter
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	30	in	Q (mgd) =	16.5	17.2
Length =	20	ft	V (fps) =	5.20	5.42
Hazen & Will. C =	100		Hv (ft) =	0.42	0.46
Minor Loss K's :	Ent/Exit	1.5	Hf (ft) =	0.09	0.09
			Hm (ft) =	0.63	0.69
			HL (ft) =	0.72	0.78
	Sum K =	1.5			

HGL in piping	**253.63**	**254.02**

Headworks to Trickling Filter
Module PIPH *Piping by Hazen-Williams Equation*

Diameter =	24	in	Q (mgd) =	16.5	17.2
Length =	470	ft	V (fps) =	8.13	8.47
Hazen & Will. C =	100		Hv (ft) =	1.03	1.12
Minor Loss K's :	Ent/Exit	7.5	Hf (ft) =	6.07	6.56
		1.5	Hm (ft) =	10.77	11.71
		1.5	HL (ft) =	16.85	18.27
	Sum K =	10.5			

HGL at Headworks effluent box	**270.48**	**272.29**

PC Bypass Gate (If not Submerged)

Module FPR *Free Surface Rectangular Port*

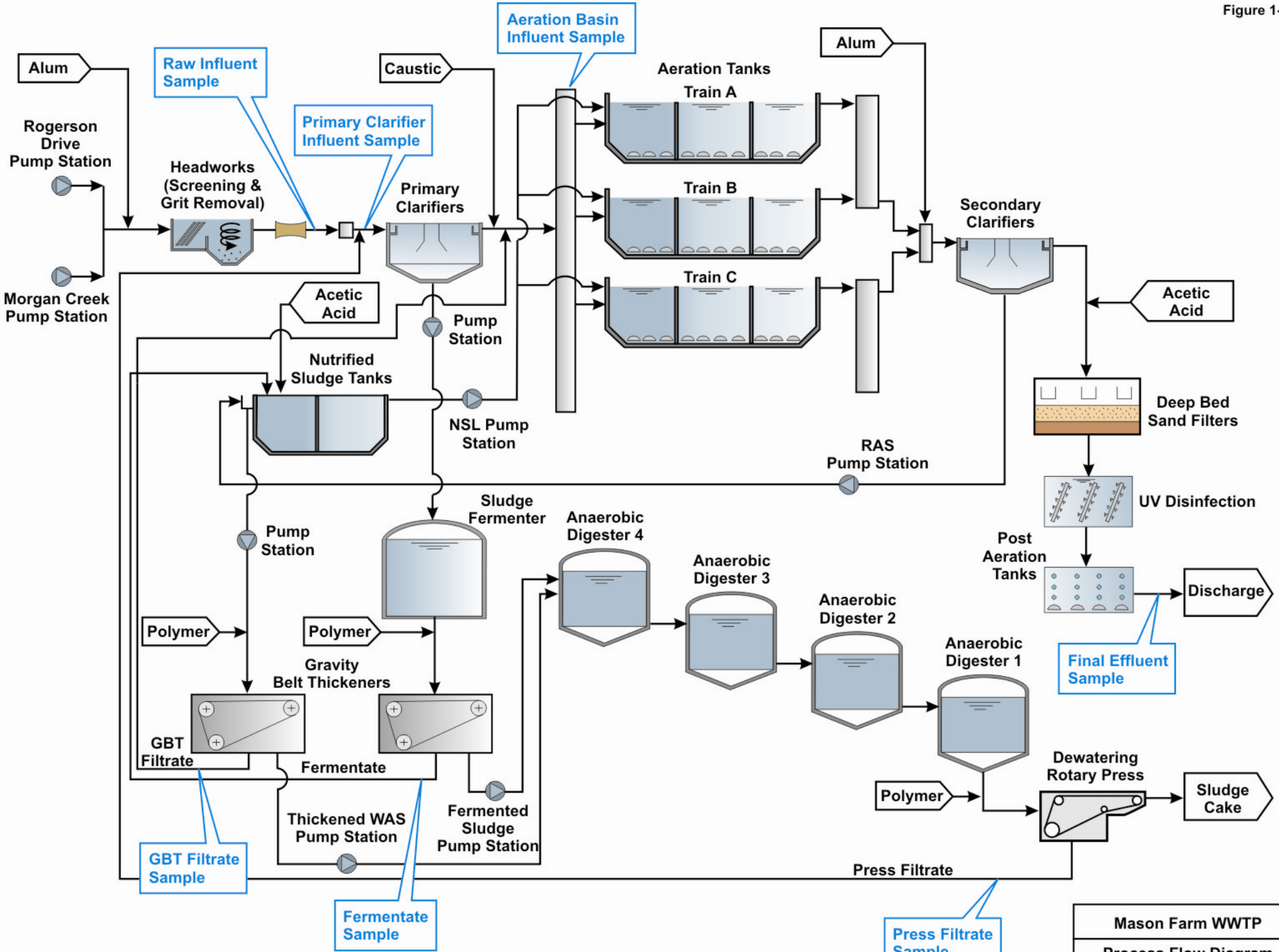
Invert El =	267.5	ft	Q (mgd) =	16.50	17.20
Width=	4	ft	Backwater (ft) =	2.98	4.79
Length =	0	in	Yc (ft) =	1.08	1.11
No. Suppressed Cont =	0	(0 - 3)	Act. Depth (ft) =	2.98	4.79
			L/P =	0.00	0.00
			C_d =	0.60	0.60
			V (fps) =	2.14	1.39
			HL (ft) =	0.20	0.08
	HGL Downstream of Flume			**270.67**	**272.37**

PC Bypass Gate (if Submerged)

Module SPR *Submerged Rectangular Port*

Heighth =	3	ft	Q (mgd) =	16.5	17.2
Width =	4	ft	V (fps) =	2.13	2.22
Length =	12	in	HL (ft) =	0.16	0.18
No. Suppressed Cont =	2	(0 - 3)			
L/P =	0.071				
C_d =	0.66				
	HGL Downstream of Flume			**270.64**	**272.46**
	Actual HGL Downstream of Flume			**270.67**	**272.46**
			Crest of Flume	**271.00**	
	Max Submergence of Flume @ 43.5 mgd				**272.50**

Figure 1-1



Mason Farm WWTP

Process Flow Diagram and Sampling Locations

30853-014-Figure 1-1.cdr

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Sarah Lothman, Joe Rohrbacher

DATE: October 26, 2009
Revised February 15, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum 3-A – Supplemental Sampling Results

1. INTRODUCTION

Special sampling was conducted at the Mason Farm WWTP from Sunday, August 16 through Saturday, August 22. Sampling included detailed influent wastewater characterization to determine carbon, nitrogen, and phosphorus speciation and two days of nutrient profiles through the activated sludge process. Raw influent, primary clarifier influent, aeration basin influent, and final effluent composite samples were collected by Mason Farm WWTP staff and analyzed by an outside laboratory. Gravity belt thickener filtrate, fermentate, and rotary press filtrate samples were manually composited by Hazen and Sawyer. Nutrient profile samples were collected, field filtered and prepared by Hazen and Sawyer. All samples were analyzed by the outside laboratory, with the exception of ammonia, nitrate, nitrite, phosphate, chemical oxygen demand (COD), alkalinity, and pH, which were analyzed by Hazen and Sawyer. A process flow diagram showing liquid and sludge treatment streams and sampling locations is shown in Figure 1-1.

2. COMPOSITE SAMPLING

Influent and effluent wastewater characteristics were determined through daily, 24-hour composite sampling. The raw influent composite samples were collected upstream of the rotary press filtrate recycle stream. Table 2-1 summarizes the results of the wastewater characterization for raw influent, primary clarifier effluent, the aeration basin influent channel and the final effluent.

Table 2-1
Composite Sampling Wastewater Characterization

Parameter	Raw Influent (mg/L)	Primary Clarifier Effluent (mg/L)	Aeration Basin Influent Channel (mg/L)	Final Effluent (mg/L)
TSS	180	148	95	<1
VSS	162	126	76	-
BOD_{XX}[1]	234	210	112	-
BOD_{XG}[1]	94	86	64	-
COD_{XX}[1]	552	481	241	19.7
COD_{XG}[1]	201	179	103	15.0
COD_{XM}[1]	96	102	77	15.7
COD_{XF}[1]	72	72	59	-
TKN_{XX}[1]	38.2	43.2	31.8	1.15
TKN_{XG}[1]	31.8	36.5	27.3	1.01
NH_3-N_{XG}[1]	31.7	28.6	22.7	0.06
NO_x	0.04	0.13	2.08	16.3
NO_2-N	-	-	-	0.03
TP_{XX}[1]	5.42	5.49	4.29	0.69
TP_{XG}[1]	3.87	2.48	2.19	0.67
Ca	12.9	-	-	-
Mg	4.5	-	-	-

1 XX = sample was not filtered, value represents total value of the parameter
XG = filtered with 1.2 µm filter to include soluble and colloidal components
XM = filtered with 0.45 µm membrane filter and represents soluble component
XF = flocculated and filtered with 0.45 µm membrane filter and used to derive readily biodegradable component

It can be observed that the final effluent COD_{xm} value is greater than the COD_{xg} value. This is theoretically impossible and the difference is likely attributed to error during analysis. Table 2-2 presents wastewater ratios for raw influent, primary clarifier effluent, and the aeration basin influent channel. Particulate fractions of COD, TKN, and TP were calculated by taking the ratio of particulate COD, TKN, and TP, the difference between the total and soluble concentrations (what is not removed through a glass/membrane filter), to total COD, TKN, and TP.

Table 2-2
Composite Sampling Wastewater Ratios

Ratio	Raw Influent	Primary Clarifier Effluent	Aeration Basin Influent Channel
COD/BOD	2.40	2.24	2.12
COD_{XG}/COD_T	0.38	0.47	0.47
COD_{XF}/COD_T	0.13	0.18	0.27
COD_P/COD_T	0.62	0.62	0.53
BOD_{XG}/BOD_T	0.43	0.45	0.58
BOD/TSS	1.42	1.57	1.36
VSS/TSS	0.90	0.86	0.81
COD/TKN	14.56	11.11	7.36
COD/NH_3	19.41	17.02	10.69
COD/TP	101.12	87.86	59.92
BOD/TKN	6.10	4.92	3.52
BOD/NH_3	8.14	7.41	4.99
BOD/TP	42.75	39.80	29.32
COD_P/VSS	2.25	2.38	1.74
NH_3/TKN	0.81	0.68	0.73
PO_4-P/TP	0.70	0.47	0.55
TKN_P/VSS	0.038	0.063	0.060
TP_P/VSS	0.010	0.025	0.024
TKN_P/COD_P	0.018	0.021	0.039
TP_P/COD_P	0.004	0.015	0.015
cBOD/BOD	0.944	0.938	0.809
COD/cBOD	2.55	2.38	2.66

The results of the special sampling were used to develop COD, nitrogen, and phosphorus fractions for input into the BioWin process model. Raw influent wastewater fractions for the Mason Farm WWTP model are summarized in Table 2-3.

Table 2-3
Raw Influent Wastewater Fractions

Fraction	BioWin Default	Raw Influent
Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD]	0.16	0.10
Fac - Acetate [gCOD/g of readily biodegradable COD]	0.15	0.15
Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0.75	0.90
Fus - Unbiodegradable soluble [gCOD/g of total COD]	0.05	0.03
Fup - Unbiodegradable particulate [gCOD/g of total COD]	0.13	0.24
Fna - Ammonia [gNH3-N/gTKN]	0.66	0.74
Fnox - Particulate organic nitrogen [gN/g Organic N]	0.50	0.60
Fnus - Soluble unbiodegradable TKN [gN/gTKN]	0.02	0.01
FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD]	0.035	0.018
Fpo4 - Phosphate [gPO4-P/gTP]	0.50	0.70
FupP - P:COD ratio for influent unbiodegradable part. COD [gP/gCOD]	0.011	0.004

3. BIOLOGICAL TREATMENT FACILITIES NUTRIENT PROFILES

Ammonia, nitrate, nitrite, and phosphorus were profiled twice a day, in the morning and in the afternoon, on Tuesday, August 18 and Thursday, August 20. In addition, dissolved oxygen concentration, temperature, and pH were recorded during sample collection. During the sampling period, the plant was operating one nutrified sludge train and three trains in the aeration basin. The nutrified sludge train consisted of two cells in series; each train in the aeration basin consisted of flow through a combined influent channel, three cells in series, and a combined effluent channel. Samples were taken at the following locations: nutrified sludge basin – end cell 1C, nutrified sludge basin – end cell 1D, aeration basin influent channel, end cell 1E, end cell 1F, end cell 2B, end cell 3A, end cell 2D, end cell 2C, end cell 3D, end cell 4A, end cell 4B, aeration basin effluent channel 1 (Channel 5B per Figure 1), and aeration basin effluent channel 2 (Channel 5A per Figure 1). The series of cells 1E, 1F, and 2B is referred to as Train A in this memorandum. Similarly, cells 3A, 2D, and 2C are Train B and 3D, 4A, 4B are Train C.

Figure 3-1
Biological Process Schematic

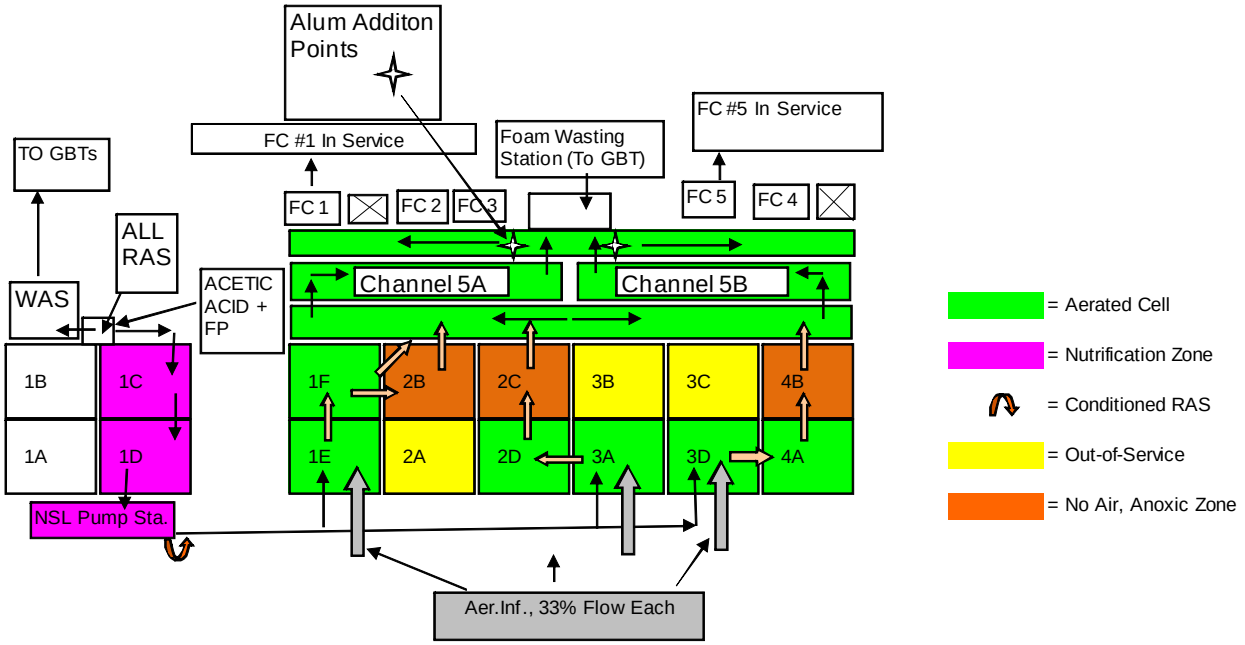


Table 3-1 summarizes the plant's operational data for the two sampling days. In this table, trains A, B, and C are averaged together (1st cell, 2nd cell, 3rd cell). Daily nutrient profiles of this information are presented in Figures 3-1 through 3-7. In order to see varying performance in each of the three basins, Table 3-2 and figures 3-8 through 3-13 show a summary of all measurements gathered from the aeration basins, in trains A, B, and C on both sampling days

Table 3-1
Summary of Plant Operational Data

Morning Average	Abbrev.	Temp.	DO	pH	NH_3-N	NO_3-N	NO_2-N	PO_4
Raw Influent	RI	25.8	-	7.1	28.9	0.374	-	8.02
Primary Clarifier Influent	PCI	26.0	-	7.0	32.6	0.357	-	5.85
Nutrified Basin Cell 1C	NS1C	28.1	0.17	6.7	1.12	0.329	0.011	4.40
Nutrified Basin Cell 1D	NS1D	28.3	0.15	6.7	3.08	0.198	0.027	17.9
Aeration Basin Influent	ABI	26.4	5.23	9.0	23.9	0.451	-	5.55
1st Cell	1st	27.4	4.35	6.9	1.51	13.0	0.350	0.993
2nd Cell	2nd	27.6	2.13	6.7	0.036	15.0	0.006	0.551
3rd Cell	3rd	27.7	0.16	6.7	0.429	13.7	0.056	0.864
Effluent Channel 1	EF1	27.8	5.67	6.8	0.032	14.5	0.008	0.610
Secondary Clarifier Effluent	SCE	27.9	-	6.7	0.127	16.5	0.049	0.511
Filtered Effluent	FE	27.4	-	7.2	0.028	13.7	0.008	0.698
RAS	RAS	28.0	0.10	6.6	0.320	14.7	-	0.648
Filter Backwash	FB	27.8	-	7.5	0.420	14.3	-	0.684

Afternoon Average	Abbrev.	Temp.	DO	pH	NH_3-N	NO_3-N	NO_2-N	PO_4-P
Raw Influent	RI	27.4	-	7.35	41.6	0.366	-	9.88
Primary Clarifier Influent	PCI	28.1	-	6.95	20.1	0.563	-	4.99
Nutrified Basin Cell 1C	NS1C	28.0	0.16	6.65	1.74	0.203	0.012	6.52
Nutrified Basin Cell 1D	NS1D	28.1	0.16	6.65	2.65	0.219	0.026	7.24
Aeration Basin Influent	ABI	27.4	2.52	8.45	12.0	4.58	-	4.27
1st Cell	1st	27.5	4.76	7.10	11.4	9.93	0.324	1.55
2nd Cell	2nd	27.7	1.23	6.68	2.65	16.7	0.347	0.807
3rd Cell	3rd	27.9	0.20	6.74	2.74	13.6	0.139	0.941
Effluent Channel 1	EF1	27.8	3.40	6.65	0.038	16.5	0.013	0.632
Secondary Clarifier Effluent	SCE	28.5	-	6.80	0.065	16.5	0.018	0.441
Filtered Effluent	FE	27.2	-	7.20	0.030	11.3	0.012	0.820
RAS	RAS	27.9	0.11	6.65	0.802	12.0	-	0.553
Filter Backwash	FB	28.2	-	7.45	0.043	16.5	-	0.562

Temperature readings were taken at each sampling location once in the morning and once in the afternoon for the two sampling days. The average of the two mornings and average of the two afternoon values for each sampling location are presented in Figure 3-2. Similarly, average DO and pH profiles are presented in Figure 3-3 and Figure 3-4, respectively. Average nutrient profiles are presented in Figures 3-5 through 3-8.

Figure 3-2
Average Temperature Profile

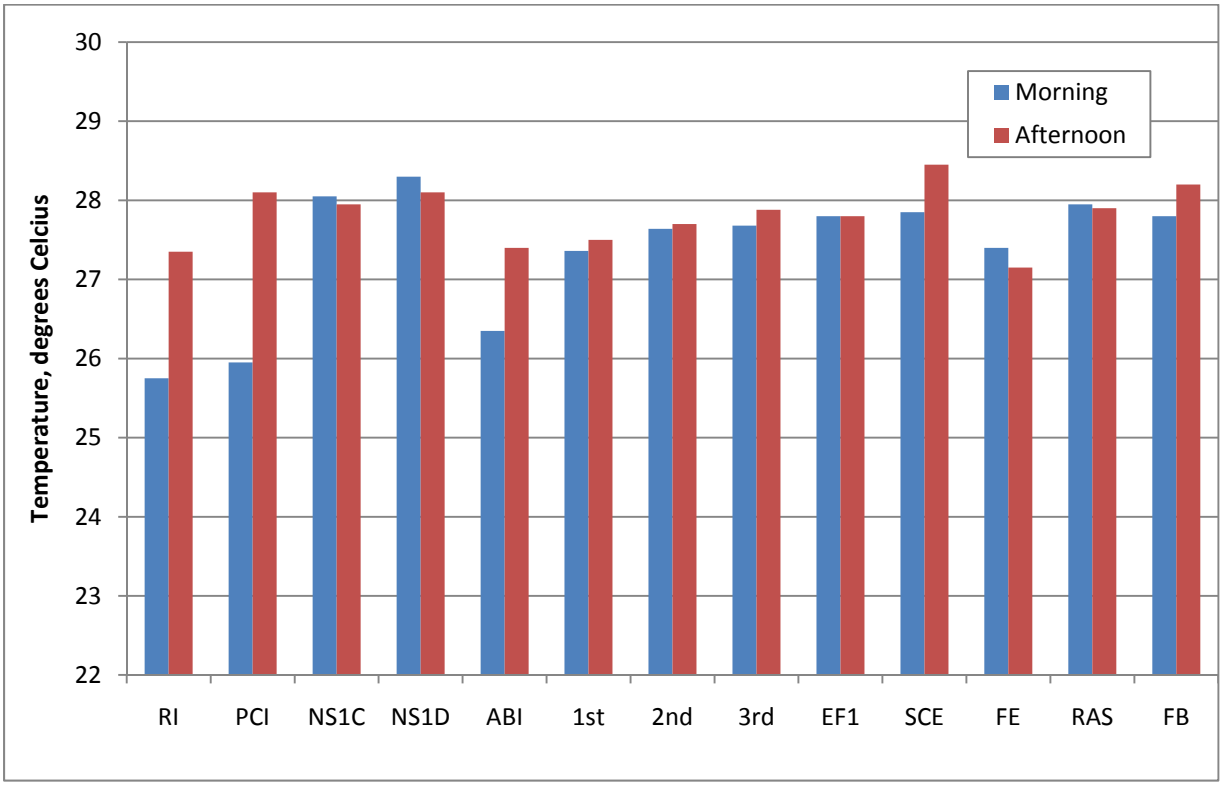


Figure 3-3
Average DO Profile

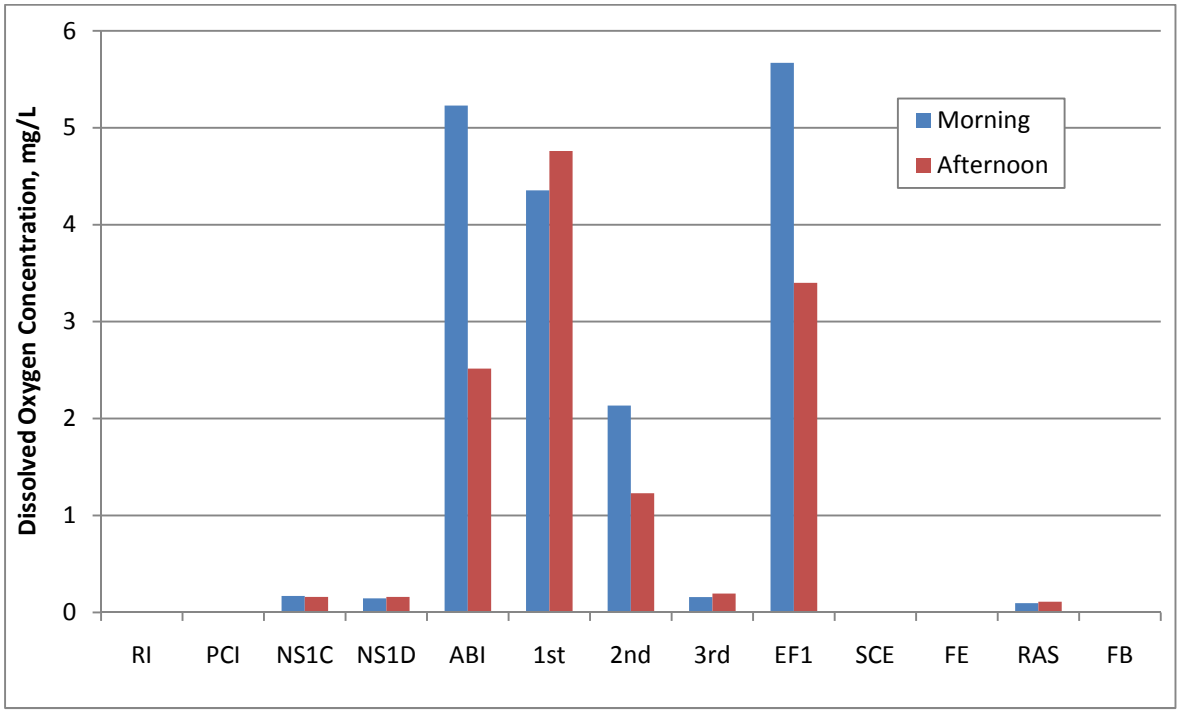


Figure 3-4
Average pH Profile

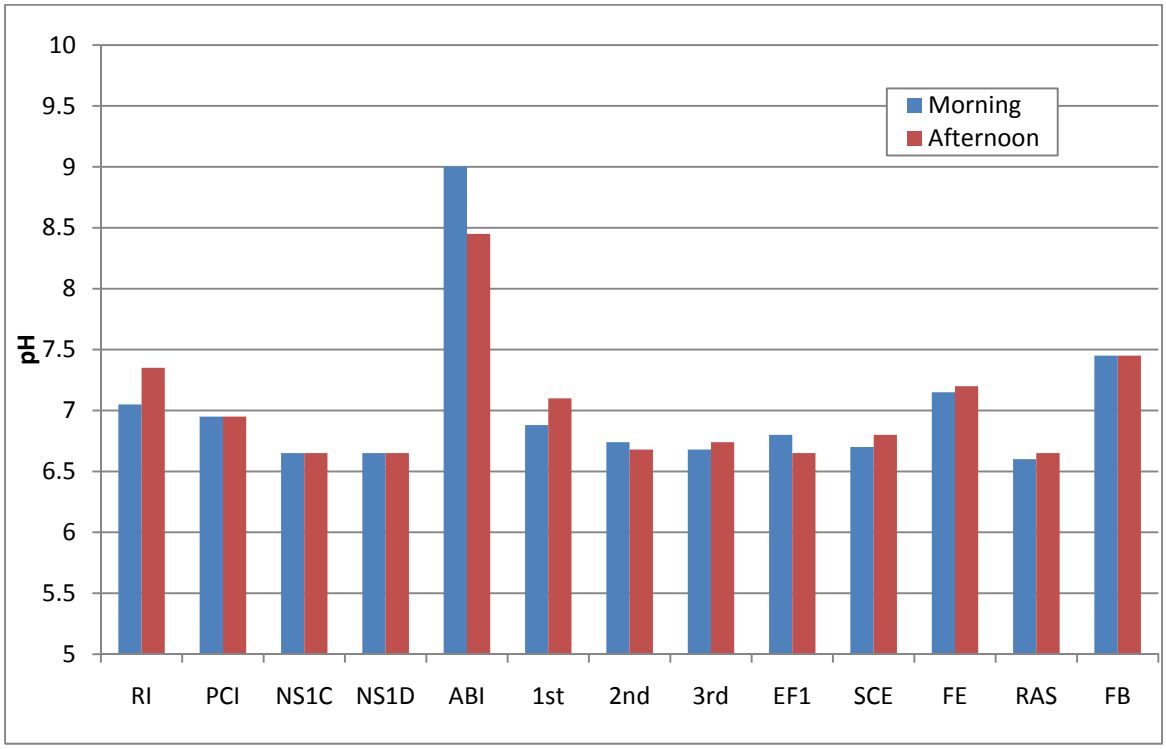
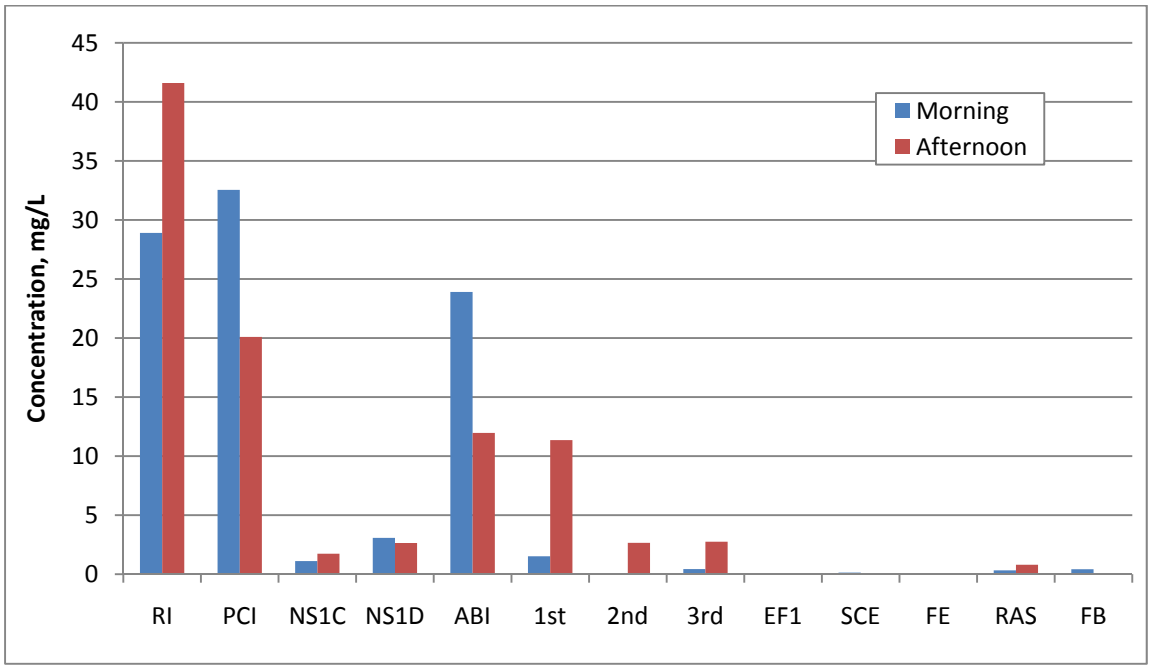


Figure 3-5
Average Ammonia Profile



It can be observed in Figure 3-5 that ammonia concentrations drop between the primary clarifier influent and the aeration basin influent. This is likely due to the introduction of nutrified sludge flow which regularly has ammonia concentrations below 5 mg/L. Figure 3-6 presents the nitrate profile. It can be observed in Figure 3-6 that nitrate concentrations drop in the third cell of the aeration basin trains. Cell 3 is anoxic and the decrease in nitrate in these cells indicates that denitrification is working in the basins.

Figure 3-6
Average Nitrate Profile

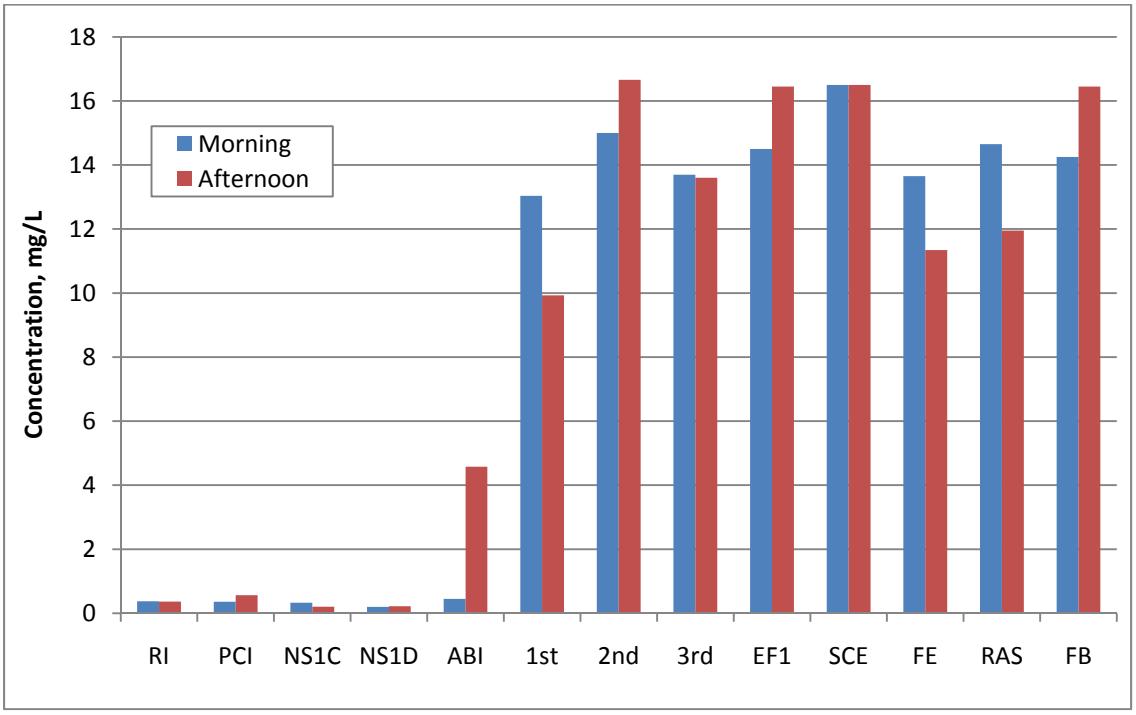


Figure 3-7
Average Nitrite Profile

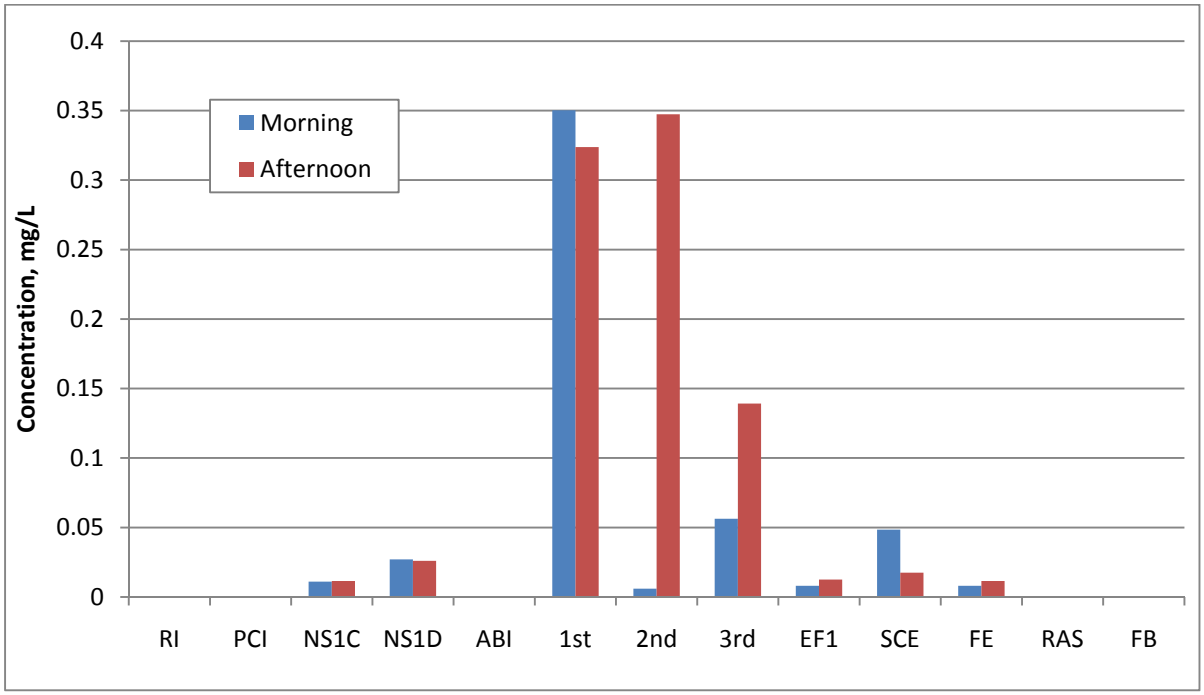
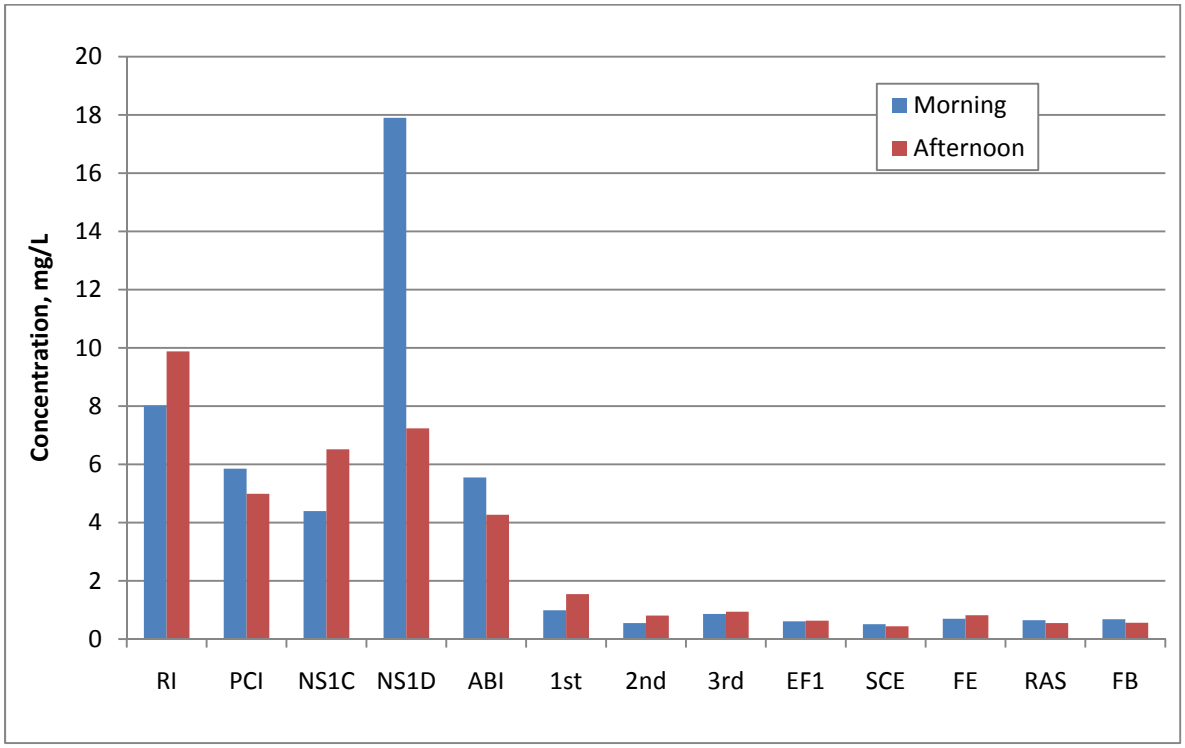


Figure 3-8
Average Phosphate Profile



Two trains were sampled on the first day. Visible short-circuiting of the aeration basins was witnessed by Hazen and Sawyer staff on the first day of sampling and it was decided that all three trains were to be sampled on the second day. Short-circuiting of flow through the aeration basins means that basin hydraulic retention time is reduced and overall basin performance is reduced. Further investigation is recommended in order to more precisely determine the impacts of this observation.

Table 3-2
Aeration Basin Performance Summary

	Cell	Temp.	DO	pH	NH_3-N	NO_3-N	NO_2-N	PO_4-P
8/18 AM Train A	1E	27.1	5.25	7.0	1.44	5.98	0.286	1.85
	1F	27.4	0.82	6.8	0.034	13.9	-	0.673
	2B	27.6	0.19	6.8	0.493	9.39	-	1.08
8/18 PM Train A	1E	27.4	4.59	7.1	7.73	9.83	0.324	1.99
	1F	27.5	0.85	6.7	2.07	14.2	0.143	0.832
	2B	27.6	0.23	6.8	2.09	10.3	0.068	1.31
8/18 AM Train C	3D	27.3	5.57	7.1	2.12	13.4	0.402	1.38
	4A	27.7	2.03	6.9	0.044	15.7	-	0.616
	4B	27.6	0.17	6.9	0.218	15.8	0.004	0.678
8/18 PM Train C	3D	27.3	4.65	7.3	13.7	8.39	0.306	2.19
	4A	27.6	1.61	6.8	3.23	16.0	0.414	1.08
	4B	27.8	0.16	6.8	1.92	13.0	0.145	0.991
8/20 AM Train A	1E	27.4	4.87	6.6	0.553	13.8	0.317	0.812
	1F	27.5	0.37	6.4	0.043	9.11	0.007	0.592
	2B	27.6	0.13	6.5	0.41	12.1	0.054	0.839
8/20 PM Train A	1E	27.9	5.04	6.9	9.76	10.7	0.367	1.29
	1F	27.9	-	6.6	5.04	14.9	0.143	0.697
	2B	28.0	-	6.8	4.76	11.5	0.090	0.871
8/20 AM Train B	3A	27.4	1.1	6.9	1.89	15.4	0.229	0.392
	2D	27.5	5.84	6.9	0.024	17.9	0.006	0.403
	2C	27.7	0.15	6.8	0.734	15.1	0.088	1.10
8/20 PM Train B	3A	27.6	-	7.0	10.3	12.0	0.272	0.916
	2D	27.8	-	6.7	1.35	20.7	0.547	0.617
	2C	28.0	-	6.7	1.06	17.8	0.205	0.798
8/20 AM Train C	3D	27.6	4.98	6.8	1.57	16.6	0.517	0.533
	4A	28.1	1.61	6.7	0.034	18.4	0.005	0.47
	4B	27.9	0.15	6.4	0.291	16.1	0.079	0.625
8/20 PM Train C	3D	27.3	-	7.2	15.3	8.71	0.350	1.34
	4A	27.7	-	6.6	1.57	17.5	0.490	0.81
	4B	28.0	-	6.6	3.88	15.4	0.188	0.733

Additionally, it was observed that unequal flow distribution of NSL effluent (RAS) is likely occurring. NSL effluent is distributed to the first cells of the three treatment trains. It is apparent that more NSL effluent is entering the first train, which is the closest train to the NSL basin.

Detailed DO and pH profiles are presented in Figure 3-9 and 3-10, respectively. DO readings were not taken on Thursday afternoon due to inclement weather. The plant has the ability to measure flow into each cell of the aeration basin and air flow data corresponding to the special sampling week was provided by plant staff. Airflow data was not used for process analysis or model calibration since some values appeared unreliable. For example high recorded air flows in certain cells do not correlate to higher dissolved oxygen concentrations.

Figure 3-9
DO Profile in Aeration Tanks

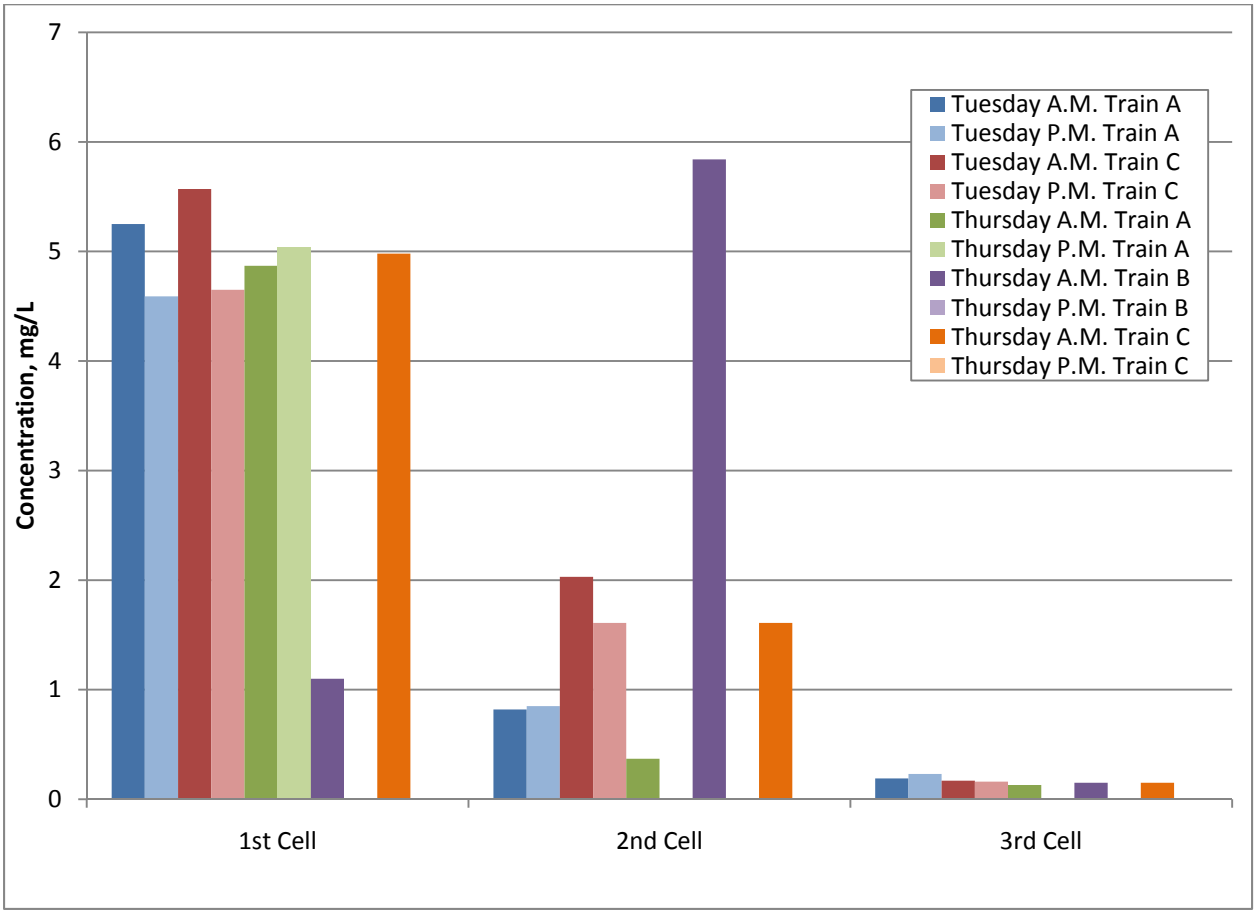
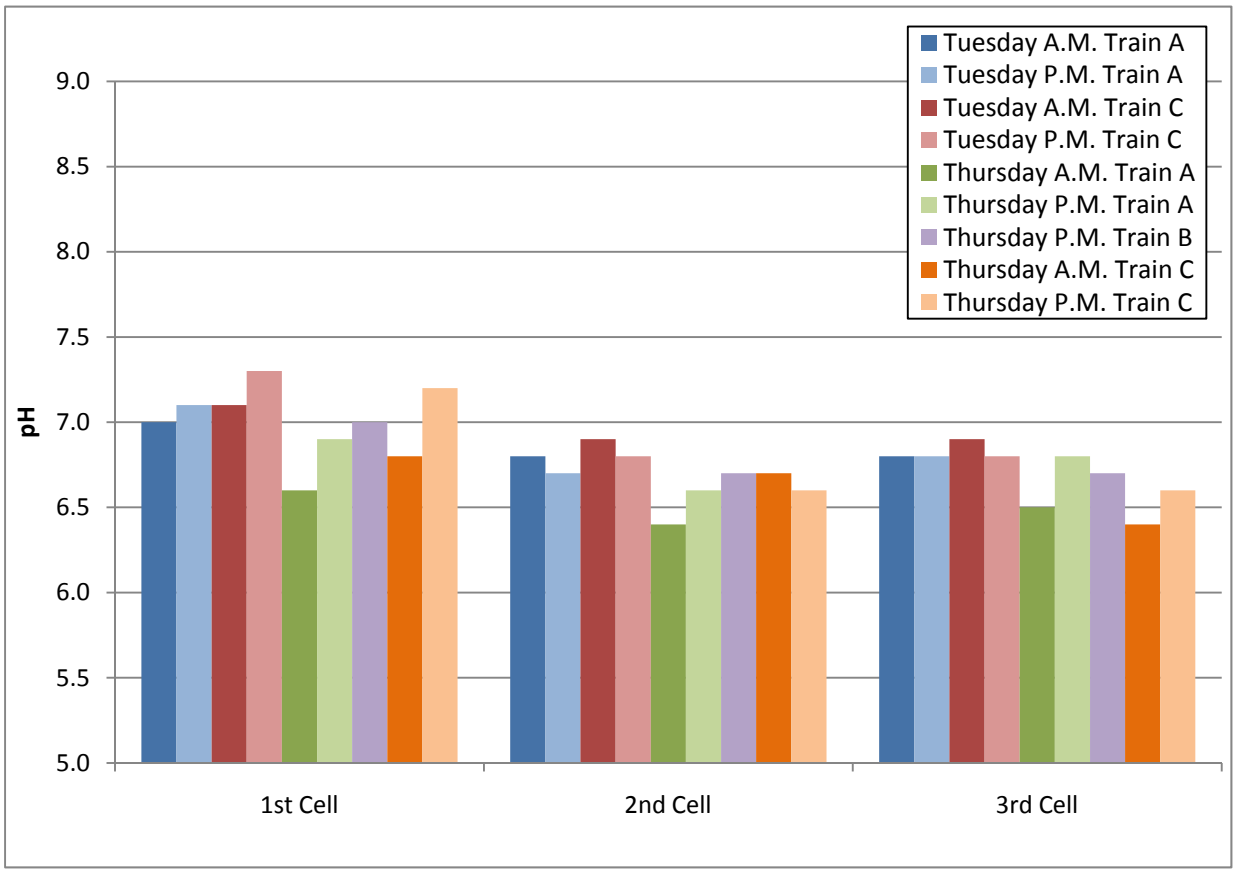


Figure 3-10
pH Profile in Aeration Tanks



Detailed nutrient profiles are presented in Figures 3-11 through 3-14.

Figure 3-11
Ammonia Profile in Aeration Tanks

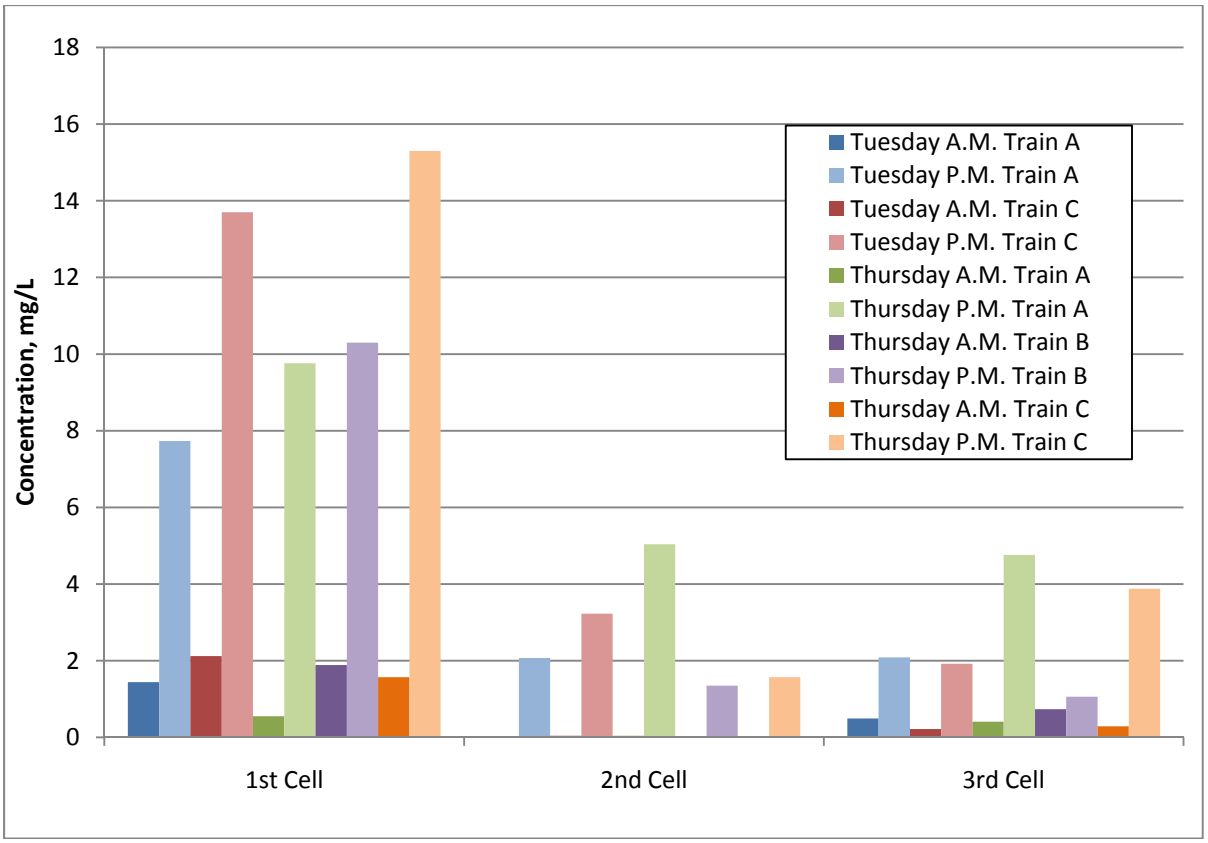


Figure 3-12
Nitrate Profile in Aeration Tanks

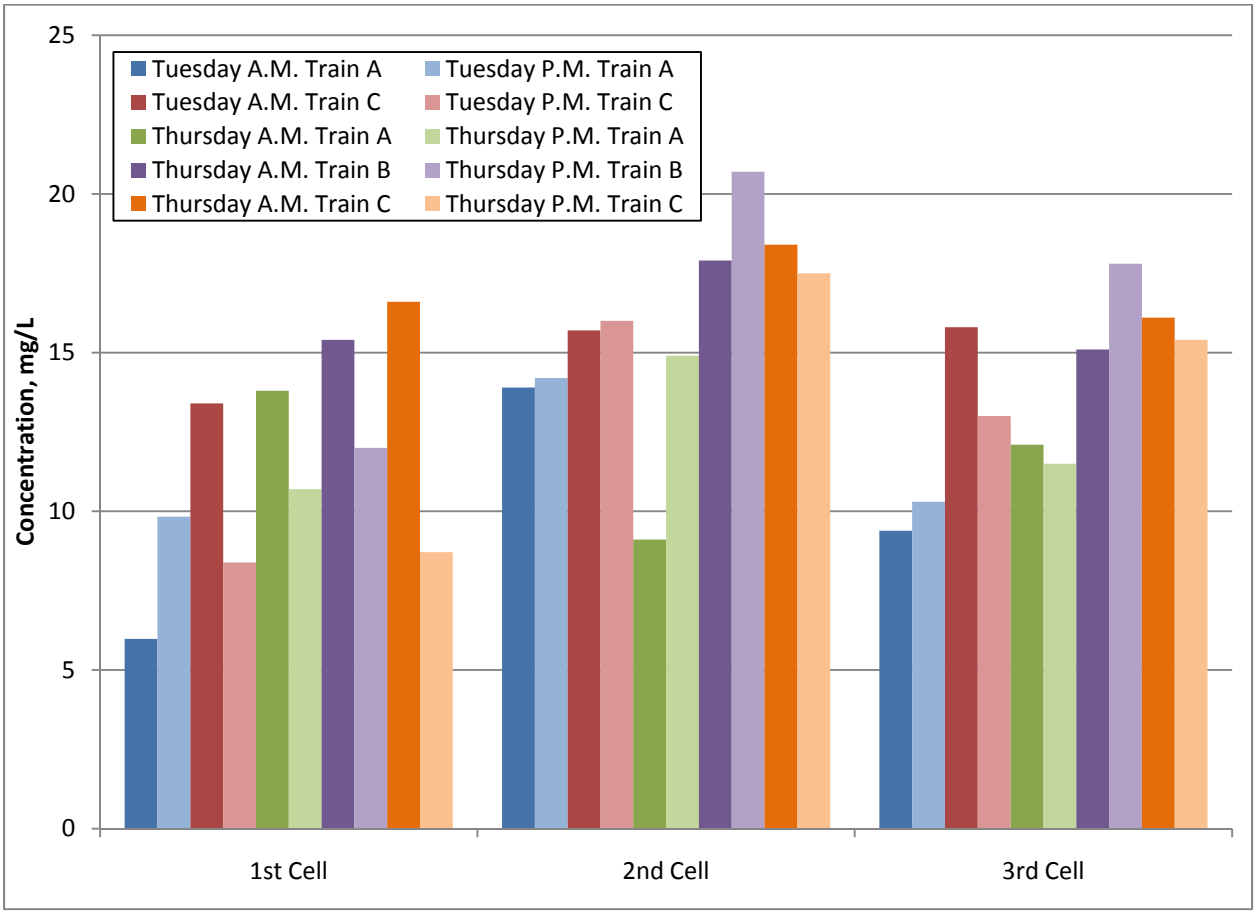


Figure 3-13
Nitrite Profile in Aeration Tanks

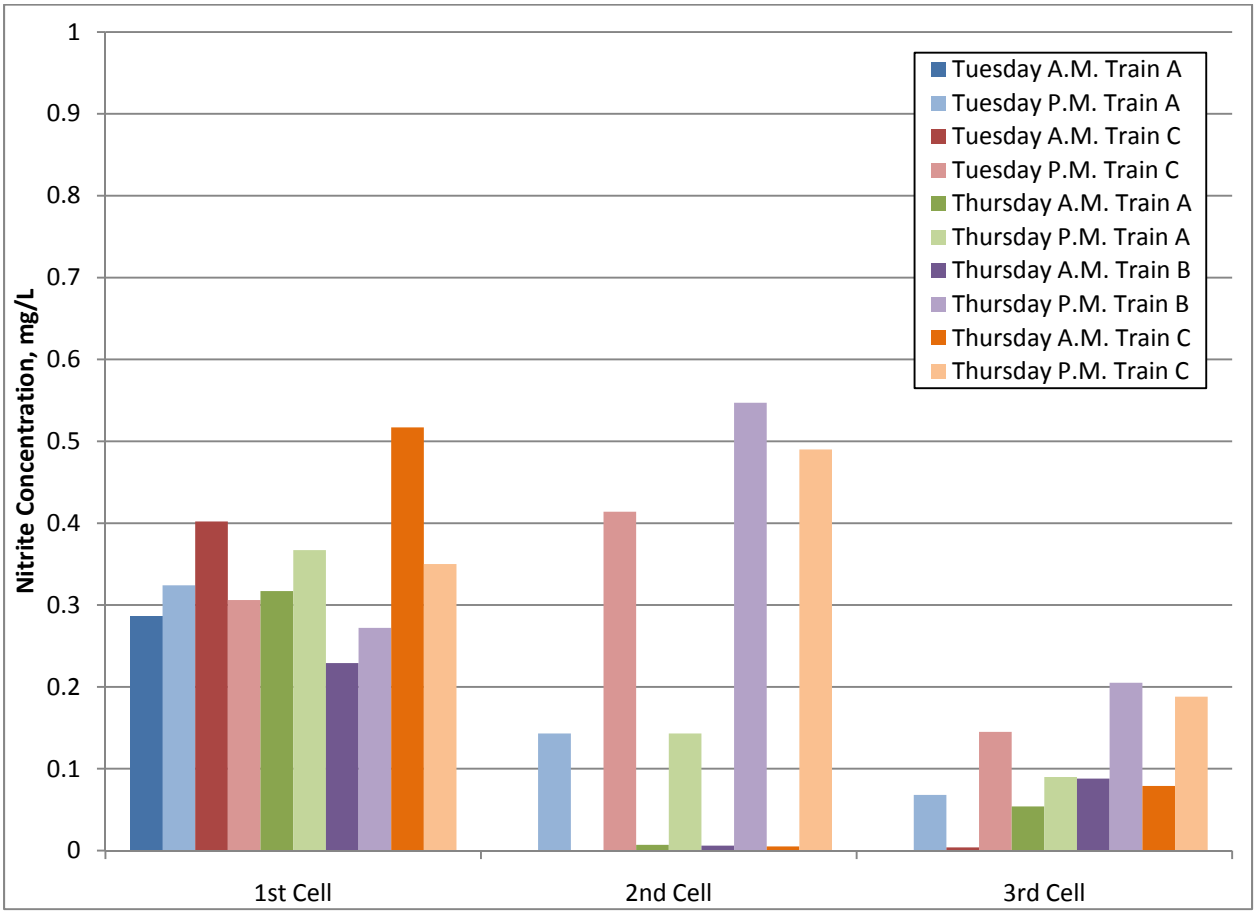
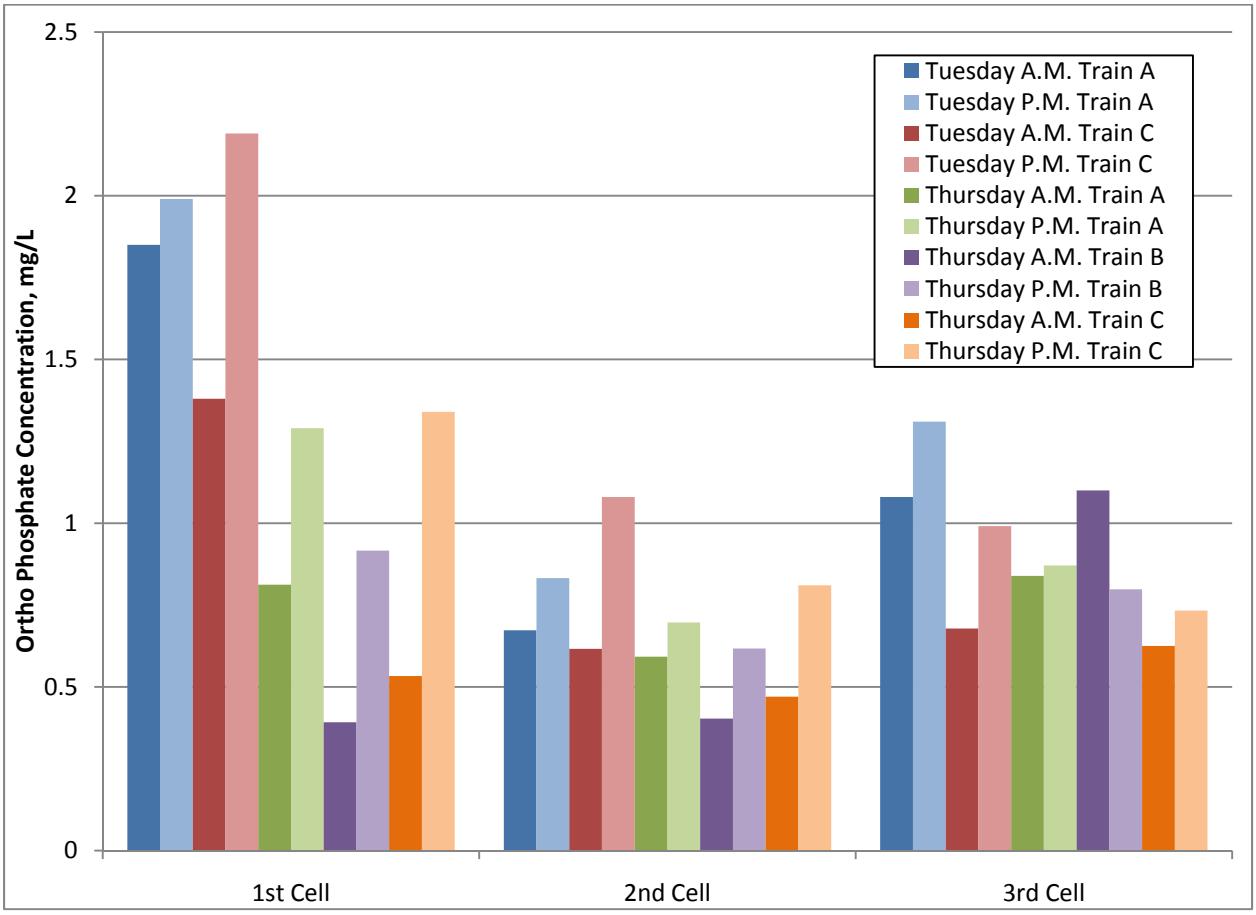


Figure 3-14
Ortho Phosphate Profile in Aeration Tanks



4. RECYCLE STREAM SAMPLING

Manual recycle stream composite samples were collected by Hazen and Sawyer August 17 through August 21. Filtrate was recirculated as washwater to the belt while the sample was collected in order to better characterize the recycle stream without the impacts of non-potable washwater. Gravity belt thickener filtrate, fermentate, and rotary press filtrate characteristics are summarized in Table 4-1.

Table 4-1
Recycle Stream Characteristics

Parameter	GBT Filtrate	Fermentate	Rotary Press Filtrate
TSS	283	6322	398
VSS	222	5200	281
BOD_{XX}[1]	535	5608	267
BOD_{XG}[1]	-	1213	-
COD_{XX}[1]	709	5580	1483
COD_{XG}[1]	-	1001	-
COD_{XM}[1]	-	897	-
COD_{XF}[1]	-	944	-
TKN_{XX}[1]	51	440	1296
TKN_{XG}[1]	-	131	-
NH_3-N_{XG}[1]	17	40	48
NO_3-N	4	0.3	3.1
NO_2-N	0.03	0.1	0.03
TP_{XX}[1]	12	132	12.7
TP_{XG}[1]	-	10	-

Primary clarifier influent and fermentate samples were analyzed for volatile fatty acids (VFAs). Quantifying the VFAs is important in understanding and assessing the OWASA Nutrification™ process; VFAs generated in the primary fermenter are sent to the nutrified sludge tanks to aid in nutrient removal. Results of the analysis are presented in Tables 4-2 and 4-3. Primary clarifier influent VFA values are not reliable, as the measured VFA concentrations are significantly greater than the VFA concentrations predicted from primary clarifier influent COD_{xf} values.

Table 4-2
VFA Analysis for Primary Clarifier Influent

Parameter	Aug 18th AM	Aug 18th PM	Aug 20th AM	Aug 20th PM
Acetic Acid (mg/L)	110.0	76.0	37.0	30.0
Butyric Acid (mg/L)	<5.0	<5.0	<5.0	<5.0
Lactic Acid (mg/L)	<120.0	<120.0	<120.0	<120.0
Propionic Acid (mg/L)	<5.0	<5.0	<5.0	<5.0
Pyruvic Acid (mg/L)	<50.0	<50.0	<50.0	<50.0
Maximum Total	290.0	256.0	217.0	210.0

Table 4-3
VFA Analysis for Fermentate

Parameter	Aug 18th	Aug 20th
Acetic Acid (mg/L)	440.0	490.0
Butyric Acid (mg/L)	120.0	140.0
Lactic Acid (mg/L)	<120.0	<120.0
Propionic Acid (mg/L)	260.0	270.0
Pyruvic Acid (mg/L)	<50.0	<50.0
Maximum Total	990	1070

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Sarah Lothman, Joe Rohrbacher, Paul Pitt

DATE: October 26, 2009
Revised February 15, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum 3-B – Process Model Calibration

1. INTRODUCTION AND OVERVIEW

The process modeling software BioWin 3.0.1 (Envirosim Ltd.) was used to develop a calibrated biological process model of the Mason Farm WWTP (MFWWTP). The calibrated model will be applied to evaluate the existing capacity of the biological treatment process at the MFWWTP and develop and assess treatment alternatives to comply with current and future treatment limits, particularly in regard to the forthcoming nutrient limitations imposed by the Jordan Lake nutrient management strategy. This memorandum describes the calibration of the process model developed for the MFWWTP.

The process model was calibrated based on historical data and the results of supplemental sampling performed in August 2009. Long-term daily dynamic simulations of the Mason Farm WWTP were performed for the period of November 1, 2008 through August 31, 2009. This period was selected since the flow pattern through the secondary process was consistent during this time and is representative of current operations. The primary purpose of the long term calibration was to accurately characterize solids production at the plant. Highlights of the long term calibration include:

- Predicted raw influent and aeration basin influent concentrations correlate very well to the observed values until May 2009. From May 2009 onwards, the model overpredicts TSS, VSS ammonia and BOD concentrations. The predicted concentrations appear to approach the reported values towards the end of August 2009. The change in calibration quality may be due to seasonal variations in influent wastewater characteristics.

- Predicted aeration basin MLVSS concentrations compared favorably with the reported

values through most of the simulated period. The model predicted a higher concentration of inert solids, increasing the simulated MLSS concentration. This could be attributed to the aluminum solids contributed through alum addition for phosphorus removal as the alum feed to individual points is not measured.

- The predicted RAS concentrations are significantly greater than the reported values. Since the MLVSS and MLSS matched relatively well, the difference in the RAS concentrations indicates that there is error in the reported RAS flow rates and/or RAS concentrations. The current RAS sampling location may be providing artificially low concentrations.

- Predicted effluent ammonia, nitrate/nitrite and total nitrogen concentrations correlated well to the reported data and support the accuracy of the reported RAS rates and modeled recycle stream nitrogen impacts.

A short term dynamic simulation of the Mason Farm WWTP for the week of supplemental sampling was performed to compare measured and simulated nutrient profiles in the bioreactor basins influent, and plant effluent. Plant operating data supplemented with data derived from the special sampling were utilized in the short term simulation model runs. Highlights of the short term calibration include:

- The simulated aeration basin MLSS and MLVSS concentrations correlated well to observed values.

- The model simulated Train A ammonia concentrations very well during the first day of sampling. There was less consistency with the observed values during the second sampling day. The model predicted significantly greater ammonia concentrations in the NSL basins.

- The model simulated Train A nitrate concentrations well during each sampling event. Effluent nitrate concentrations matched very well, indicating that the impact of nitrogen from the solids recycle stream is being appropriately modeled as the nitrogen balance through the plant is consistent between modeled and observed values.

- Ammonia through Train B was simulated fairly well in Cells 2D and 2C. Nitrate concentrations through the basins matched very well.

- Predicted nitrogen concentrations in Train C exhibited more variability in matching observed values compared to Trains A and B.

- The model predicted a much greater degree of biological phosphorus release in the nutrified sludge cells than observed in the field. Aeration basin influent ortho-phosphate concentrations were similar to the observed values without modeling phosphorus precipitation. The model predicted full phosphate removal in the aeration basins.

- The simulated secondary effluent nutrient concentrations match reasonably well with the exception of total phosphorus, were predicted values were lower than observed even without chemical phosphorus removal modeling.

2. INFLUENT FRACTIONS

Special sampling was conducted at the Mason Farm WWTP from Sunday, August 16 through Saturday, August 22, 2009. Sampling included detailed influent wastewater characterization to determine carbon, nitrogen, and phosphorus speciation and two days of nutrient profiles through the activated sludge process. The results of the special sampling, summarized in a separate memorandum, along with the historical data were used to develop influent wastewater fractions. Wastewater fractions used in the model calibration are summarized in Table 2-1.

Table 2-1
Raw Influent Wastewater Fractions

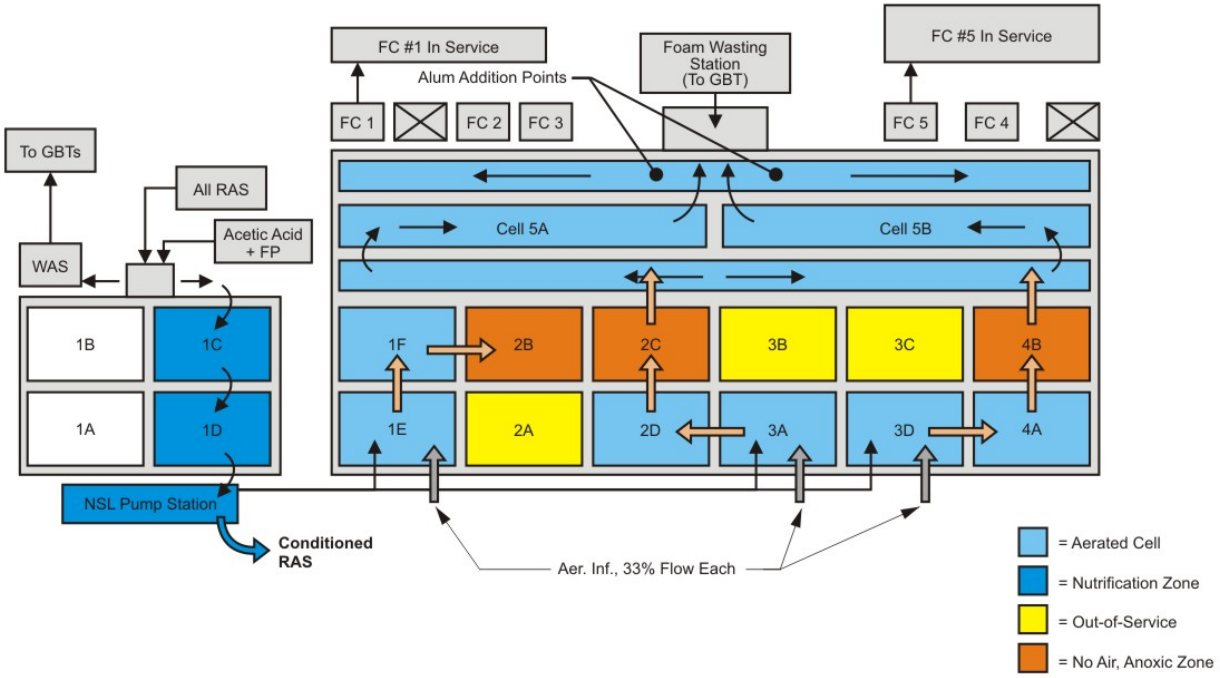
Fraction	BioWin Default	MFWWTP
Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD]	0.16	0.10
Fac - Acetate [gCOD/g of readily biodegradable COD]	0.15	0.15
Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0.75	0.90
Fus - Unbiodegradable soluble [gCOD/g of total COD]	0.05	0.03
Fup - Unbiodegradable particulate [gCOD/g of total COD]	0.13	0.24
Fna - Ammonia [gNH3-N/gTKN]	0.66	0.74
Fnox - Particulate organic nitrogen [gN/g Organic N]	0.5	0.6
Fnus - Soluble unbiodegradable TKN [gN/gTKN]	0.02	0.01
FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD]	0.035	0.018
Fpo4 - Phosphate [gPO4-P/gTP]	0.5	0.7
FupP - P:COD ratio for influent unbiodegradable part. COD [gP/gCOD]	0.011	0.004

3. LONG-TERM SOLIDS CALIBRATION

3.1 Long-Term Calibration Set-up and Assumptions

Long-term daily dynamic simulations of the MFWWTP were performed for the period of November 1, 2008 through August 31, 2009. This period was selected since the flow pattern through the secondary process was consistent during this time and is representative of current operations. Figure 3-1 presents a schematic of the biological process operations during the special sampling period, which is representative of the number of basins online and flow path through the long-term calibration period.

Figure 3-1
Biological Process Schematic



The available plant operating data was supplemented with data derived from the supplemental sampling, and wastewater fractions summarized in Table 1 were utilized in the model runs. The development of the process model and assumptions made are provided as follows:

- Daily plant influent flows were input directly into the model. Influent concentrations were developed from the daily reported influent data included in the discharge monitoring reports (DMRs) and the results of the special sampling.

- Daily mixed liquor suspended solids (MLSS) temperature data were directly input into

the model.

- Daily reported primary sludge flow rates and calculated % solids removal were directly input into the model.

- A dissolved oxygen (DO) concentration of 2.0 mg/L was assumed for all aerated process tankage and channels.

- An ideal secondary clarifier element was used, and secondary clarifier solids removal performance was set at 99.85%. Reported daily return activated sludge (RAS) flow rates were directly input into the model.

- The tertiary filters were modeled as a point clarifier assuming 90% removal. The reported daily filter backwash rates were directly input into the model.

- Reported daily waste activated sludge (WAS) flow rates were directly input into the model.

- Daily 25% sodium hydroxide flow rates were directly input upstream of the aeration basin influent channel.

- Daily 20% acetic acid flow rates were directly input upstream of the nutrified sludge basins.

- Alum addition was simulated as inert suspended solid (ISS) material since modeling complex chemical precipitation reactions severely slows the simulation process. A state variable input was developed based on a net solids production of 1.8 lbs of inert solids per gallon per day (gpd) of 48% alum solution added. The mass of solids contributed through alum assumed half of the alum reacted with phosphate to form $AlPO_4$. 65% of the total daily alum dose was added to the node where filtrate is returned upstream of the primary clarifiers. The remaining 35% was added upstream of the secondary clarifiers.

- A gravity belt thickener (GBT) solids capture efficiency of 85% was assumed. The GBT thickened solids flow rate was set at 12% of the total flow to the GBT.

- The anaerobic digesters were modeled as a single digester operated at a temperature of 40°C, which is the maximum temperature the BioWin model will allow operation at.

- The flow to the rotary press was assumed to be 60% of the total digested solids flow,

which is based on historical data. The solids capture through the rotary press was assumed to be 99%, and the rotary press dewatered cake flow was assumed to be 11.4% of the input flow to the press.

- Two nutrified sludge cells were assumed online during the simulation to match typical operations.

- Three aeration trains of three cells each (first two aerobic and last anoxic) and Cell 5a and 5b were assumed online during the simulation to match typical operations.

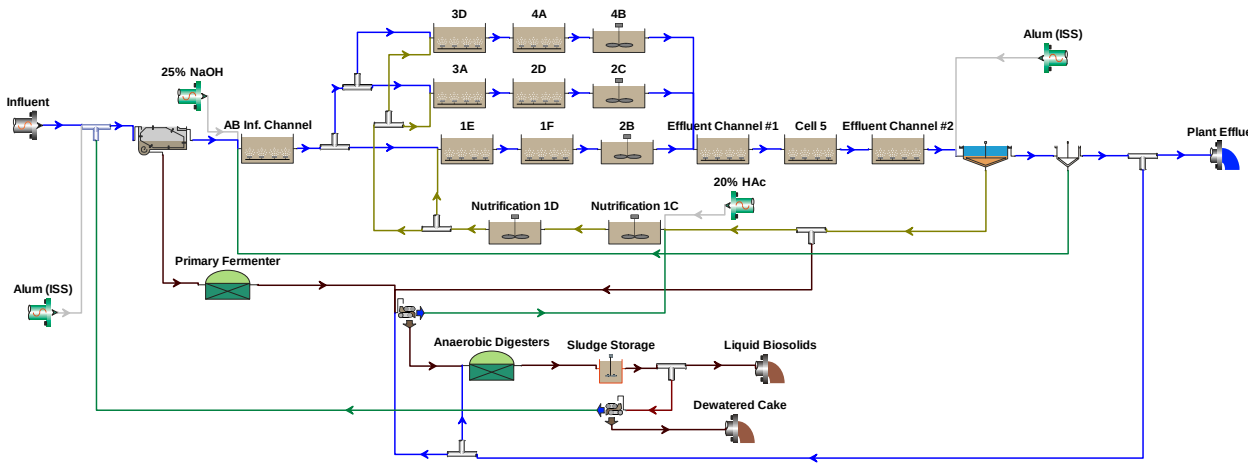
- The numbers of clarifiers online through the modeling period varied. Since ideal clarifier models were used in the model the area and volume of clarifiers online does not affect the simulation results.

- The default BioWin 3.0.1 kinetic and stoichiometric parameters were assumed.

- pH and DO modeling were turned off during the simulations.

The model setup for the long-term solids calibration simulations is illustrated in Figure 3-2.

Figure 3-2
Calibration Model Setup



3.2 Long-Term Calibration – Influent BOD and TSS Comparison

Since historical COD data is unavailable the reported influent $cBOD_5$ concentrations were multiplied by 2.56 to develop influent COD values to be input into the model. The 2.56 factor is

based on the COD:BOD_5 ratio associated with the influent fractionation (2.41) divided by the $cBOD_5$:BOD_5 ratio (0.94) observed during special sampling. Figure 3-3 provides the influent BOD pattern and historically observed data.

Figure 3-3
Predicted vs. Measured Influent BOD Values

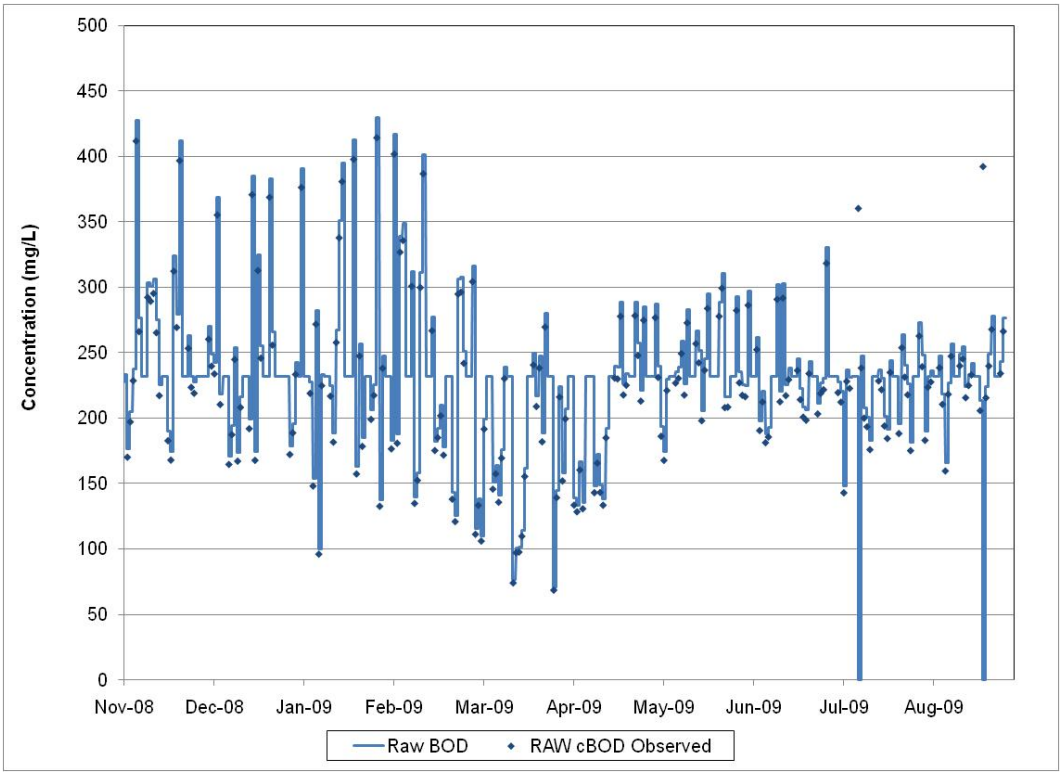
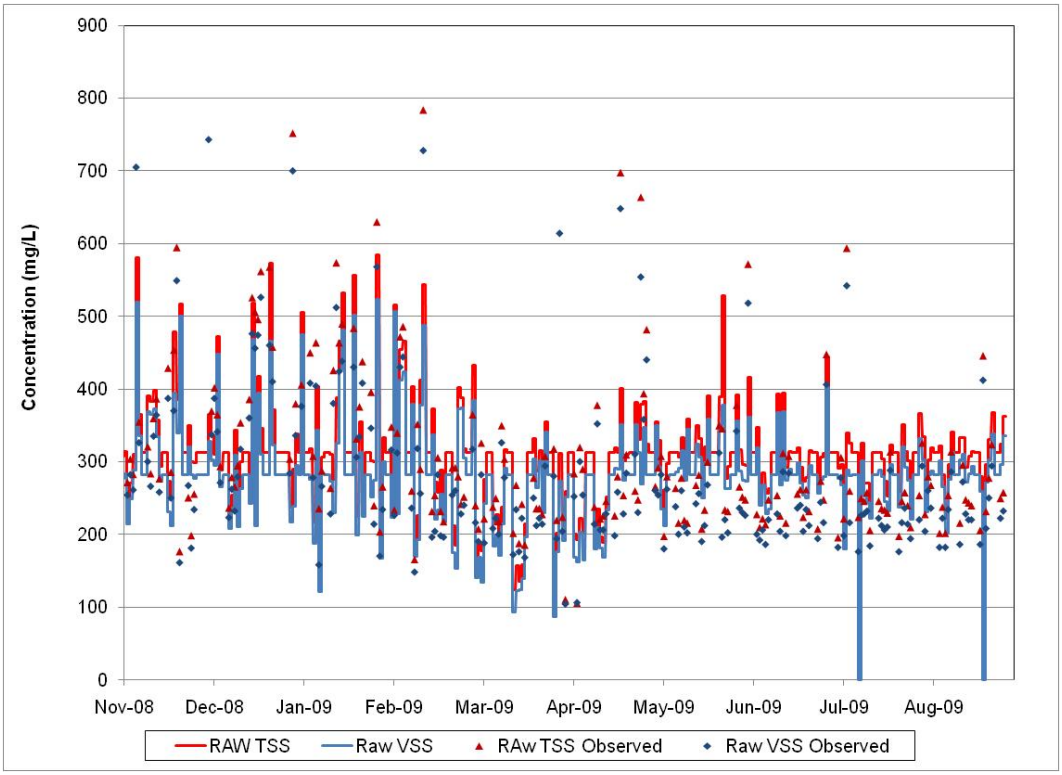


Figure 3-4 compares simulated and reported influent TSS and VSS concentrations. The model calculates the VSS based on the influent COD and wastewater fractionation. The TSS is calculated by adding the VSS to the inert suspended solids (ISS) concentrations, which are directly input to the model. The predicted influent TSS and VSS concentrations correlate very well to the observed values until May 2009. From May 2009 onwards, the model overpredicts influent VSS and TSS concentrations. The predicted concentrations appear to approach the reported values towards the end of August 2009.

The MFWWTP receives significant flow (~ 25%) from the University of North Carolina, and many of OWASA's customers are affiliated with the University. Differences in the service area population during the academic year compared to the summer months, when many students and others affiliated with the University may not be contributing flow, could possibly account for seasonal variability in influent wastewater characteristics.

Figure 3-4
Predicted vs. Measured Influent TSS Values



3.3 Long-Term Calibration – Aeration Basin Influent BOD and TSS Comparison

Figure 3-5 and Figure 3-6 present modeled and observed BOD_5 and solids concentrations at the influent to the aeration basin. Predicted values are greater than measured values from May 2009 until the middle of August, where they appear to correlate better. Since the model correlates well prior to May 2009 (and may recover at the end of August), changes in wastewater properties, solids handling operations or primary clarifier operation may account for the difference between how well the model simulates observed performance.

Figure 3-7 presents aeration influent ammonia concentrations. The same pattern of better correspondence to observed values prior to May 2009 is observed. Since ammonia is a soluble species primary clarifier removal rates should not impact the concentration. Differences in wastewater characteristics or solids handling likely explain the variation in predicted and observed values.

Alum is added to the rotary press filtrate, which is returned upstream of the primary clarifiers. The removal of phosphate was not modeled (only the inert solids production); therefore predicted aeration basin influent concentrations are expected to be greater than observed values, which is confirmed in Figure 3-8.

Figure 3-5
Predicted vs. Measured Aeration Basin Influent BOD Values

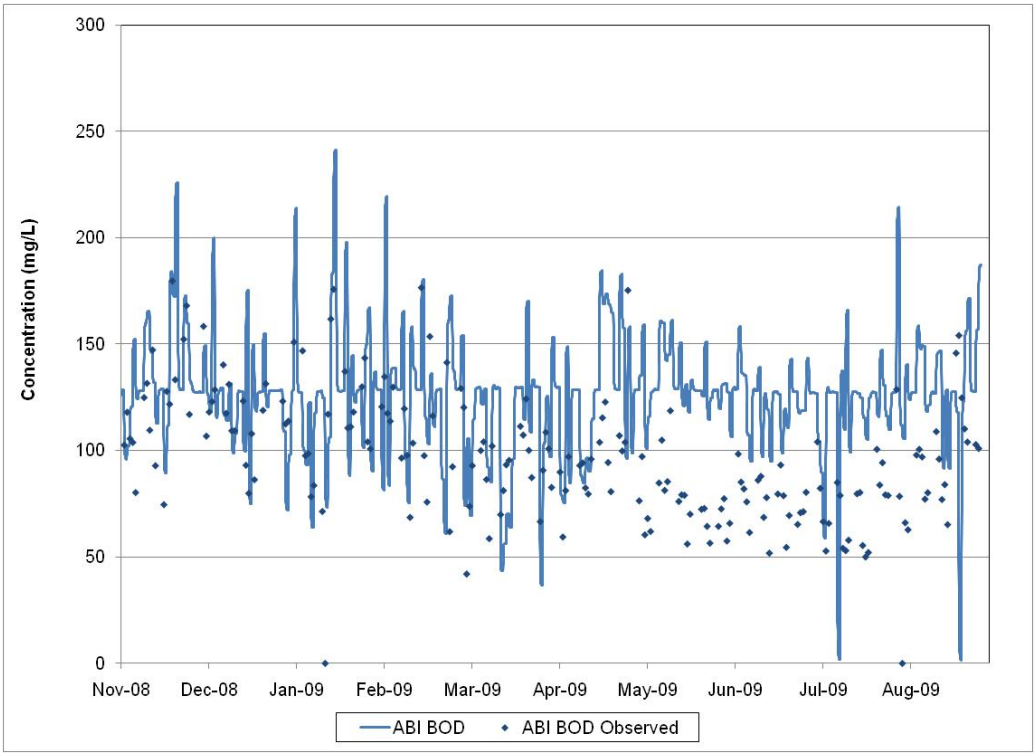


Figure 3-6
Predicted vs. Measured Aeration Basin Influent TSS and VSS Values

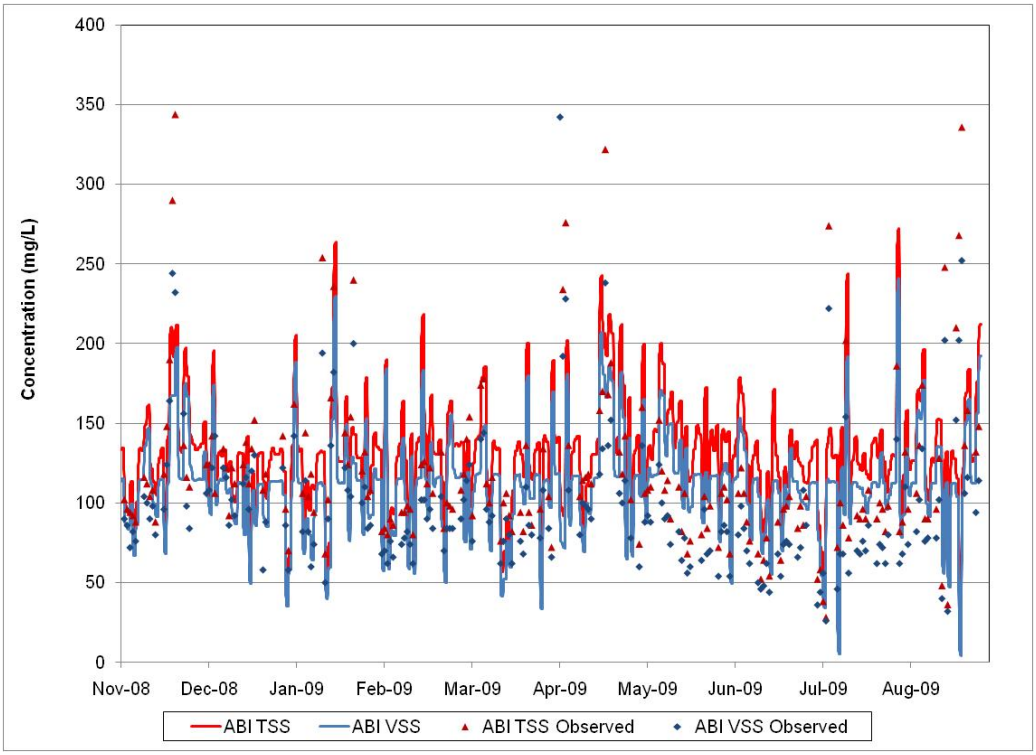


Figure 3-7
Predicted vs. Measured Aeration Basin Influent Ammonia Values

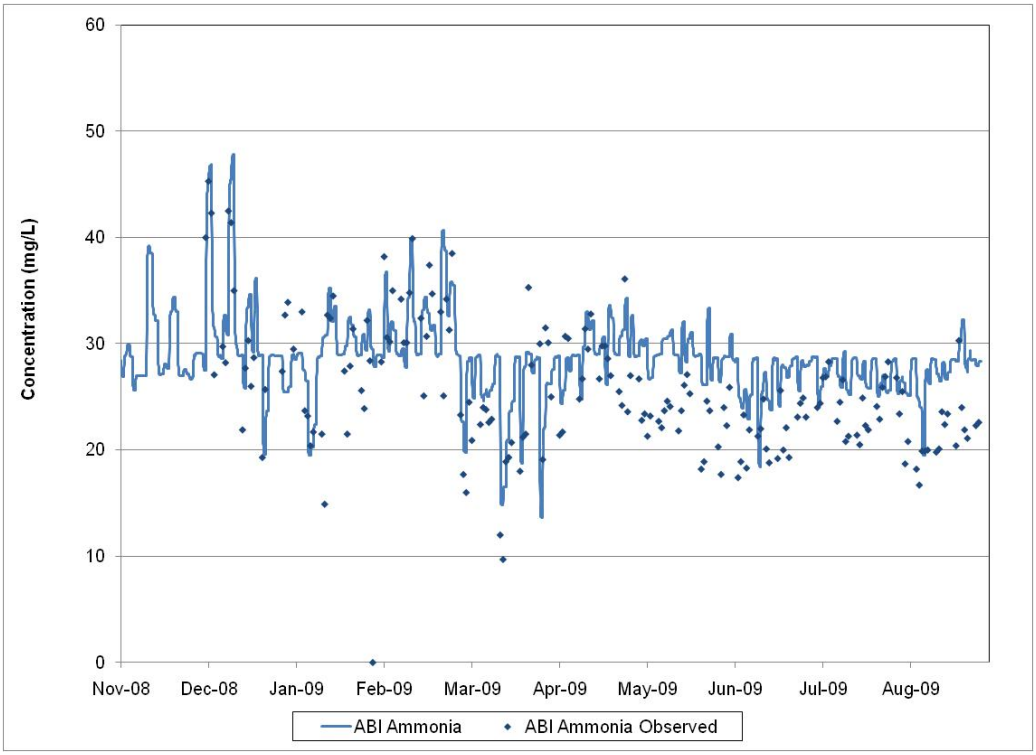
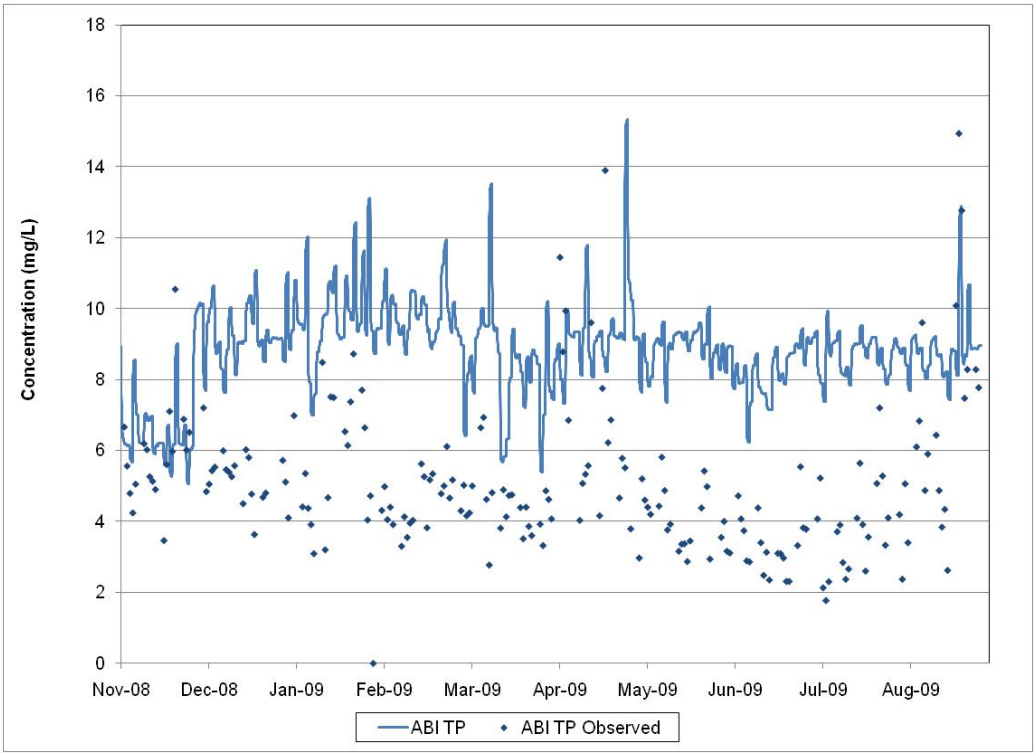


Figure 3-8
Predicted vs. Measured Aeration Basin Influent TP Values



3.4 Long-Term Calibration – Mixed Liquor Suspended Solids Comparison

The predicted aeration basin MLSS and mixed liquor volatile suspended solids (MLVSS) concentrations are compared to the reported values in Figure 3-9. The MLVSS concentrations compared favorably with the reported values through most of the simulated period. The model predicted a higher concentration of inert solids, increasing the simulated MLSS concentration. This could be attributed to the aluminum solids contributed through alum addition for phosphorus removal as the alum feed to individual points is not measured. The mass of solids contributed through alum assumed half of the alum reacted with phosphate to form $AlPO_4$.

Figure 3-9
Predicted vs. Measured MLSS and MLVSS Values

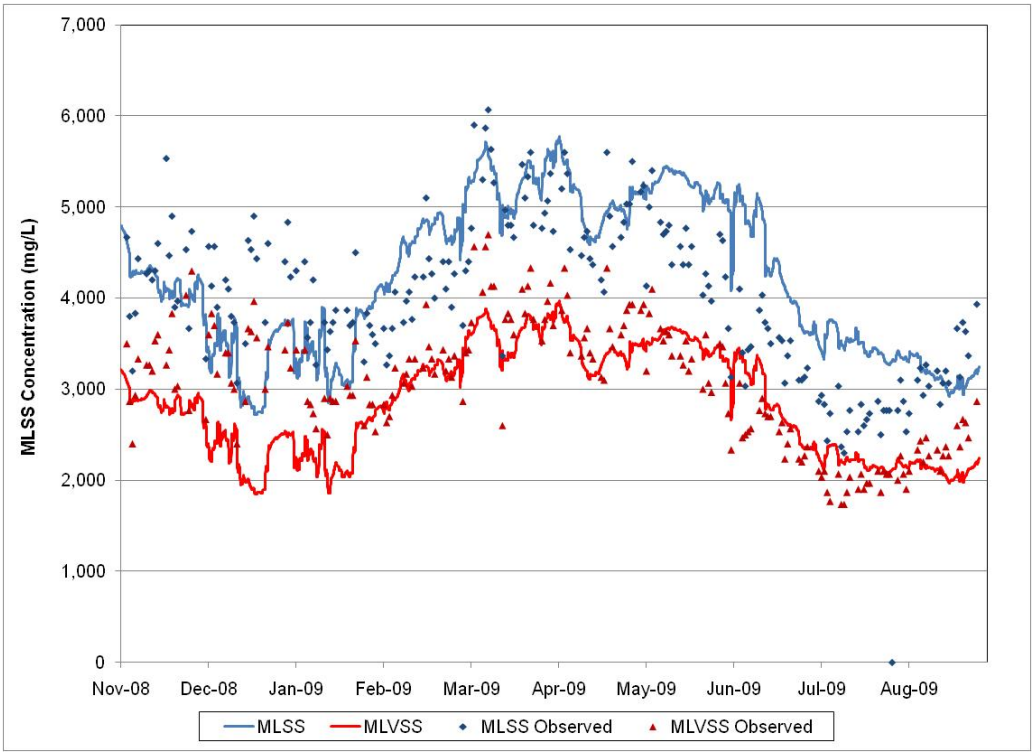
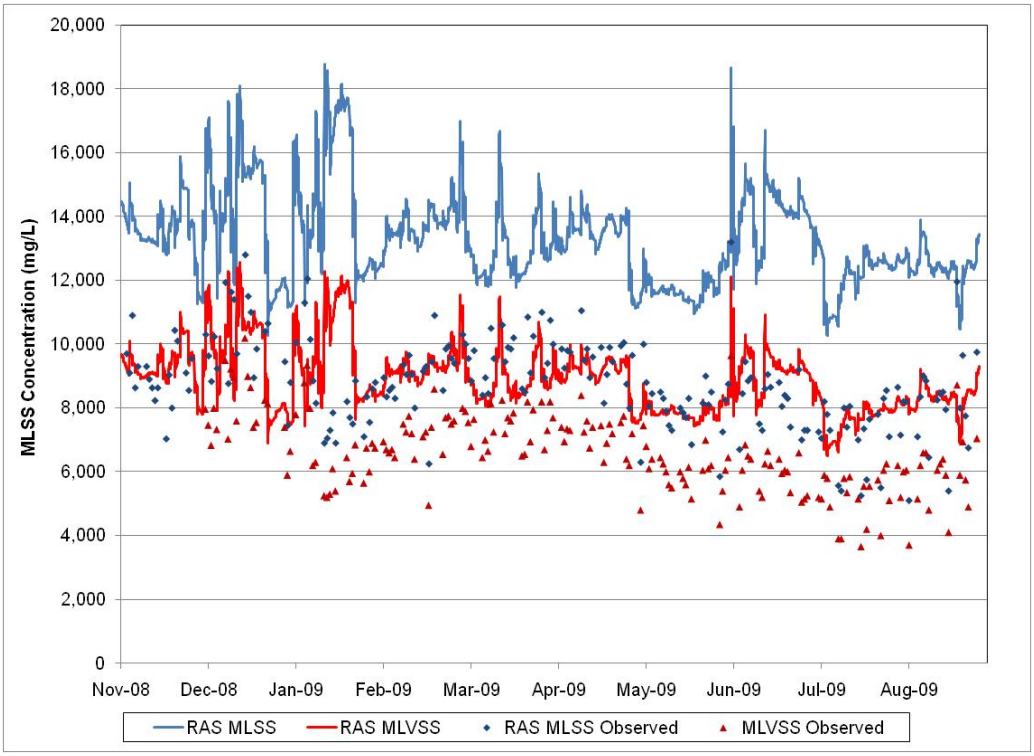


Figure 3-10 compares the measured and modeled results of the RAS TSS and VSS values. The predicted RAS concentrations are significantly greater than the reported values. Since the MLVSS and MLSS matched relatively well, the difference in the RAS concentrations indicates that there is error in the RAS flow rates input into the model (taken directly from the operational data) and/or reported RAS concentrations. Effluent suspended solids were reported as less than 1 mg/L through the period and are not expected to significantly impact the solids balance around the secondary clarifiers.

Figure 3-10
Predicted vs. Measured RAS TSS and VSS Values



RAS samples are collected from the nutrified sludge basin influent channel and likely include the impact of acetic acid and gravity thickener filtrate addition. Since the relative flows and solids concentrations of these two streams are expected to be minor compared to the RAS flow and solids concentrations, these impacts are likely insignificant in determining the RAS concentration. The results of the special sampling indicate that the gravity thickeners operate well in regards to solids removal as filtrate concentrations were relatively low (less than 300 mg/L) even accounting for the impact of washwater flows.

The RAS influent channel exhibits low velocities at current flows which may lead to solids settling at the channel bottom. It is possible that a significant portion of the RAS solids settle in the channel, and the RAS sample collected for process control is somewhat dilute. Significantly greater solids concentrations were reported at the end of the nutrified sludge basins in comparison to the RAS in the influent channel during the special sampling period. This confirms that the current RAS sampling location may be providing artificially low results as the nutrified sludge basins appear well mixed. Since nutrified sludge basin MLSS data is unavailable, it is hypothesized that the reported RAS concentrations are artificially low due to sample location. The actual RAS concentrations are likely closer to the increased values predicted by the modeling. A comparison of RAS solids collected from the influent channel to those collected from the nutrified sludge basins or other well mixed location (such as the RAS pump discharge) should be performed. The location of the RAS sample collection should be modified as

necessary.

3.5 Long-Term Calibration – Plant Effluent Comparison

Although the purpose of the long-term dynamic model was to calibrate the model to observed solids production, predicted effluent quality was also compared to values reported in the DMRs. Figures 3-11 through 3-14 compare the effluent nitrogen species.

The model predicted full nitrification year round with the exception of two periods in December 2008, where effluent ammonia concentrations spiked. It should be noted that the highest reported effluent TKN concentration was observed during one of these spikes. The model also did not predict the increase in observed effluent ammonia data between February and March 2009. An increase in effluent TKN concentration was not observed during this period indicating that there may have been analytical issues in ammonia measurement during this period. Predicted effluent TKN data was slightly higher than reported values, indicating that lowering the the influent soluble unbiodegradable TKN fraction from 0.002 to 0.001 is appropriate for the MFWWTP. Predicted nitrate/nitrite and total nitrogen concentrations correlated well to the reported data and support the accuracy of the reported RAS rates and modeled recycle stream nitrogen impacts.

Figure 3-11
Predicted vs. Measured Effluent NH_3 Values

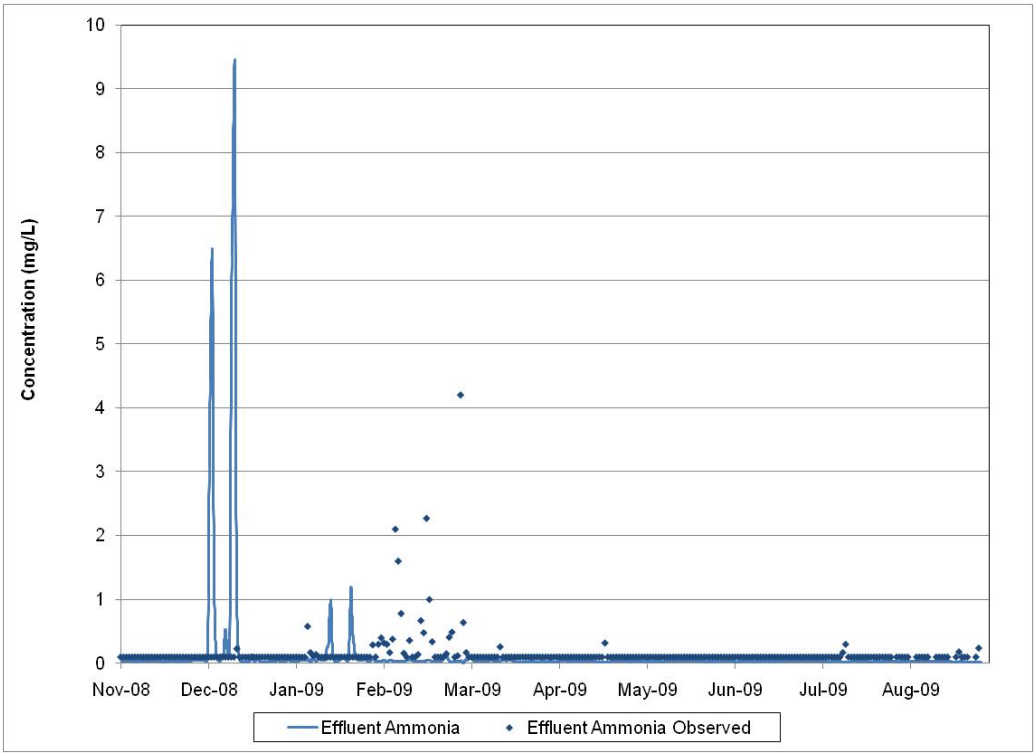


Figure 3-12
Predicted vs. Measured Effluent TKN Values

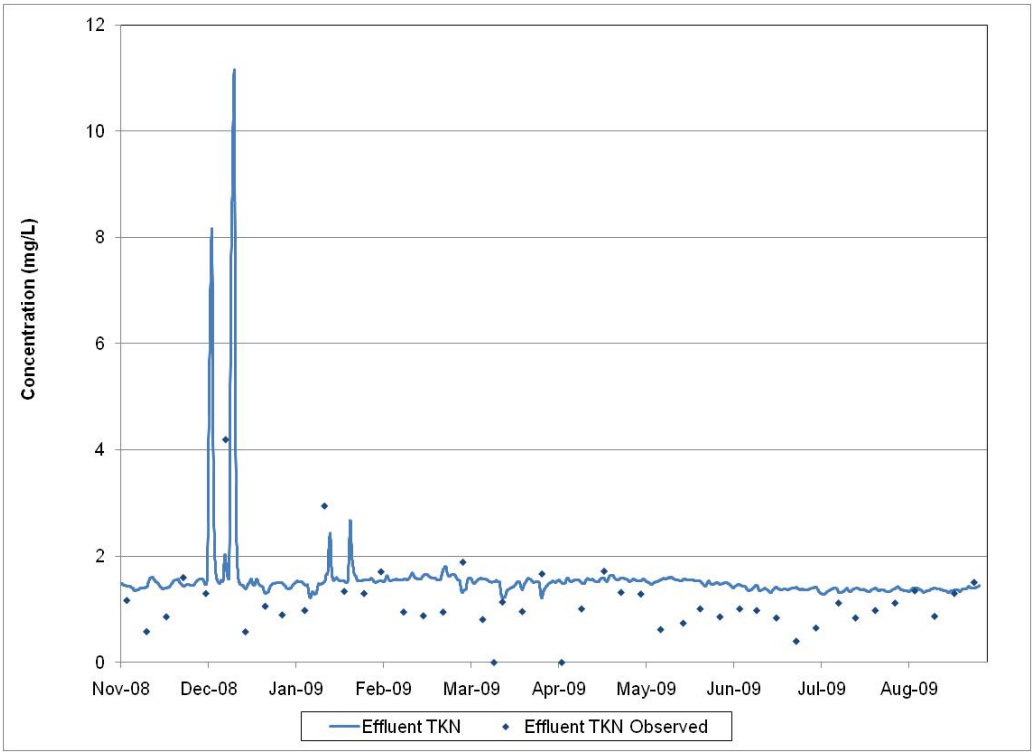


Figure 3-13
Predicted vs. Measured Effluent NO_X Values

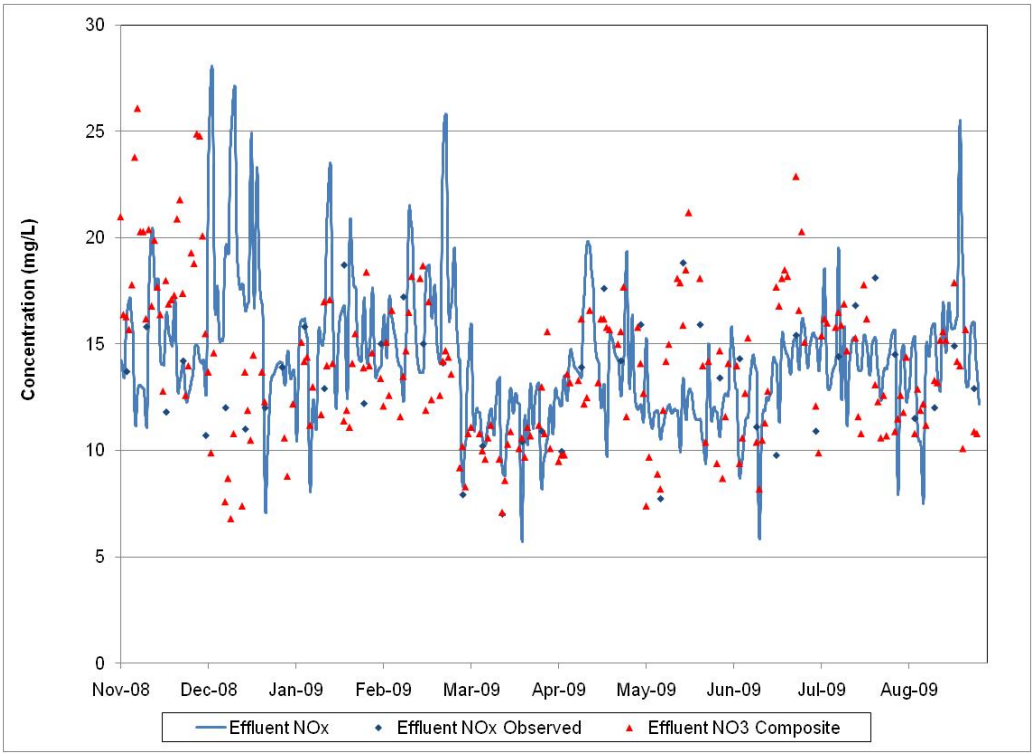
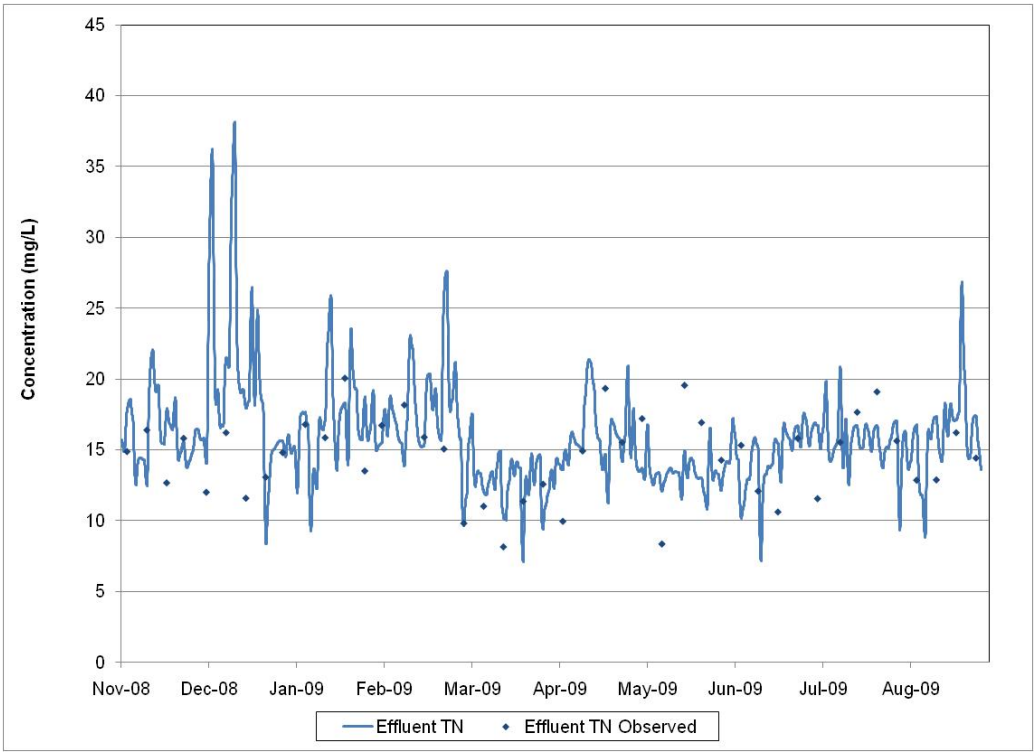


Figure 3-14
Predicted vs. Measured Effluent TN Values



Modeled and measured effluent TP values are shown in Figure 3-15. As expected the model predicted greater effluent phosphorus concentrations since alum precipitation was not modeled. Figure 3-16 presents simulated and measured effluent TSS values. The model predicts a largely constant effluent TSS concentration of 1 mg/L. Measured values are close to the modeled values and range from 0.5 to approximately 1.5 mg/L. Effluent cBOD concentrations are compared in Figure 3-17. The model predicted values less than the cBOD reportable limit (2 mg/L).

Figure 3-15
Predicted vs. Measured Effluent TP Values

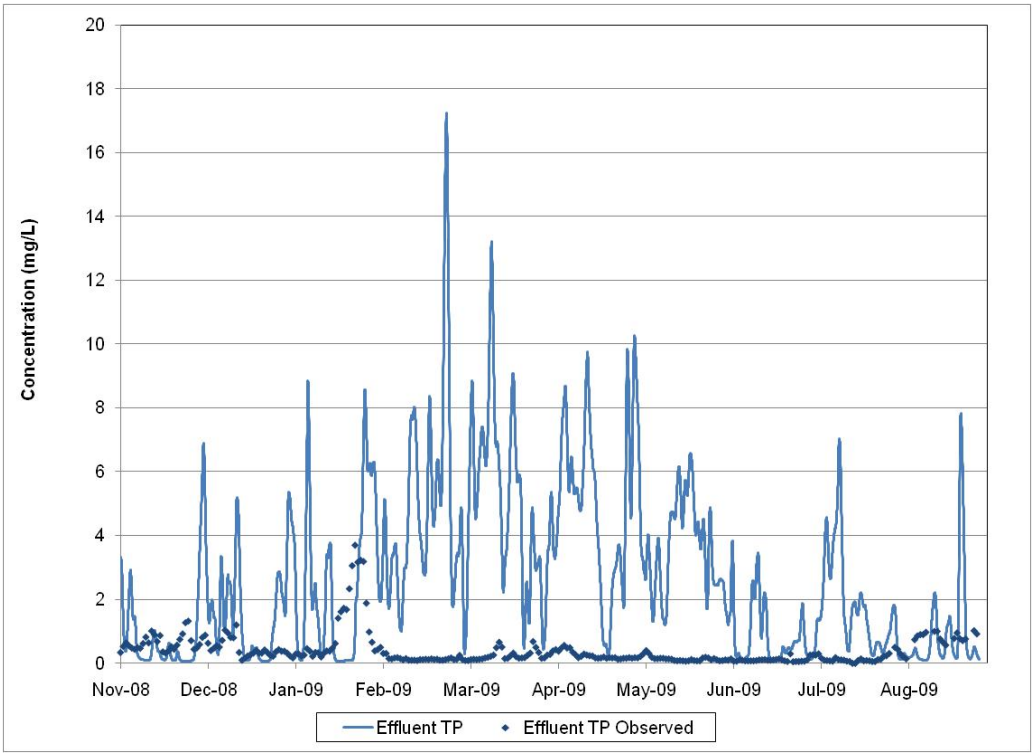


Figure 3-16
Predicted vs. Measured Effluent TSS Values

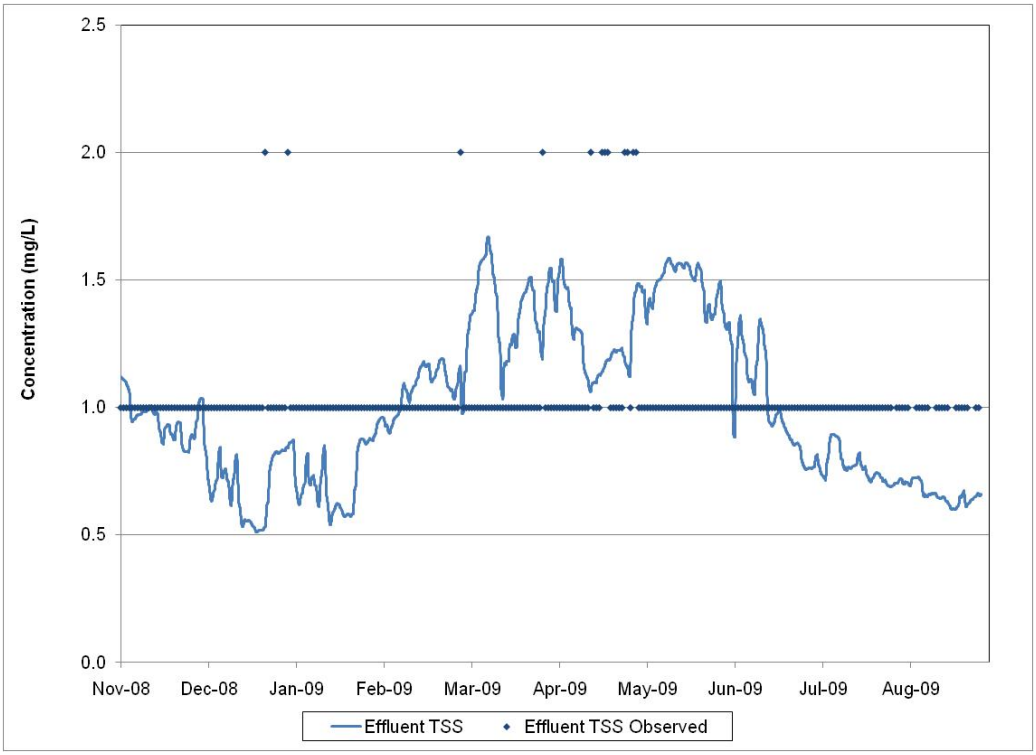
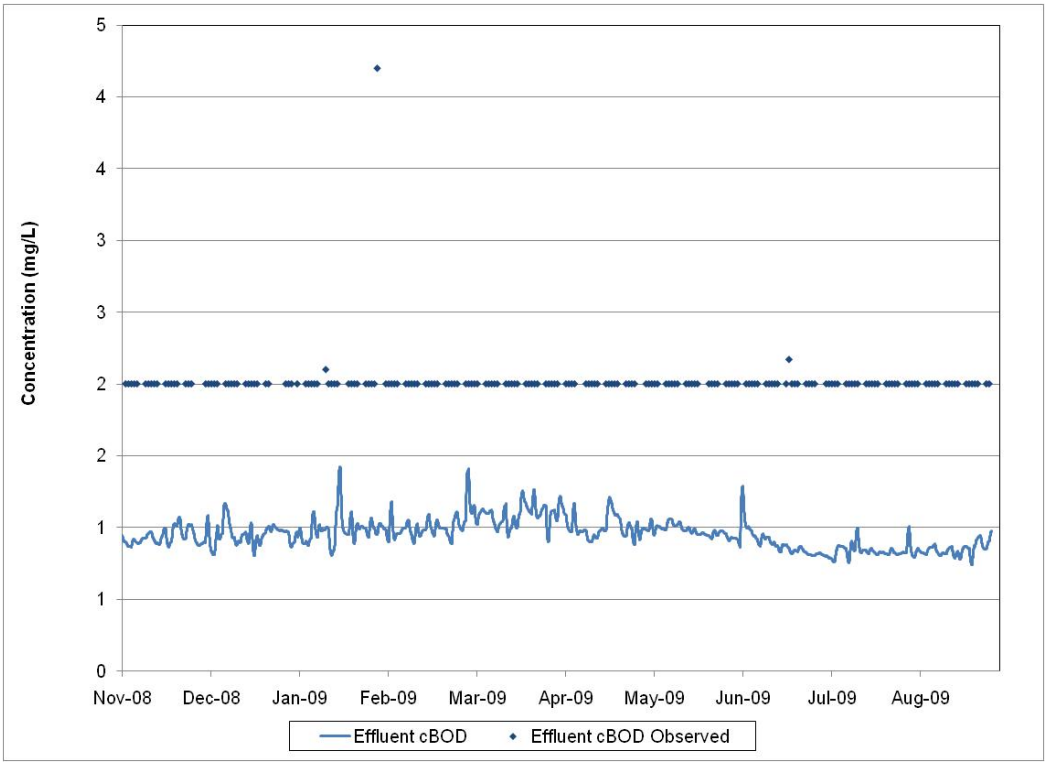


Figure 3-17
Predicted vs. Measured Effluent cBOD Values



3.6 Long-Term Calibration – Solids Processing Comparison

Figures 3-18 through 3-21 compare predicted and reported solids concentrations (in percent total solids) and volatile percentages at various points in the MFWWTP solids handling process. Fermenter effluent solids characteristics matched fairly well to observed values. Observed WAS solids percent solids (taken from the same location the RAS sample is collected) are significantly less than the model predictions, and the volatile fractions of the WAS solids were greater than the model prediction. Since the MLVSS correlated well through most of the simulated period and given the questionable quality of the RAS concentration data, the observed WAS data may be erroneous.

The characteristics of the thickened waste activated / fermented primary sludge out of the gravity belt thickeners matched well to the historical data. The model underpredicted volatile solids destruction through the digester. The existing digesters are typically operated at temperatures above 40°C, which is the upper limit of the BioWin model. Thermophillic operation of the digesters may not be accurately modeled by BioWin based on the maximum temperature limit of the model. The local kinetic parameters could be adjusted to potentially increase the solids destruction in the digester, but this was not examined during the model calibration.

Figure 3-18
Predicted vs. Measured Fermenter Percent Solids Values

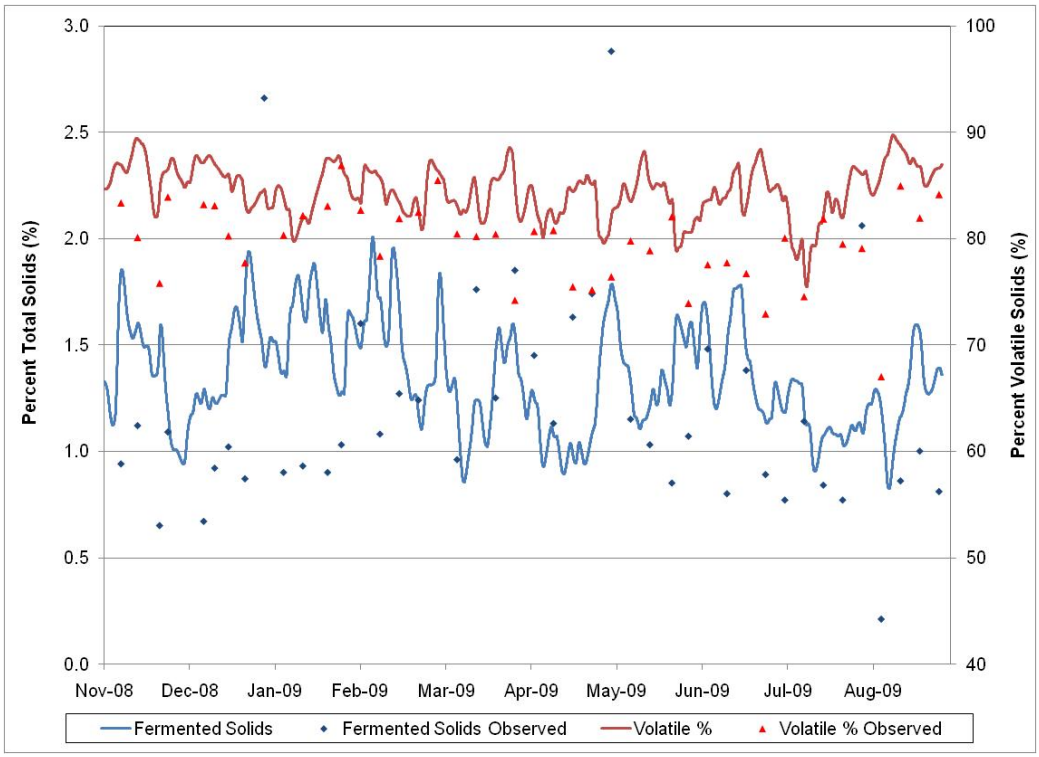


Figure 3-19
Predicted vs. Measured WAS Percent Solids Values

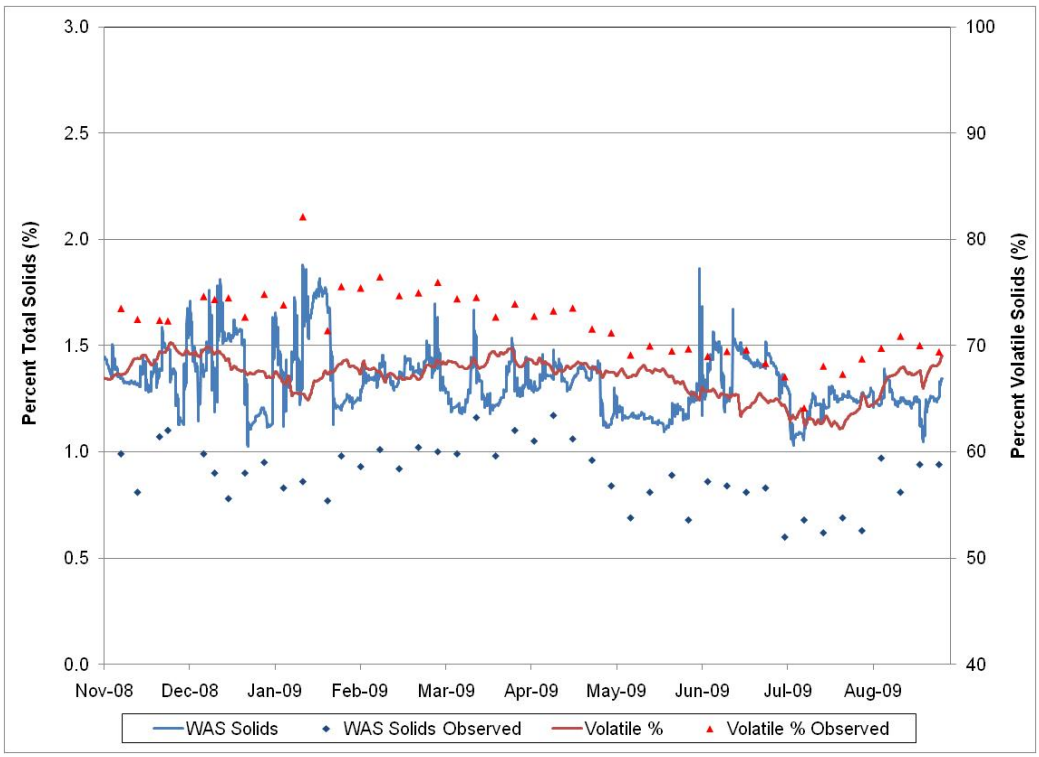


Figure 3-20
Predicted vs. Measured TWAS Percent Solids Values

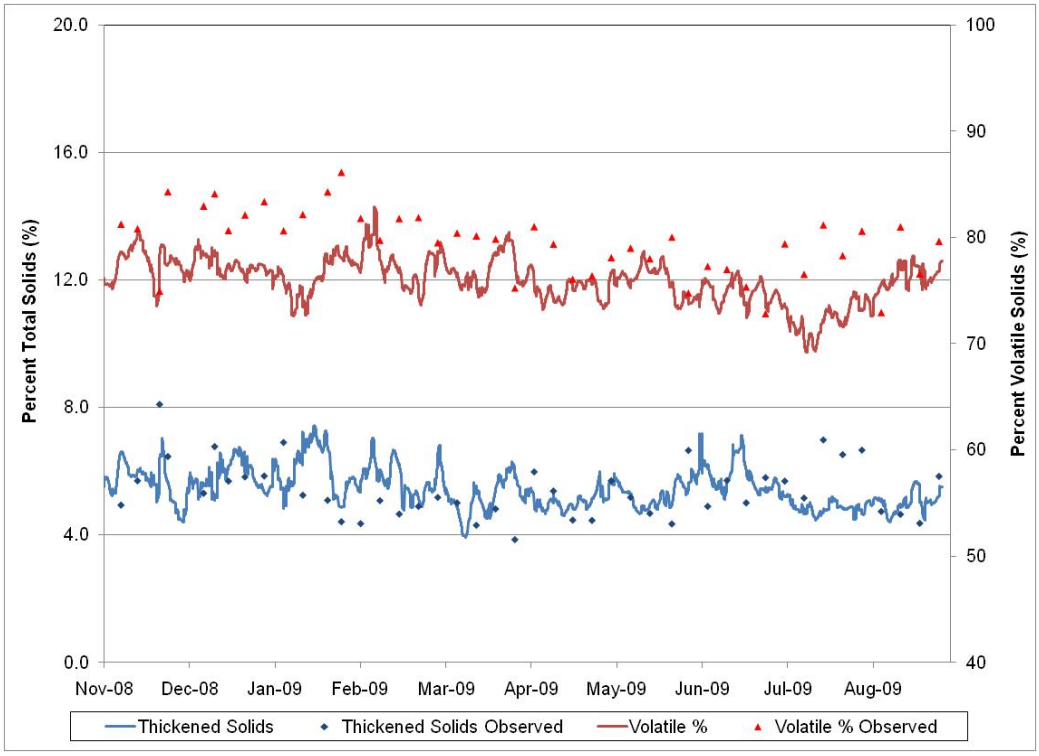
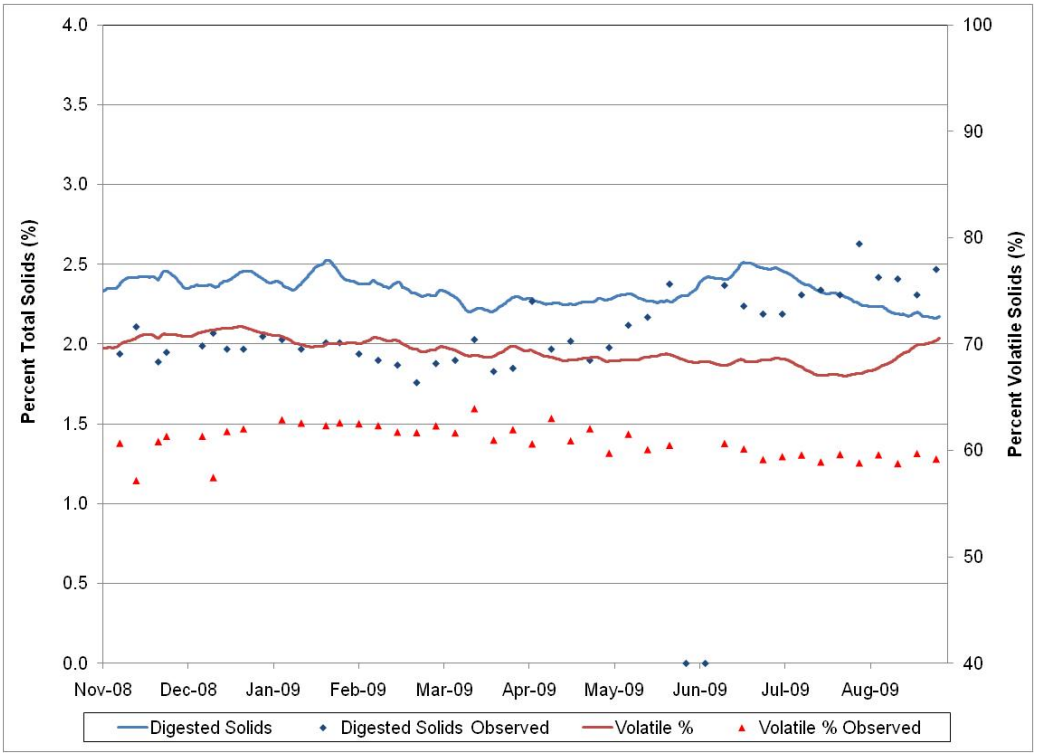


Figure 21
Predicted vs. Measured Digested Sludge Percent Solids Values

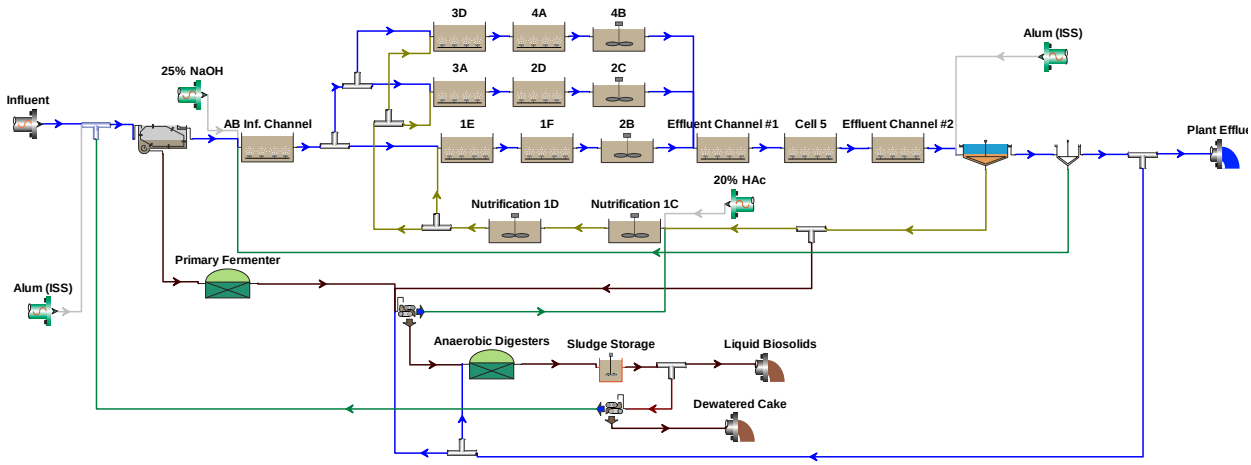


4. SHORT-TERM MODEL CALIBRATION

4.1 Short-Term Calibration Set-up and Assumptions

The next step in calibration of the model was to perform a bi-hourly dynamic simulation of the MFWWTP for the week of supplemental sampling to compare measured and simulated nutrient profiles in the bioreactor basins, influent, and plant effluent. Plant operating data supplemented with data derived from the special sampling were utilized in the model runs. The influent wastewater fractions previously presented in Table 1 were maintained. The model setup for the short-term solids calibration was similar to the long-term model and is illustrated in Figure 4-1.

Figure 4-1
Short-Term Model Calibration Setup



The short-term calibration model calibration was developed as follows:

- A two week period from August 9 through August 22, 2009 was simulated. Since special sampling was performed from August 16 through August 22, a one week period prior to the sampling period was included to develop conditions, particularly MLSS concentrations, just prior to the sampling period.

- The diurnal flow and load profiles observed during the special sampling were used to develop influent characteristics. Diurnal flow and load peaking factors were applied to the average reported flow and load for each day to develop a bi-hourly influent pattern. Diurnal profiles were performed on a weekday and weekend and applied accordingly.

- Daily mixed liquor suspended solids (MLSS) temperature data were directly input into the model.

- Daily reported primary sludge flow rates and calculated % solids removal were directly input into the model.

- The average DO concentration observed in each cell and channel during the special sampling period was input as a constant value for each cell/channel.

- An ideal secondary clarifier element was used, and secondary clarifier solids removal performance was set at 99.85%. Reported daily return activated sludge (RAS) flow rates were directly input into the model.

- The tertiary filters were modeled as a point clarifier assuming 90% removal. The reported daily filter backwash rates were directly input into the model.

- Reported daily waste activated sludge (WAS) flow rates were directly input into the model.

- Daily 25% sodium hydroxide flow rates were directly input upstream of the aeration basin influent channel.

- Daily 20% acetic acid flow rates were directly input upstream of the nutrified sludge basins.

- Alum addition was simulated as inert suspended solid (ISS) material since modeling alum precipitation dynamically severely slowed the simulations. A state variable input was developed based on a net solids production of 1.8 lbs of inert solids per gallon per day (gpd) of 48% alum solution added. The mass of solids contributed through alum assumed half of the alum reacted with phosphate to form $AlPO_4$. 65% of the total daily alum dose was added to the node where filtrate is returned upstream of the primary clarifiers. The remaining 35% was added upstream of the secondary clarifiers.

  Steady state analyses were performed with alum precipitation modeled. The results of these steady state models did not indicate major differences in predicted effluent quality as the model predicted full biological removal of phosphorus without alum addition. The addition of alum and alum precipitation did not appear to reduce BPR efficacy in the model.

- A gravity belt thickener (GBT) solids capture efficiency of 85% was assumed. The GBT thickened solids flow rate was set at 12% of the total flow to the GBT.

- The anaerobic digesters were modeled as a single digester operated at a temperature of

40°C, which is the maximum temperature the BioWin model will allow operation at.

- The flow to the rotary press was assumed to be 60% of the total digested solids flow, which is based on historical data. The solids capture through the rotary press was assumed to be 99%, and the rotary press dewatered cake flow was assumed to be 11.4% of the input flow to the press.

- Two nutrified sludge cells were assumed online during the simulation to match sampling week operation.

- Three aeration trains of three cells each (first two aerobic and last anoxic) and Cell 5a and 5b were assumed online during the simulation to match sampling week operation.

- Ideal clarifier models were used in the model.

- The default BioWin 3.0.1 kinetic and stoichiometric parameters were assumed.

- pH and DO modeling were turned on during the simulations.

4.2 Short-Term Calibration – Influent BOD and TSS Comparison

Figure 4-2 presents measured and simulated influent BOD values. A comparison of measured and modeled influent TSS values are presented in Figure 4-3. Measured influent BOD values come from daily monitoring reports (DMRs) and from composite sampling conducted during the special sampling period.

Figure 4-2
Predicted vs. Measured Influent BOD Values

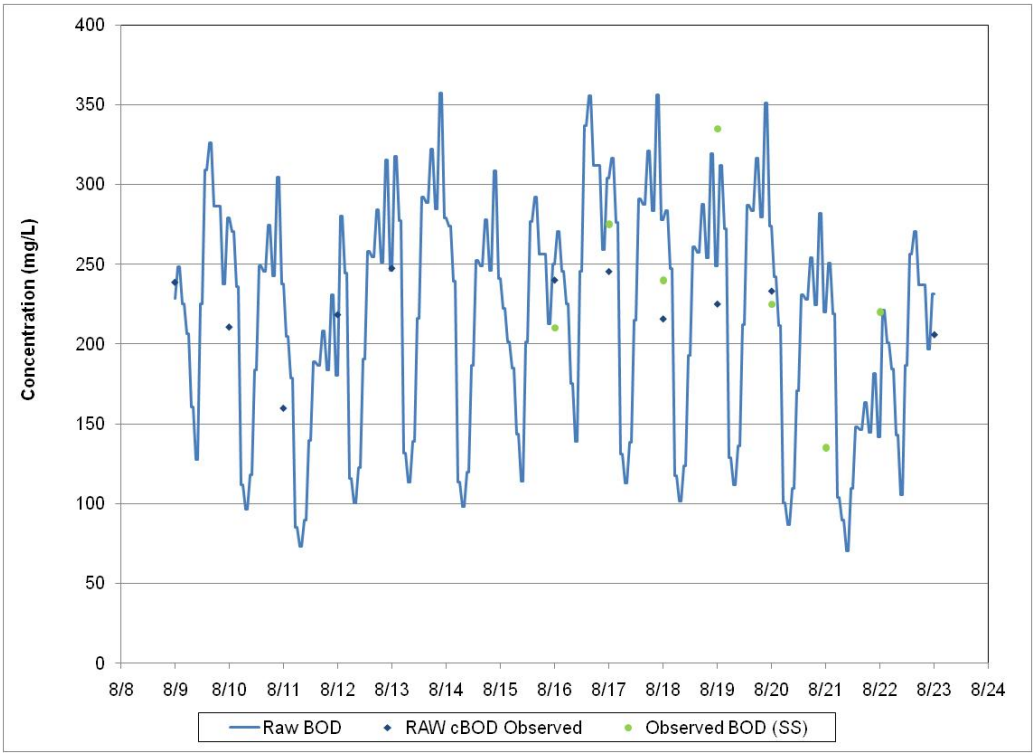
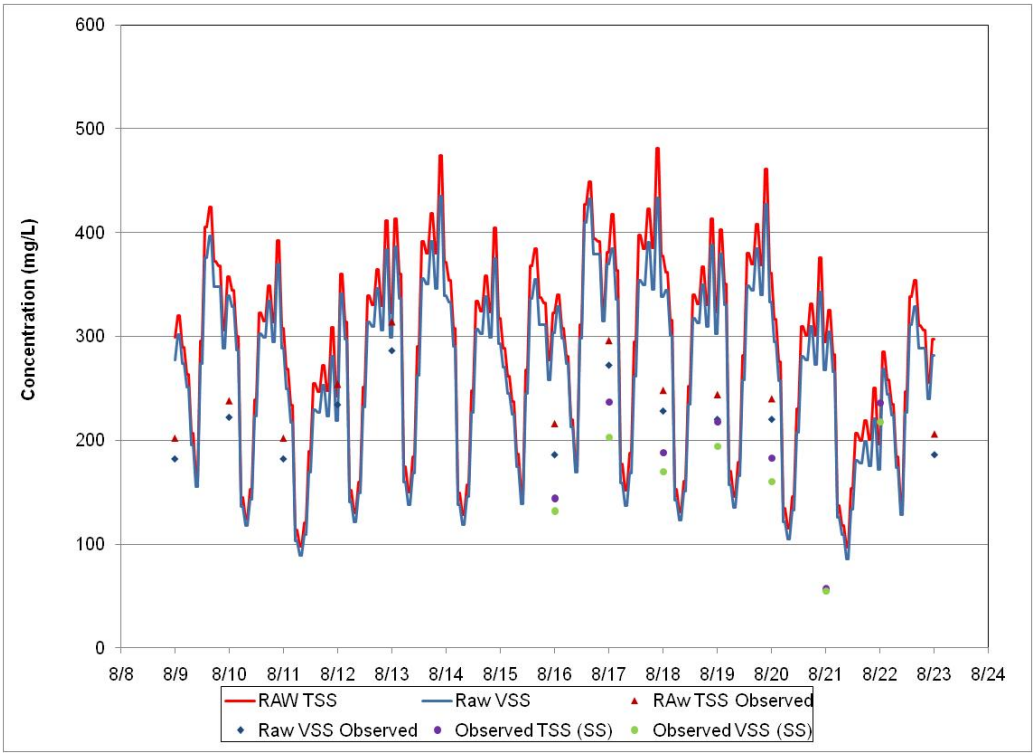


Figure 4-3
Predicted vs. Measured Influent TSS Values



4.3 Short-Term Calibration – Aeration Basin Influent Comparison

Comparisons of observed and modeled aeration basin influent parameters are presented in Figures 4-4 through 4-7.

Figure 4-4
Predicted vs. Measured Aeration Basin Influent BOD Values

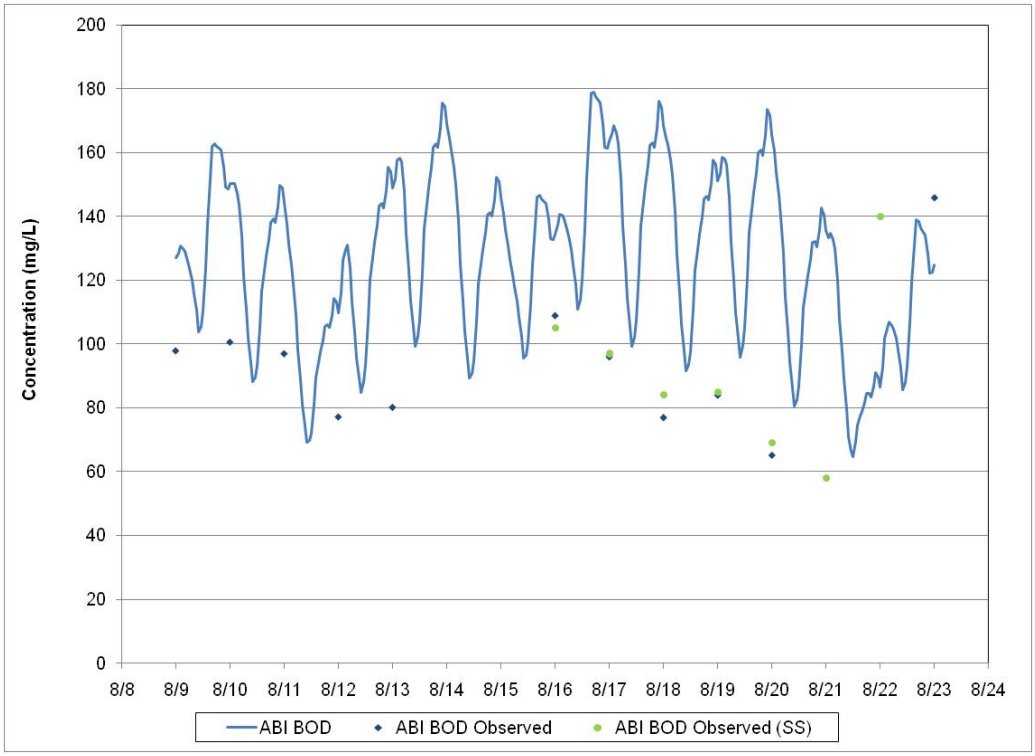


Figure 4-5
Predicted vs. Measured Aeration Basin Influent TSS Values

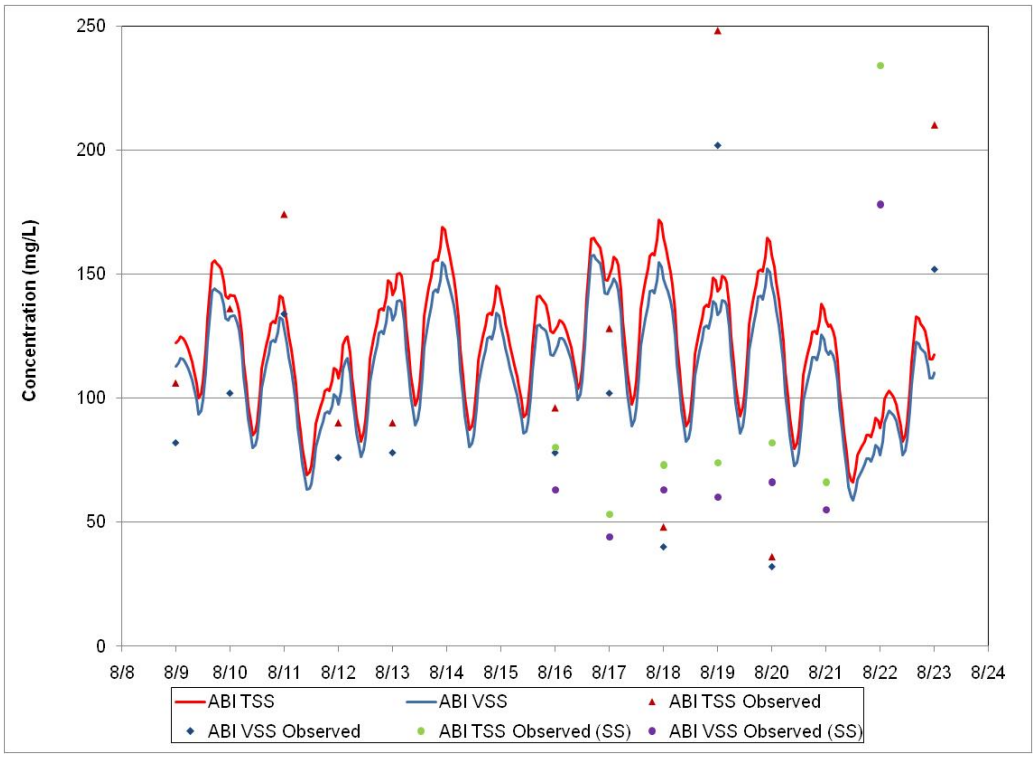


Figure 4-6
Predicted vs. Measured Aeration Basin Influent TP Values

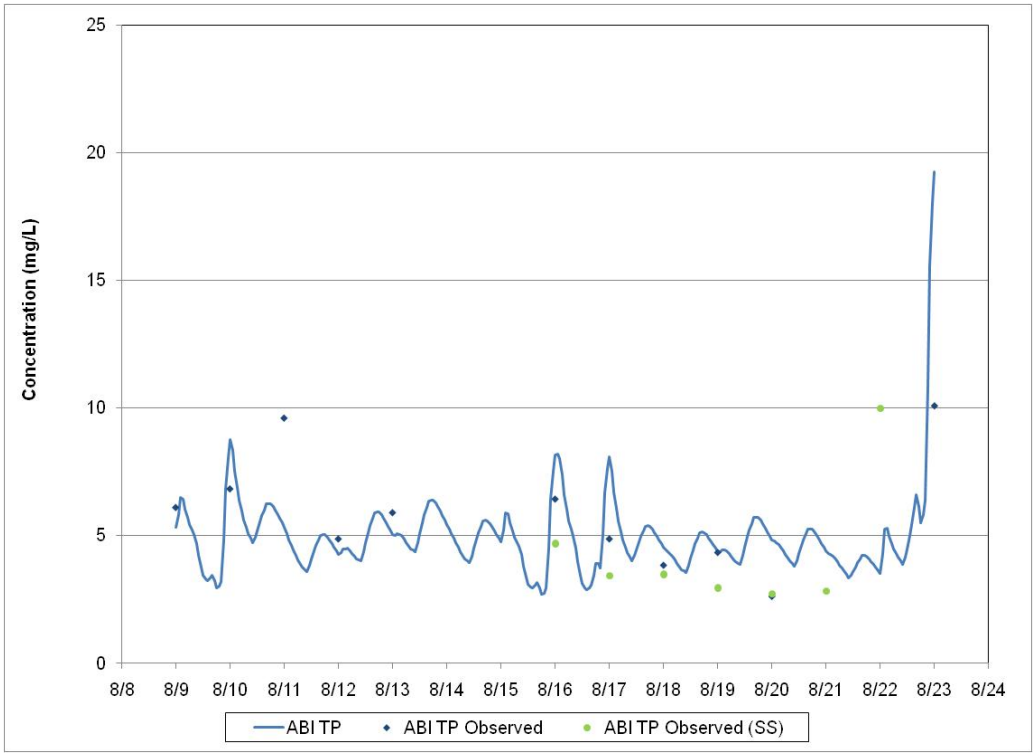
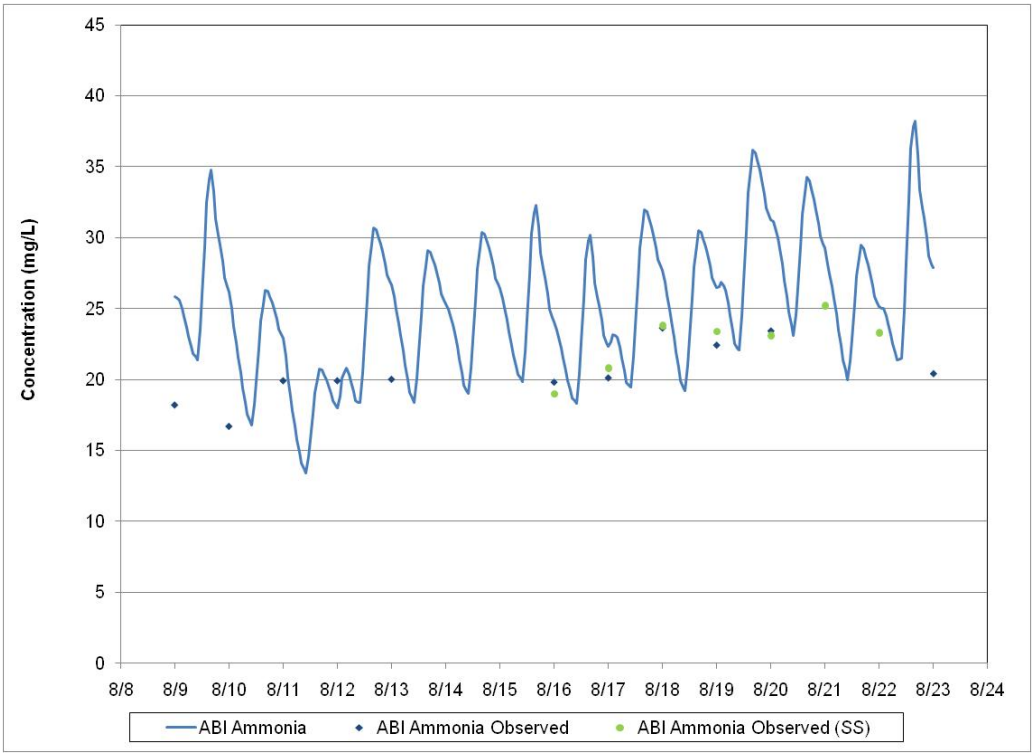


Figure 4-7
Predicted vs. Measured Aeration Basin Influent NH_3 Values

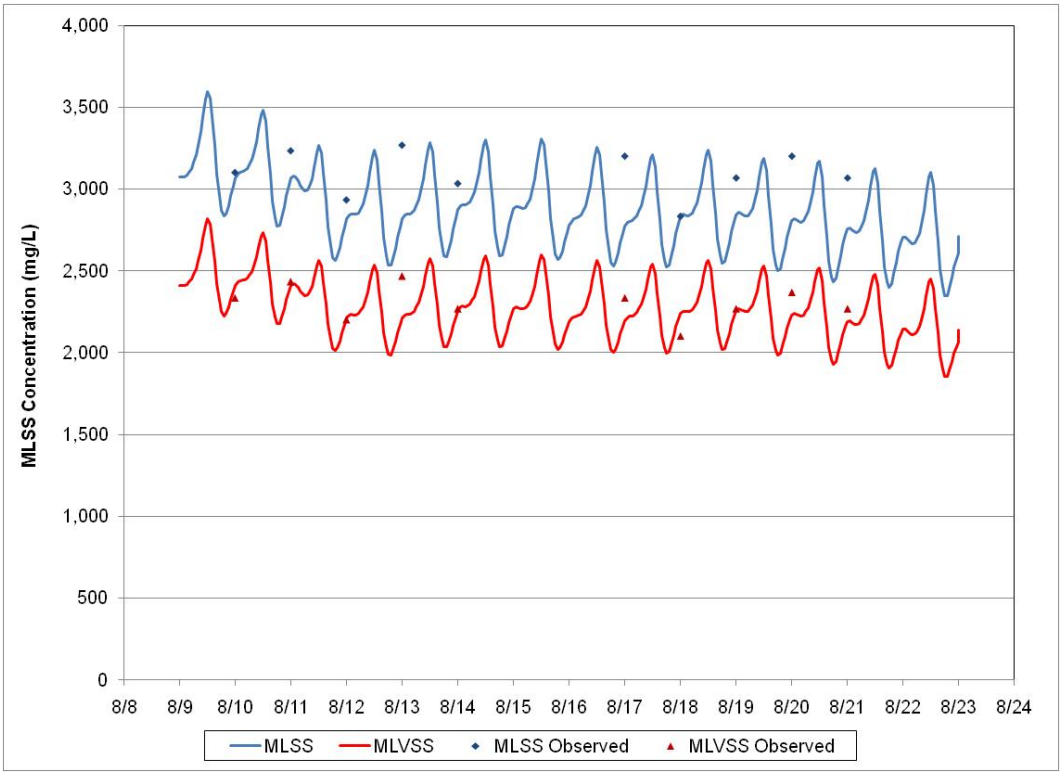


4.4 Short-Term Calibration – Mixed Liquor Suspended Solids Comparison

The simulated aeration basin MLSS and MLVSS concentrations are compared to observed concentrations in Figure 4-8. There was very good correlation, and the sampling week simulation was started from an appropriate MLSS concentration.

Figure 4-8

Predicted vs. Measured MLSS and MLVSS Values



4.5 Short-Term Calibration – Nutrient Profile Comparison

Nutrient profiles through the MFWWTP were performed twice per day for two days during the special sampling week. The nutrient profiles consisted of single grab samples at numerous locations through the process. The measured nutrient concentrations were compared to the simulated values. The simulated values are based on the dynamic model result at the approximate time of sampling.

4.5.1 Train A Ammonia Profile Comparison

Figures 4-9 through 4-12 compare the simulated and measured ammonia concentrations through Train A (Cells 1E, 1F, 2B) during the special sampling period. The model simulated the ammonia concentrations very well during the first day of sampling. There was less consistency with the observed values during the second sampling day. The morning nutrient profile on Day 2 indicated better Cell 1E nitrification performance than predicted by the model. The afternoon profile indicated poorer performance than predicted. Full nitrification was observed and predicted by the end of Effluent Channel 1. The model predicted significantly greater ammonia concentrations in the NSL basins. Aeration basin influent ammonia values were similar with the exception of the afternoon of Day 2.

Figure 4-9
Predicted vs. Measured NH_3 Values – Train A – Day 1 AM

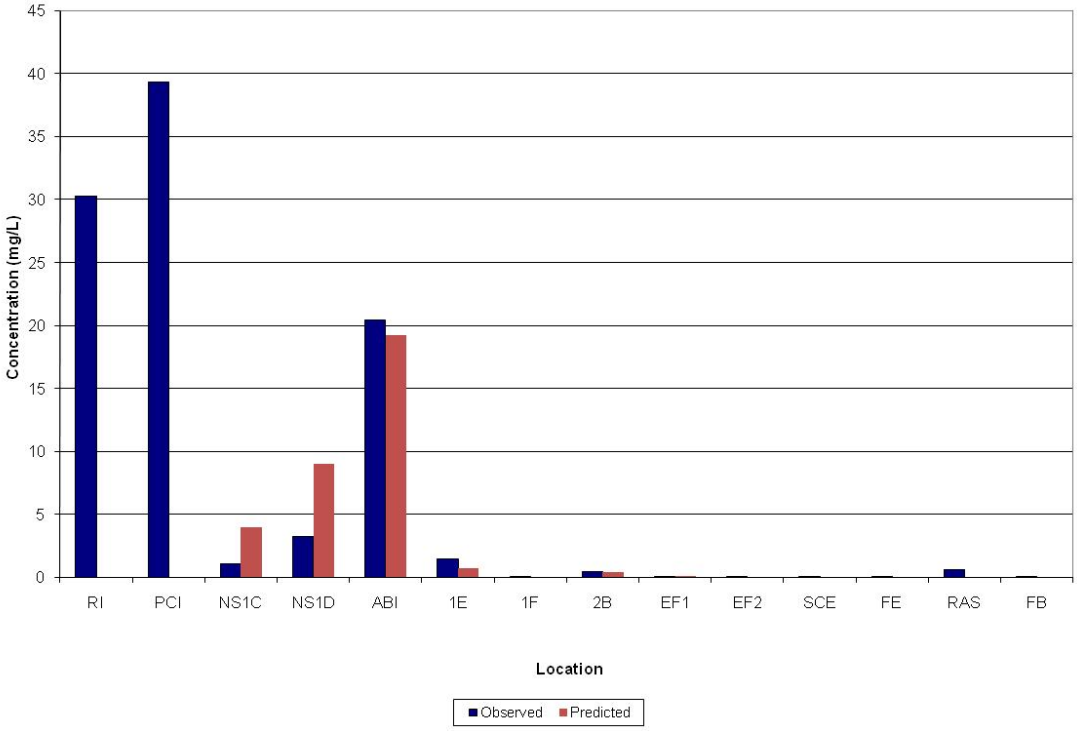


Figure 4-10
Predicted vs. Measured NH_3 Values – Train A – Day 1 PM

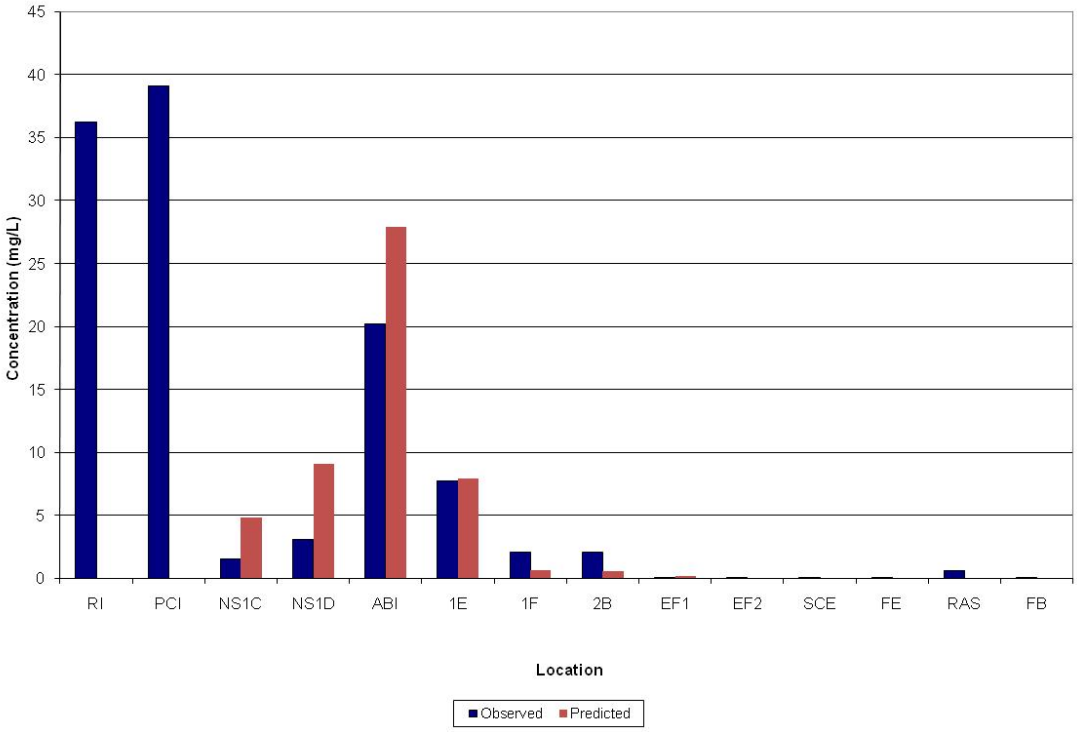


Figure 4-11
Predicted vs. Measured NH_3 Values – Train A – Day 2 AM

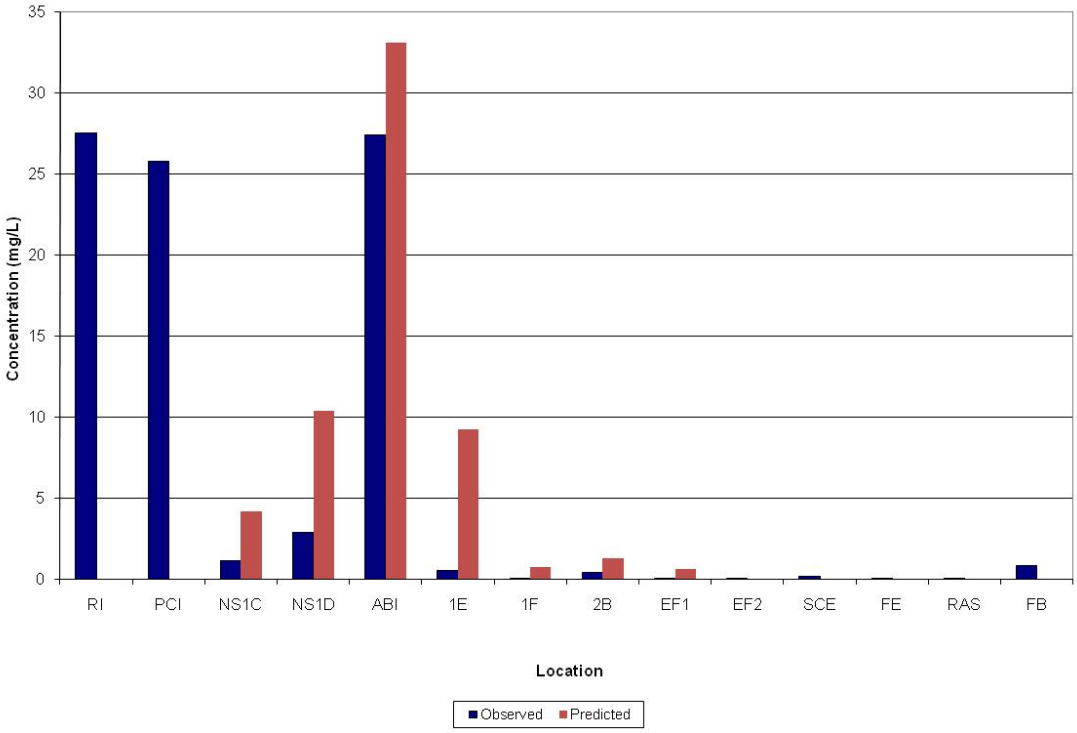
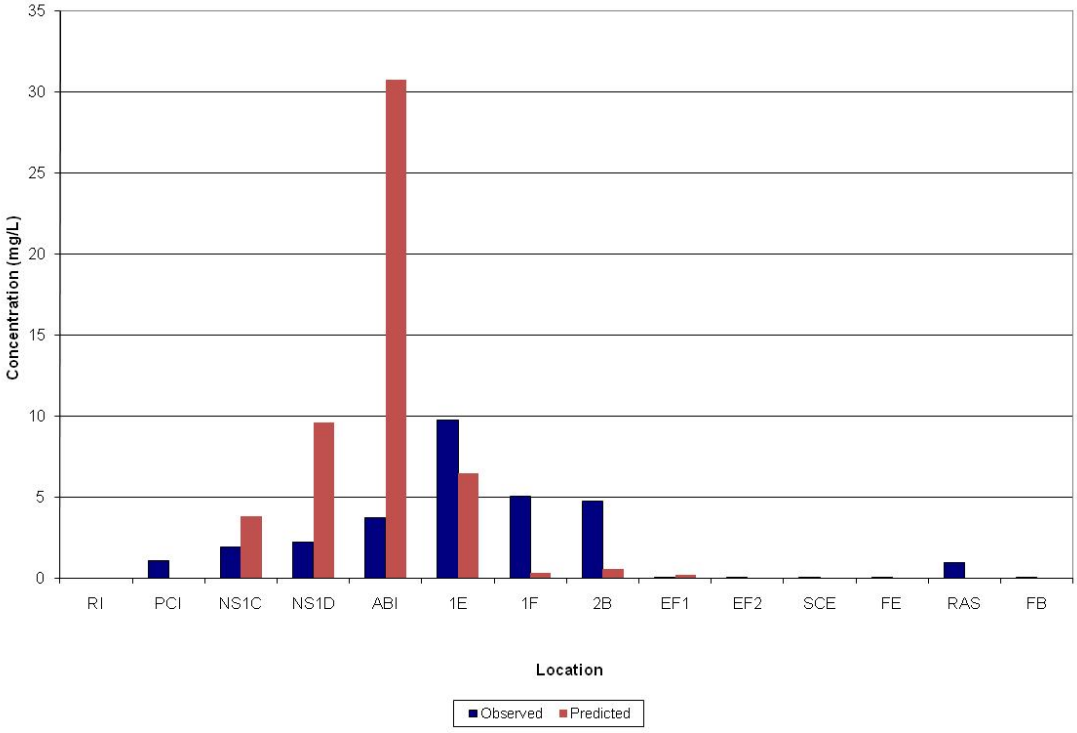


Figure 4-12
Predicted vs. Measured NH_3 Values – Train A – Day 2 PM



4.5.2 Train A Nitrate Profile Comparison

Figures 4-13 through 4-16 compare the simulated and measured nitrate concentrations through Train A during the special sampling period. The model simulated nitrate concentrations well during each sampling event. Effluent nitrate concentrations matched very well, indicating that the impact of nitrogen from the solids recycle stream is being appropriately modeled as the nitrogen balance through the plant is consistent between modeled and observed values.

Figure 4-13
Predicted vs. Measured NO_3 Values – Train A – Day 1 AM

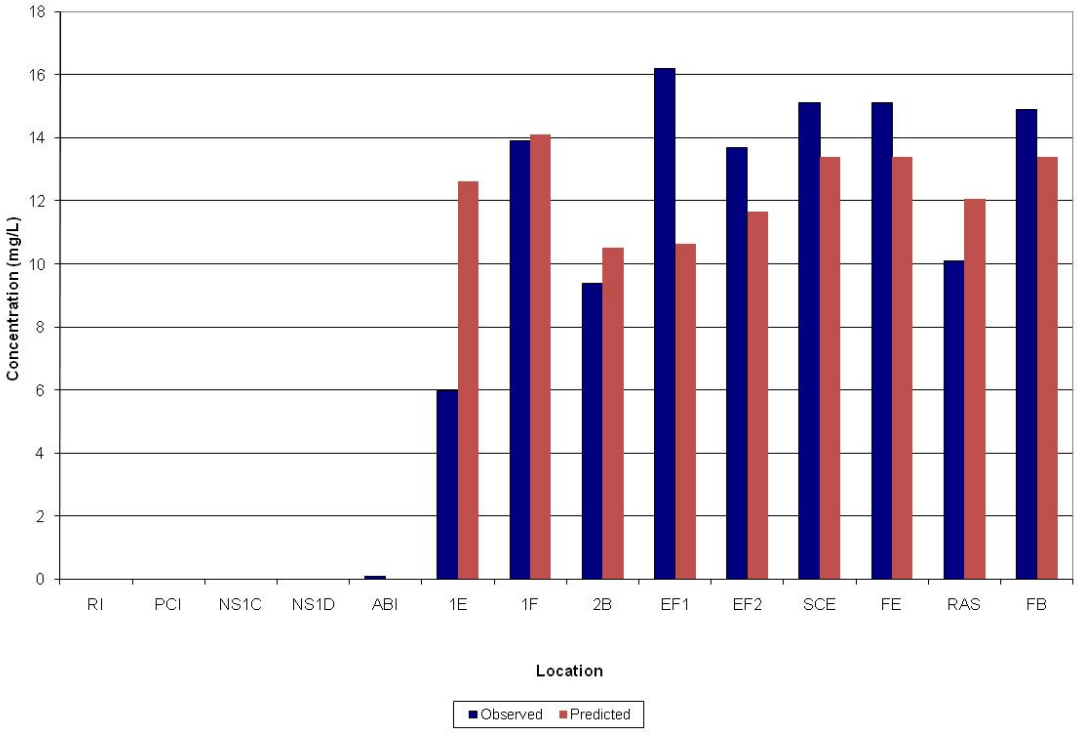


Figure 4-14
Predicted vs. Measured NO_3 Values – Train A – Day 1 PM

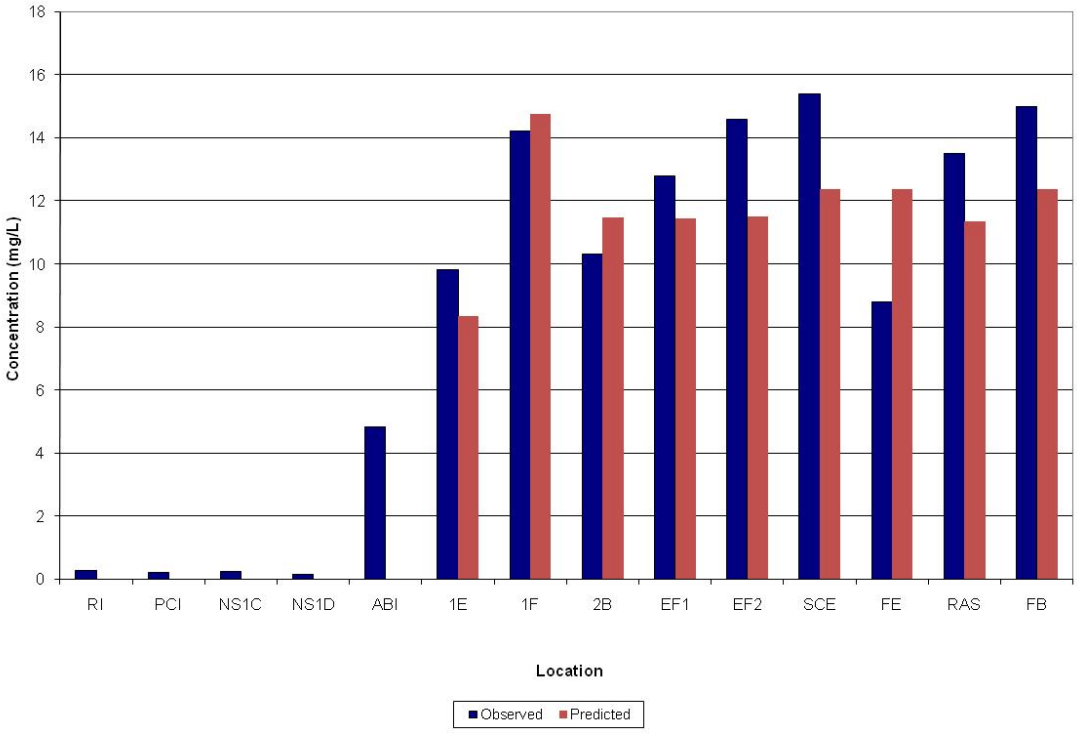


Figure 4-15
Predicted vs. Measured NO_3 Values – Train A – Day 2 AM

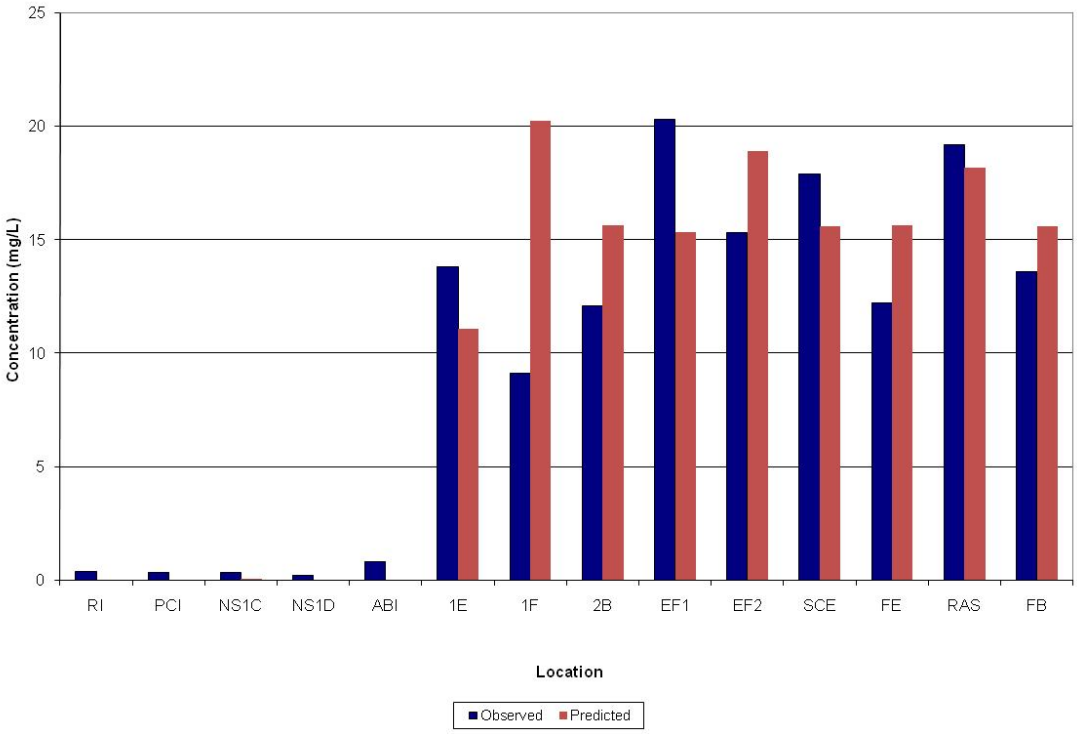
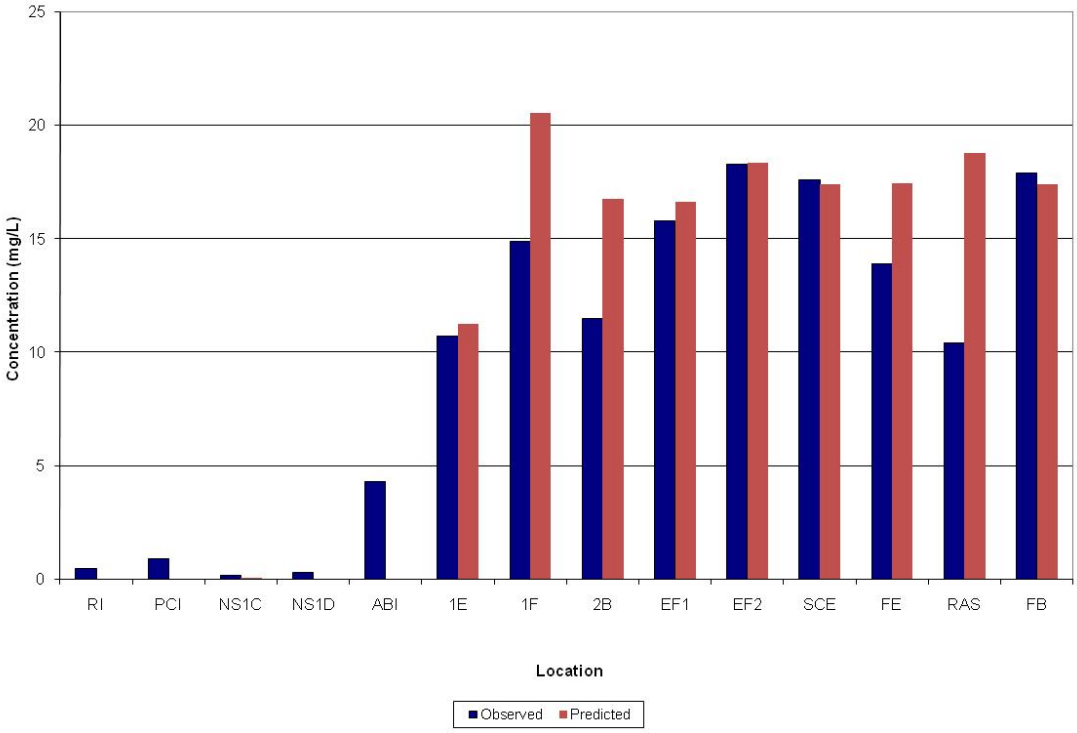


Figure 4-16
Predicted vs. Measured NO_3 Values – Train A – Day 2 PM



4.5.3 Train A Phosphate Profile Comparison

Figures 4-17 through 4-20 compare the simulated and measured ortho-phosphate - phosphorus (PO_4-P) concentrations through Train A during the special sampling period. The model predicted a much greater degree of biological phosphorus release in the nutrified sludge cells than observed in the field. Steady-state sensitivity analyses were performed to determine whether the fermentate or acetic acid feed was contributing to the high degree of release. The model was run under conditions of no fermentation in the fermenter (by reducing the fermentation kinetics to zero) and no acetic acid addition to the basins. Excessive biological phosphorus release was still observed during this sensitivity analysis. Steady state sensitivity analyses were also performed to determine the impacts of phosphorus precipitation modeling on phosphorus release in the nutrified sludge cells. As in the case of the other simulations very high ortho-phosphate concentrations were predicted.

Aeration basin influent ortho-phosphate concentrations were similar to the observed values without modeling phosphorus precipitation. The model predicted full phosphate removal in the aeration basins, which was confirmed by the model. Phosphorus was removed by the second aerated cell (Cell 1F) for each sample.

Figure 4-17
Predicted vs. Measured PO_4-P Values – Train A – Day 1 AM

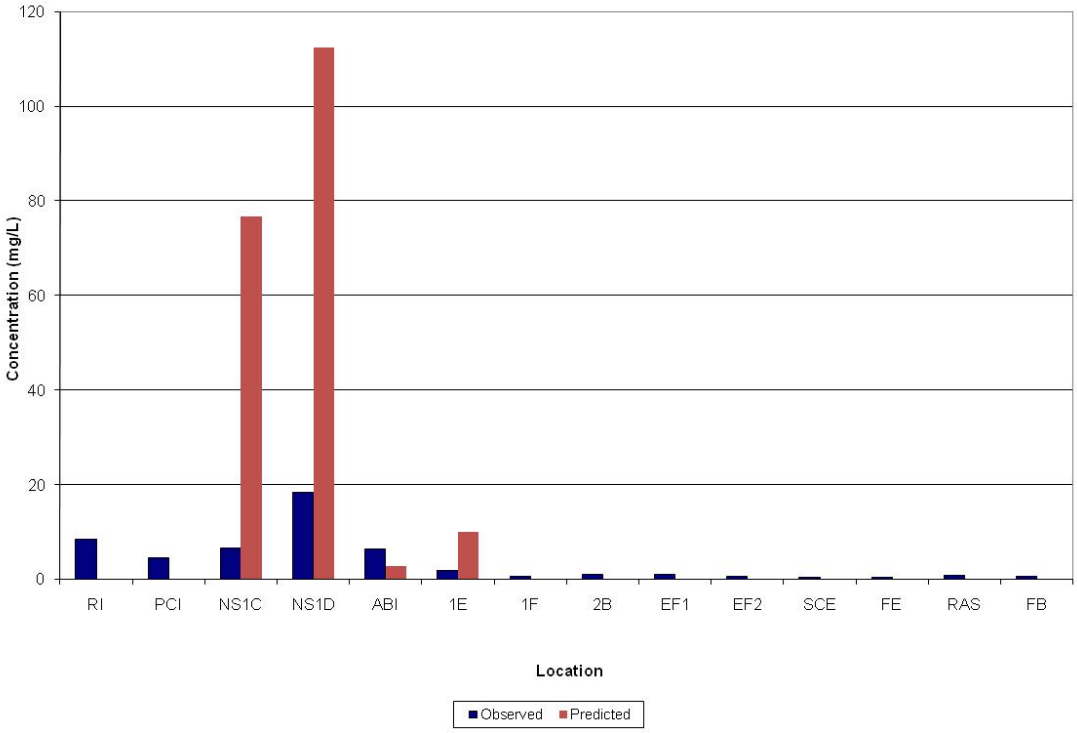


Figure 4-18
Predicted vs. Measured PO_4-P Values – Train A – Day 1 PM

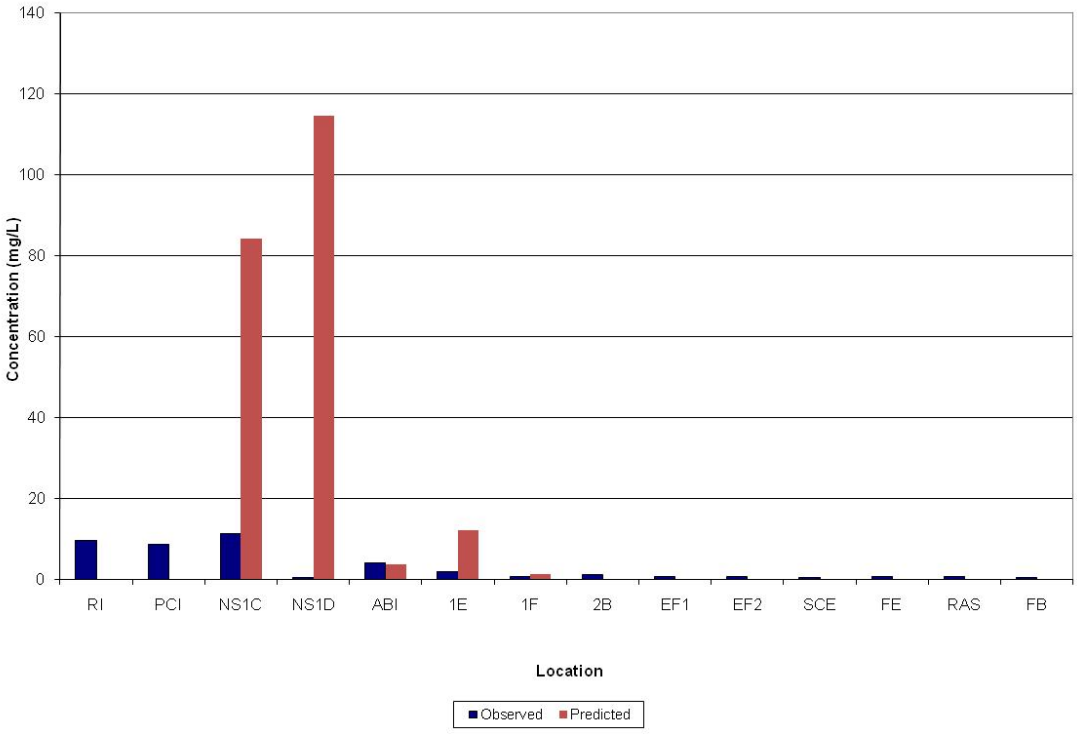


Figure 4-19
Predicted vs. Measured PO_4-P Values – Train A – Day 2 AM

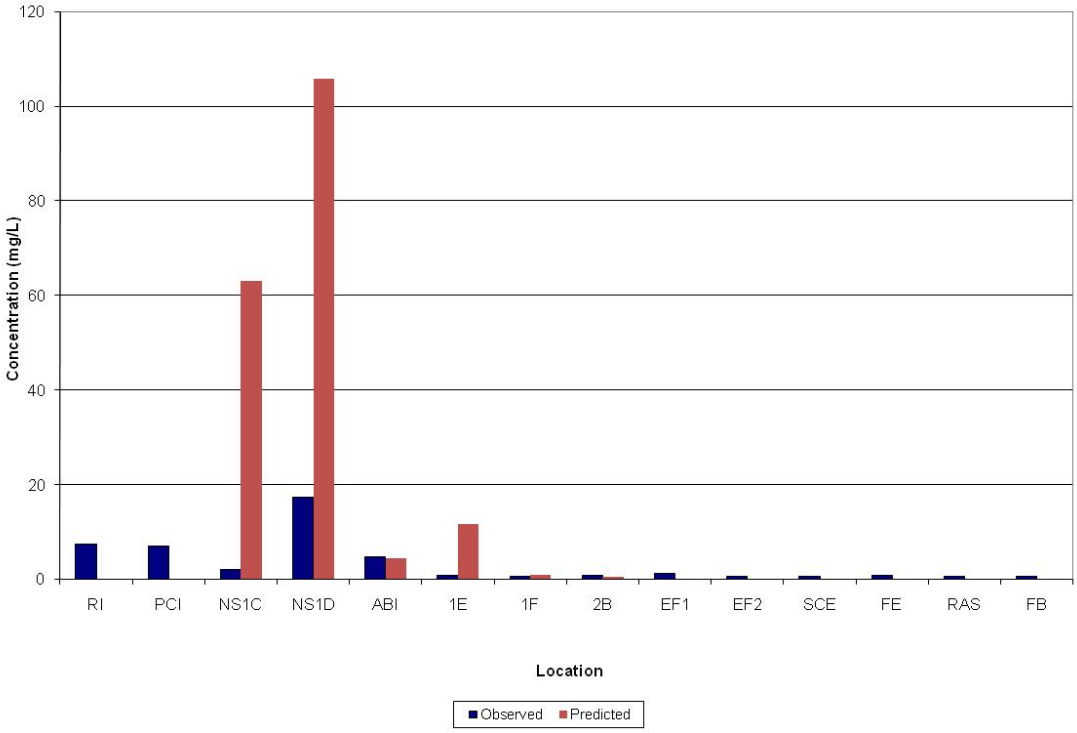
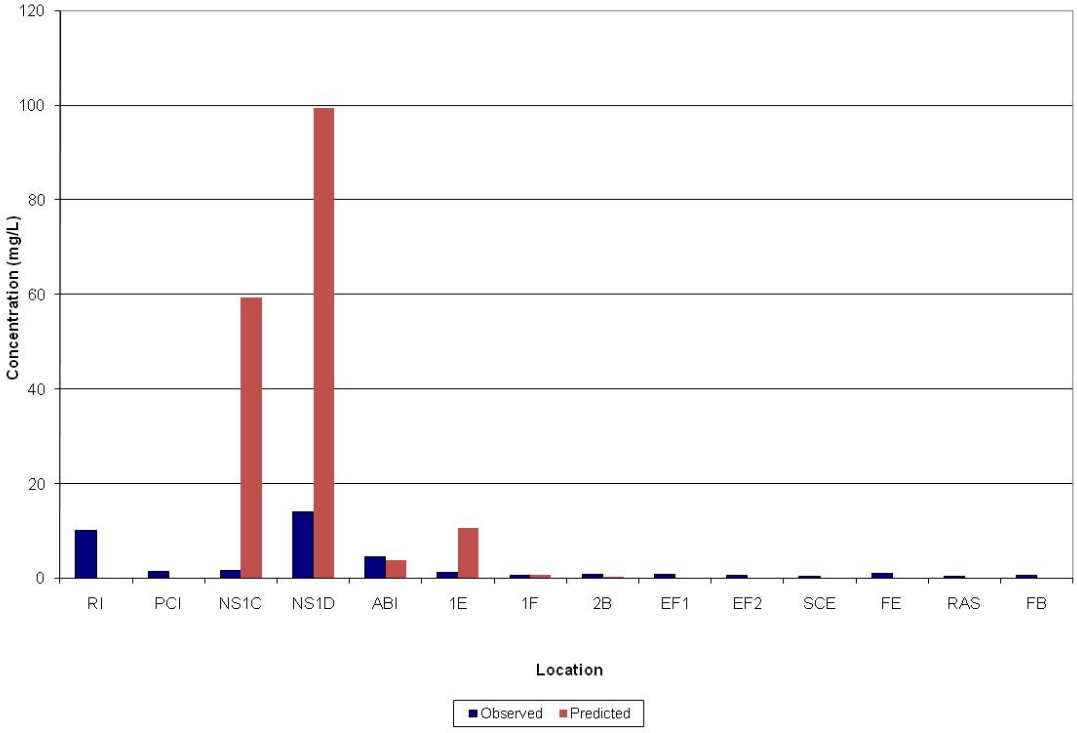


Figure 4-20
Predicted vs. Measured PO_4-P Values – Train A – Day 2 PM



4.5.4 Train B Profile Comparison

Figure 4-21 through 4-26 present a comparison of modeled and measured ammonia, nitrate and orthophosphate concentrations through Train B (Cells 3A, 2D, 2C). The simulated values are based on the dynamic model result at the approximate time of sampling. The measured values are from single grab samples at each location. Train B was only sampled on the second sampling day.

Ammonia through Train B was simulated fairly well in Cells 2D and 2C. Nitrate concentrations through the basins matched very well. Ortho-phosphate was reduced by Cell 2D as observed, although Cell 3A concentrations were greater due to the high release in the nutrified sludge basins.

Figure 4-21
Predicted vs. Measured NH_3 Values – Train B – Day 2 AM

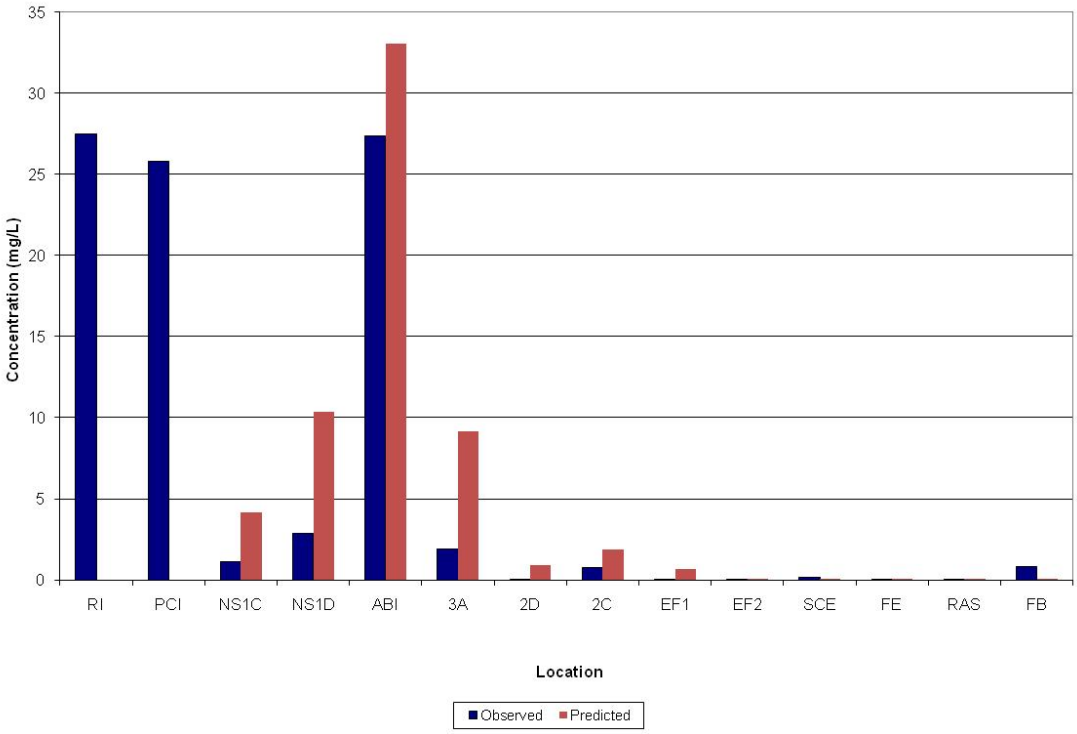


Figure 4-22
Predicted vs. Measured NH_3 Values – Train B – Day 2 PM

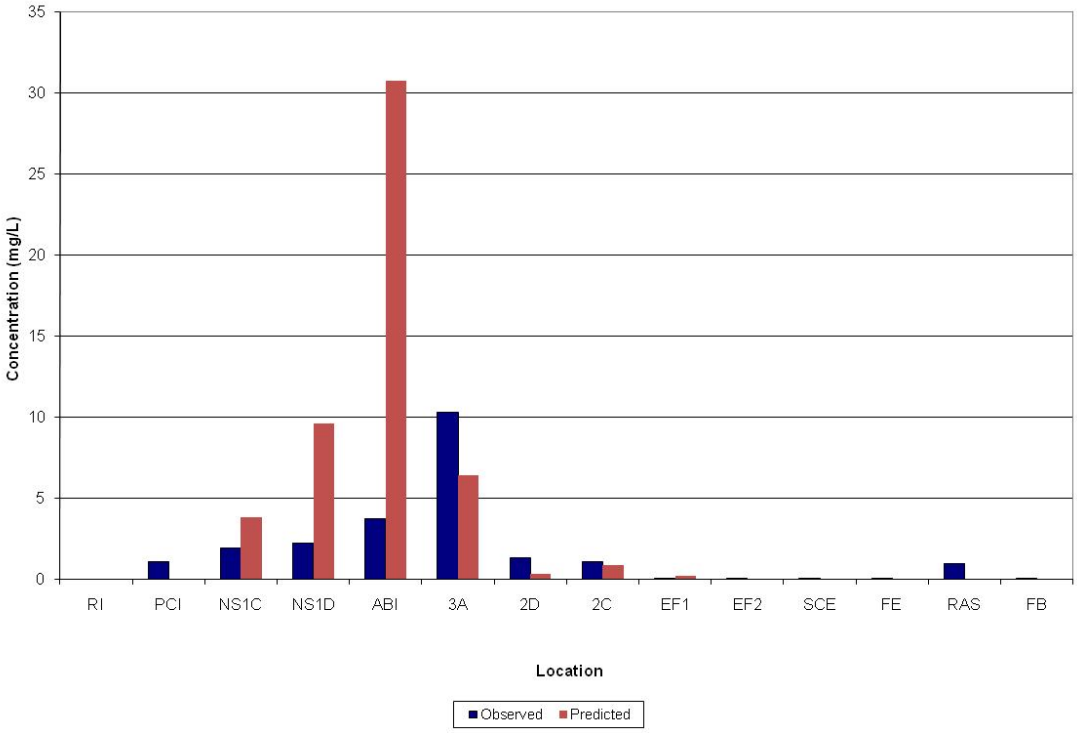


Figure 4-23
Predicted vs. Measured NO_3 Values – Train B – Day 2 AM

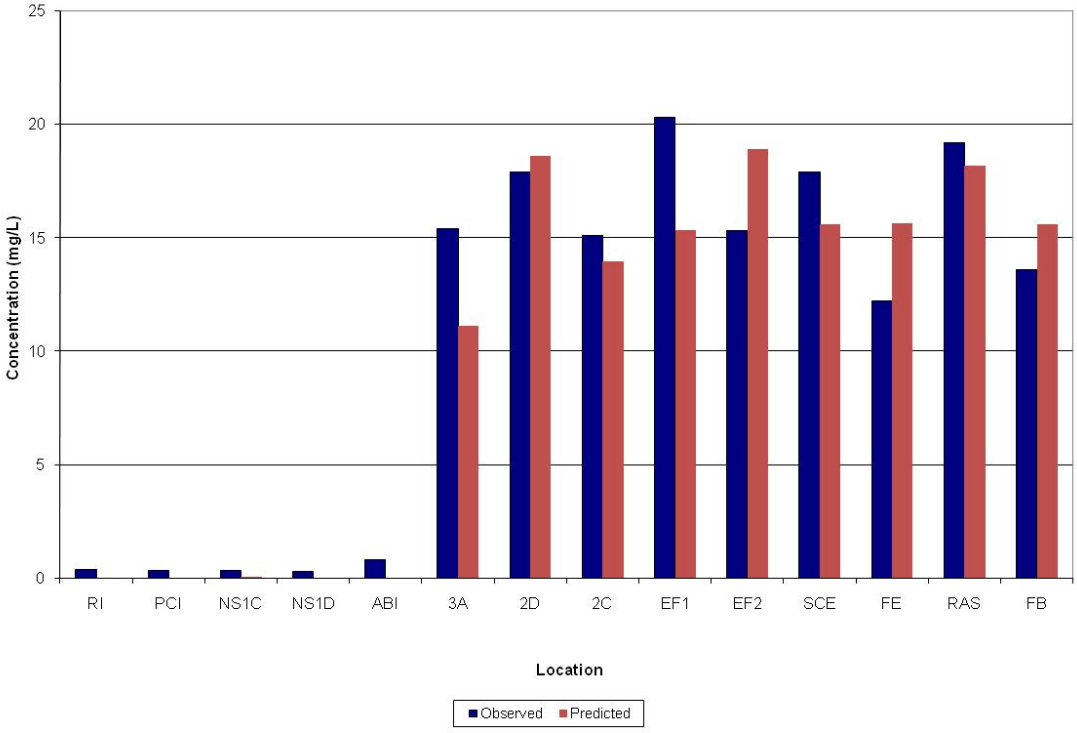


Figure 4-24
Predicted vs. Measured NO_3 Values – Train B – Day 2 PM

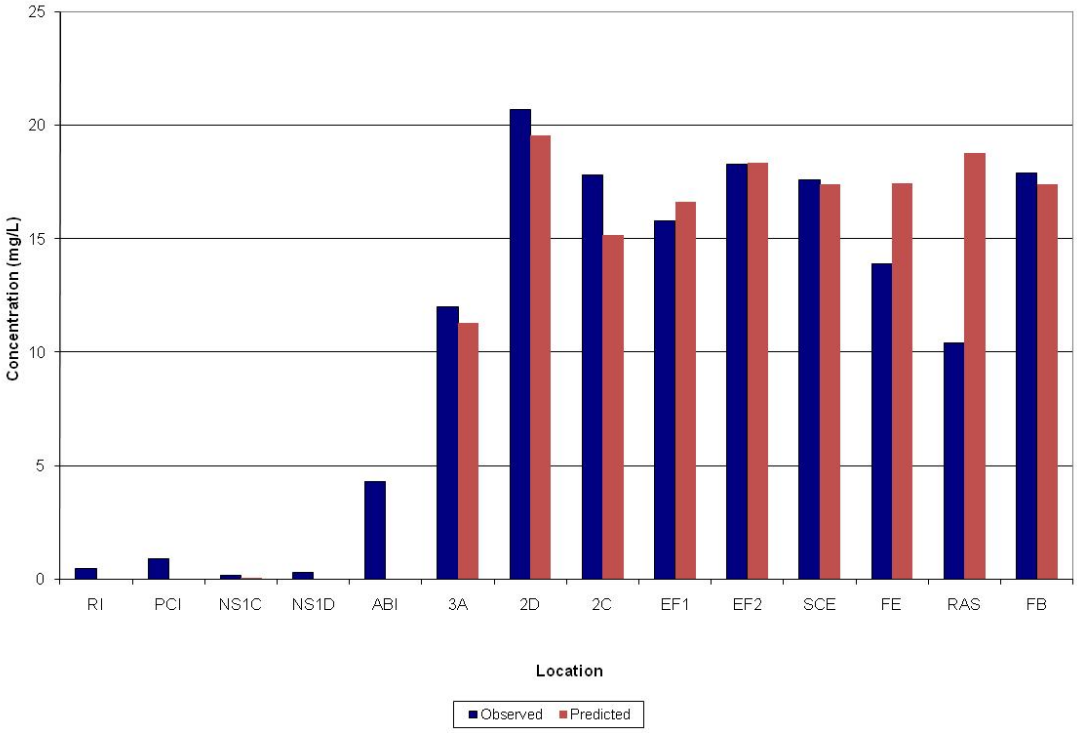


Figure 4-25
Predicted vs. Measured PO_4-P Values – Train B – Day 2 AM

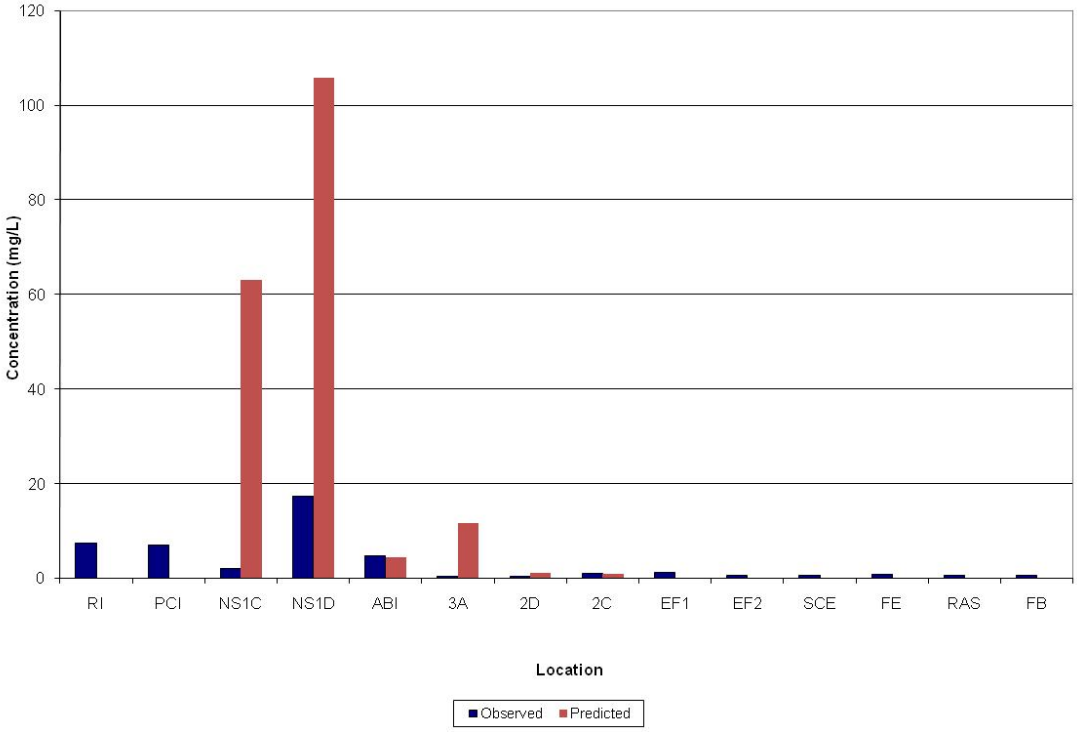
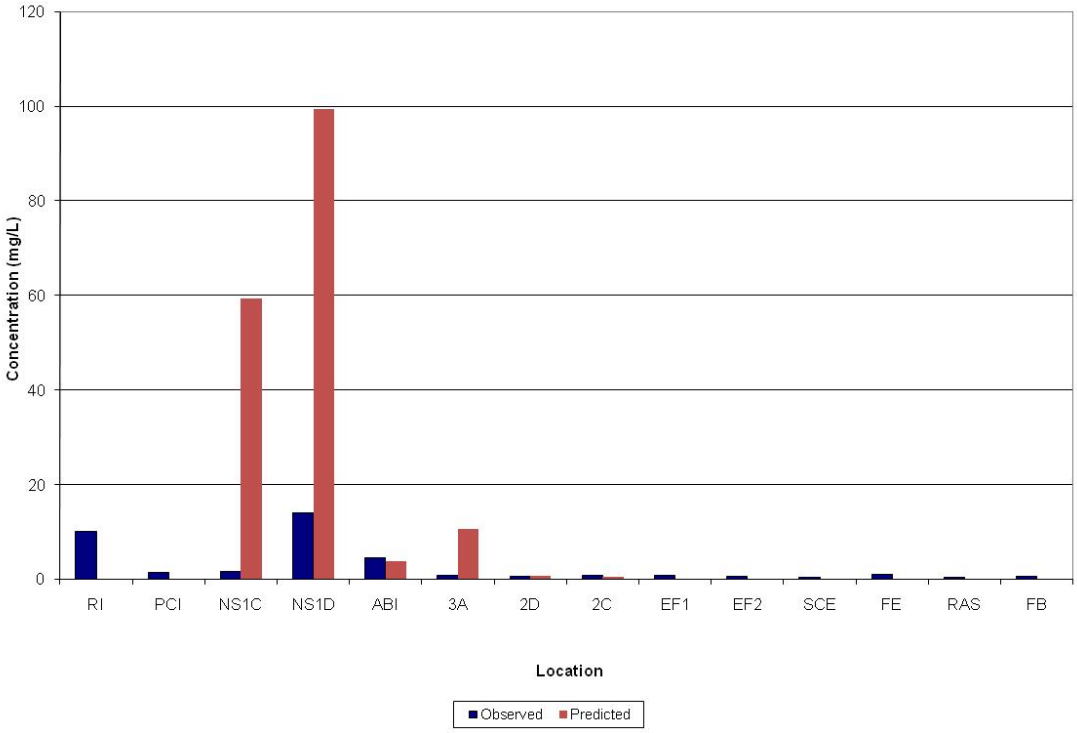


Figure 4-26
Predicted vs. Measured PO_4-P Values – Train B – Day 2 PM



4.5.5 Train C Profile Comparison

Observed and predicted nutrient concentrations through Train C (Cells 3D, 4A, 4B) are presented in Figure 4-27 through Figure 4-28. Similar to the other trains, the simulated values are based on the dynamic model result at the approximate time of sampling. The measured values are from single grab samples at each location. Predicted nitrogen concentrations in Train C exhibited more variability in matching observed values compared to Trains A and B.

Figure 4-27
Predicted vs. Measured NH_3 Values – Train C – Day 1 AM

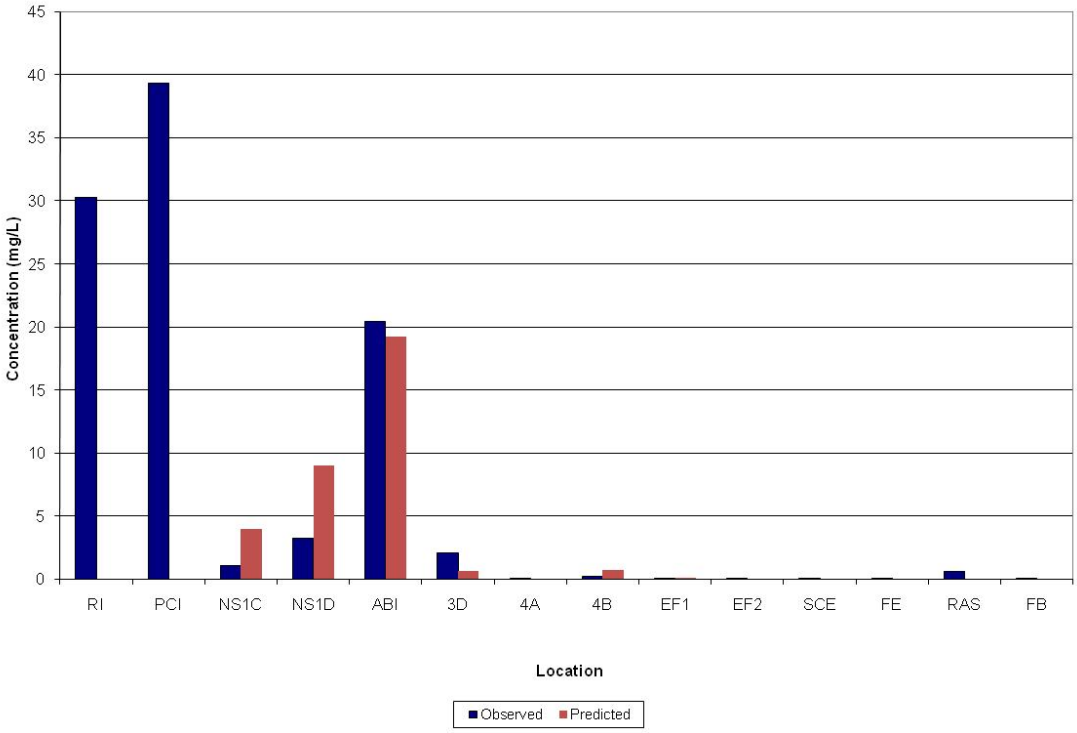


Figure 4-28
Predicted vs. Measured NH_3 Values – Train C – Day 1 PM

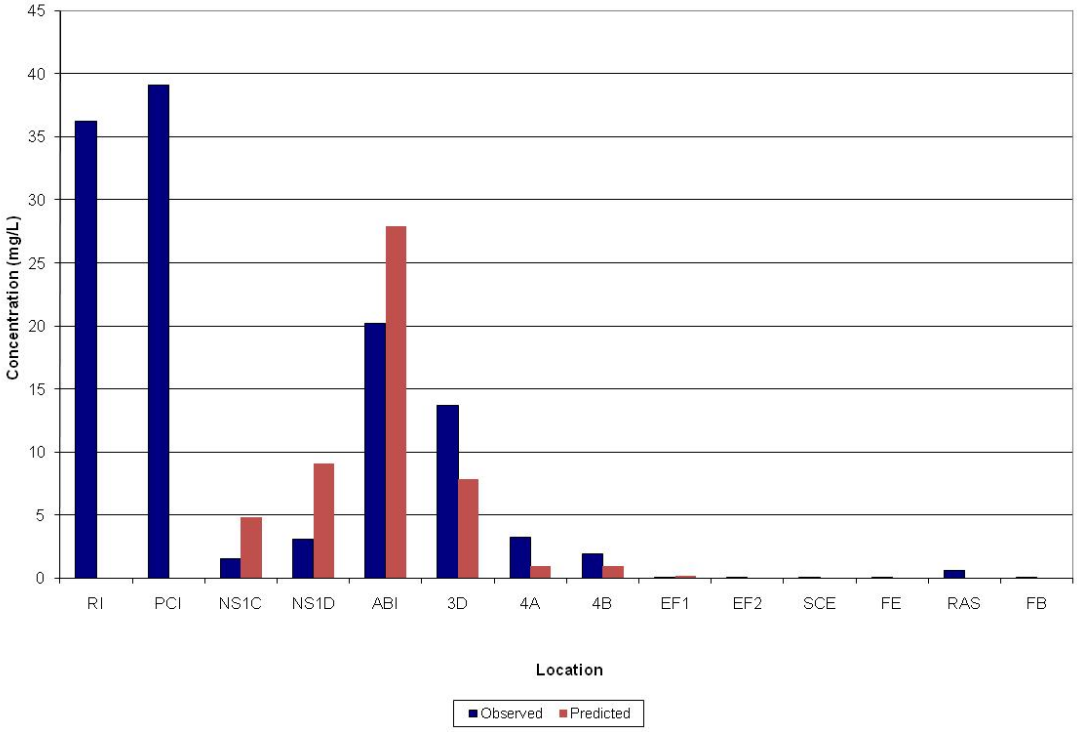


Figure 4-29
Predicted vs. Measured NH_3 Values – Train C – Day 2 AM

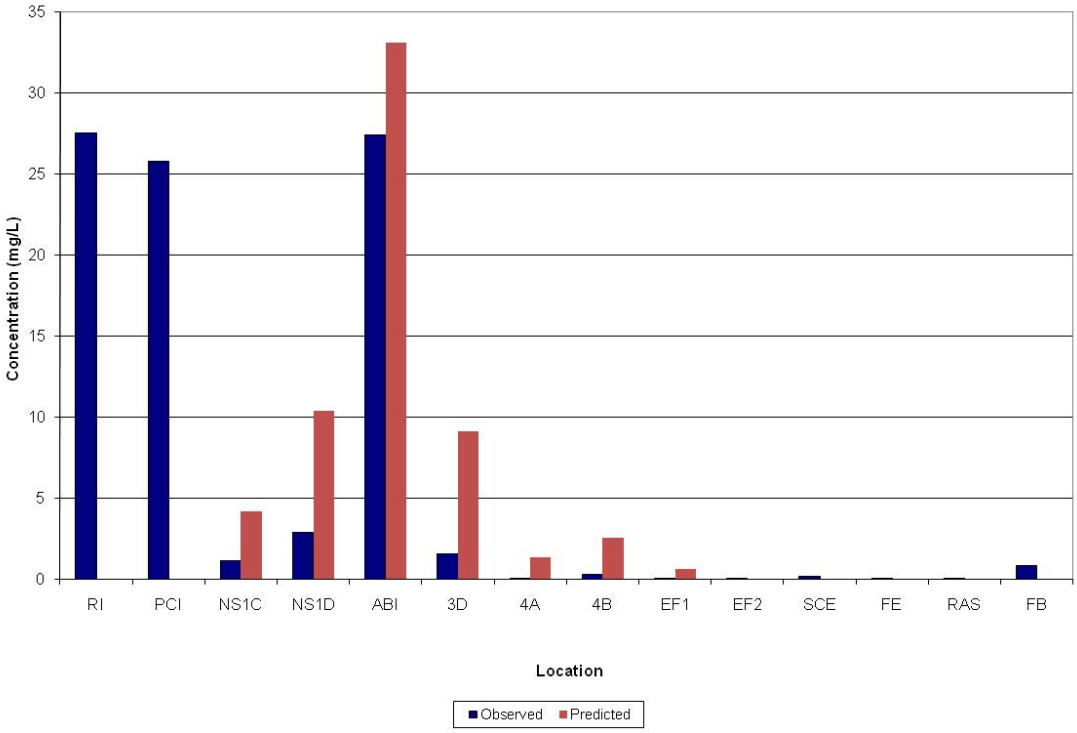


Figure 4-30
Predicted vs. Measured NH_3 Values – Train C – Day 2 PM

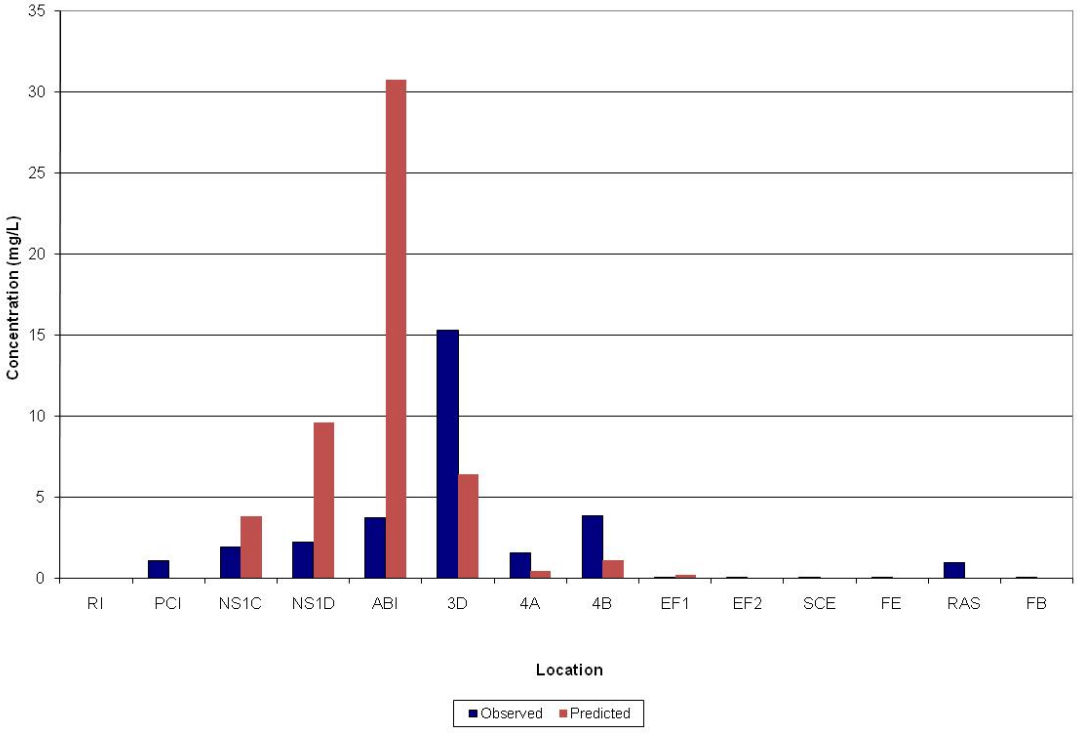


Figure 4-31
Predicted vs. Measured NO_3 Values – Train C – Day 1 AM

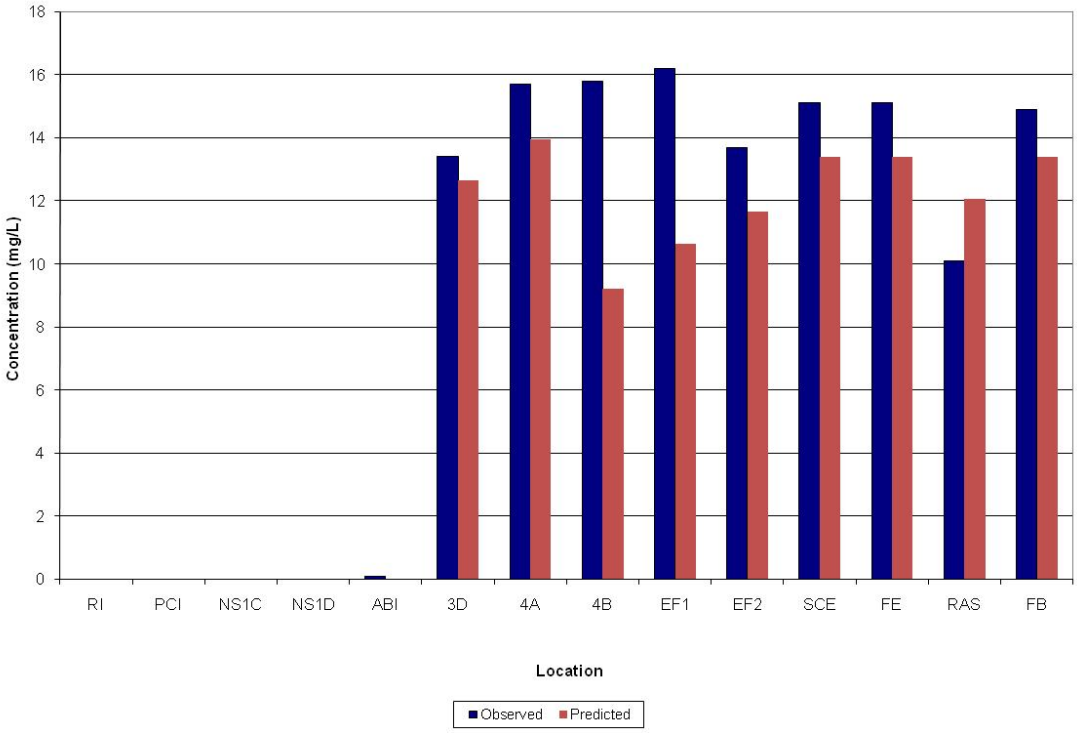


Figure 4-32
Predicted vs. Measured NO_3 Values – Train C – Day 1 PM

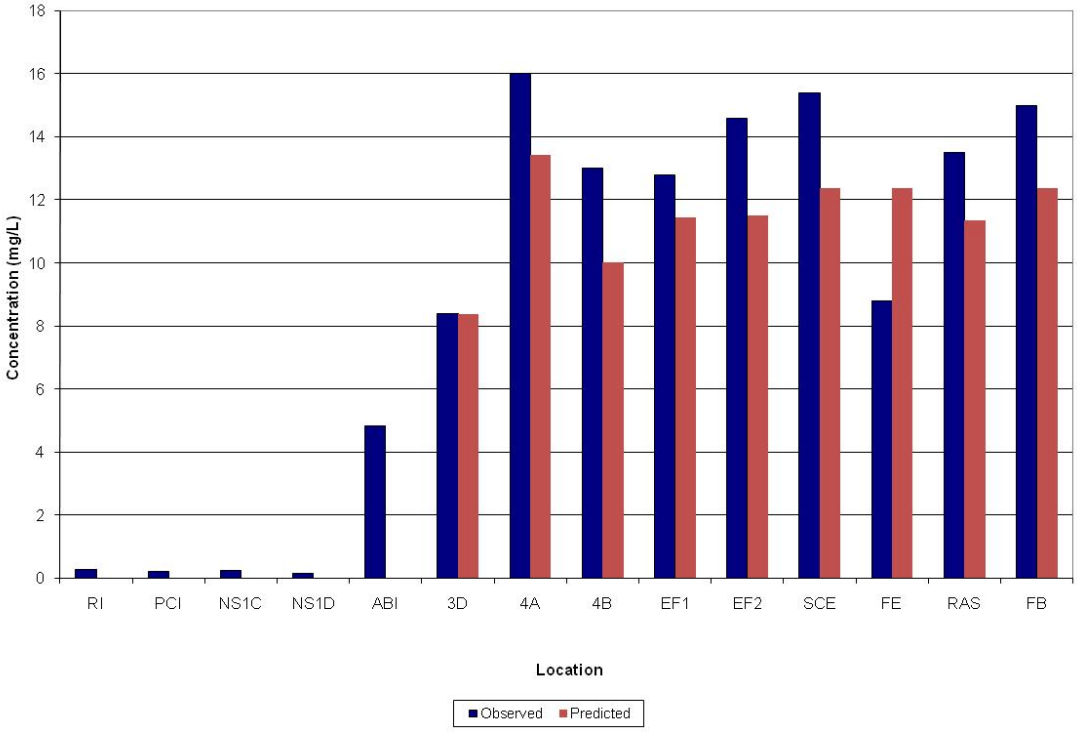


Figure 4-33
Predicted vs. Measured NO_3 Values – Train C – Day 2 AM

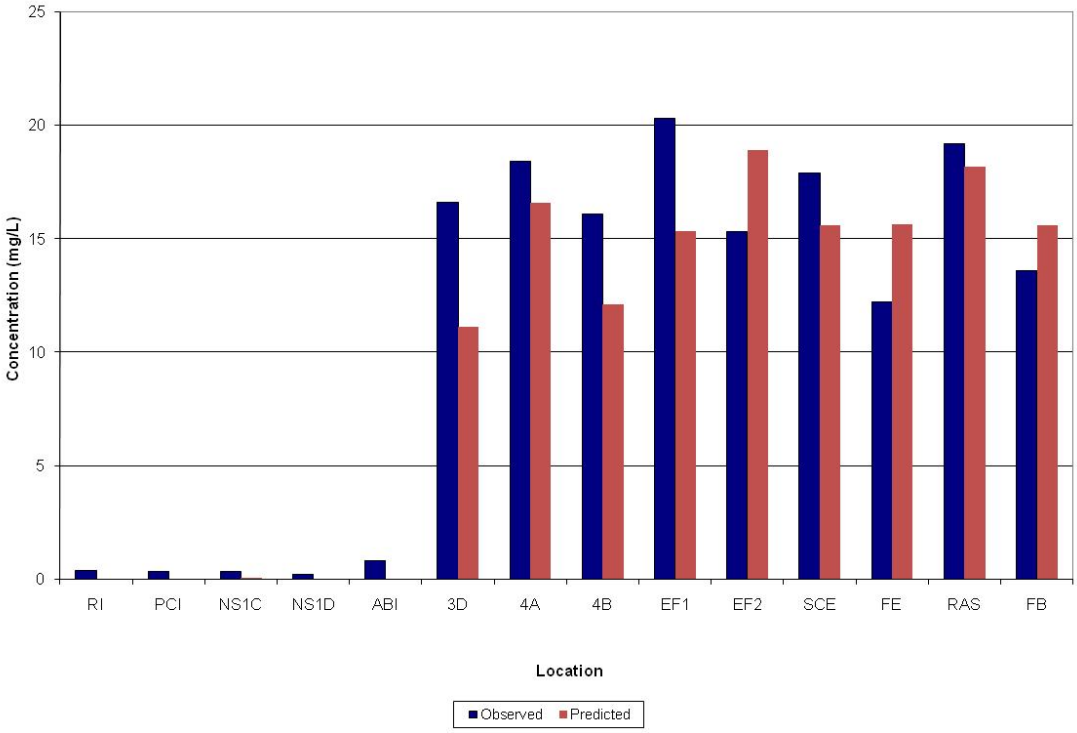


Figure 4-34
Predicted vs. Measured NO_3 Values – Train C – Day 2 PM

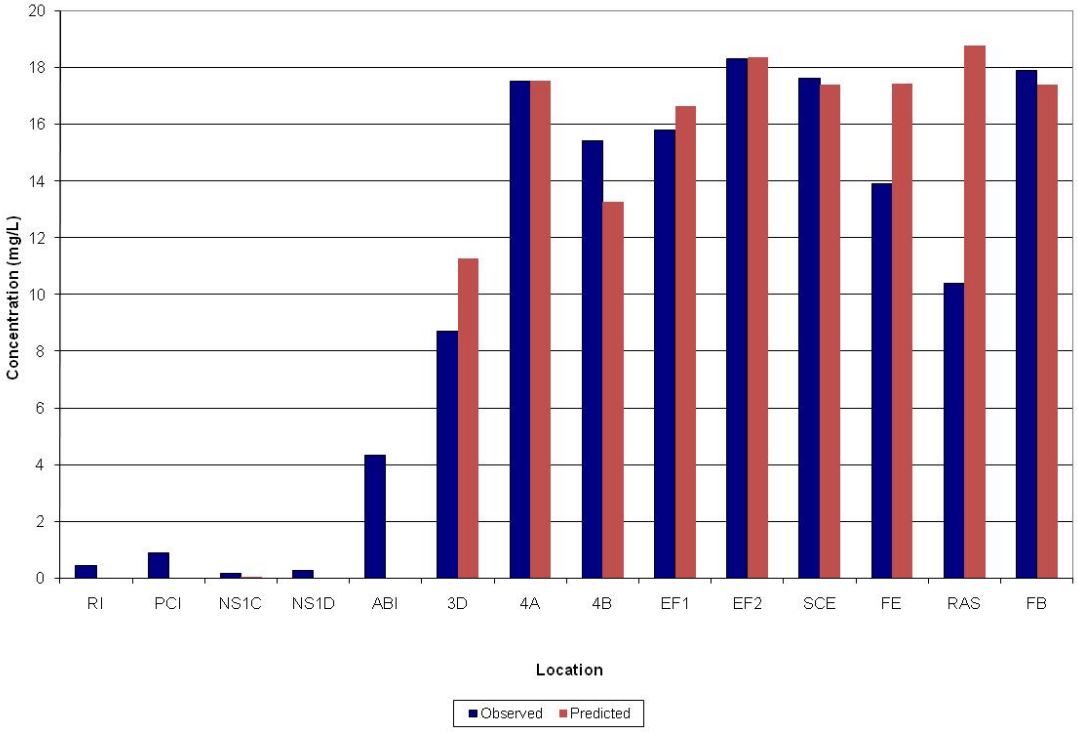


Figure 4-35
Predicted vs. Measured PO_4-P Values – Train C – Day 1 AM

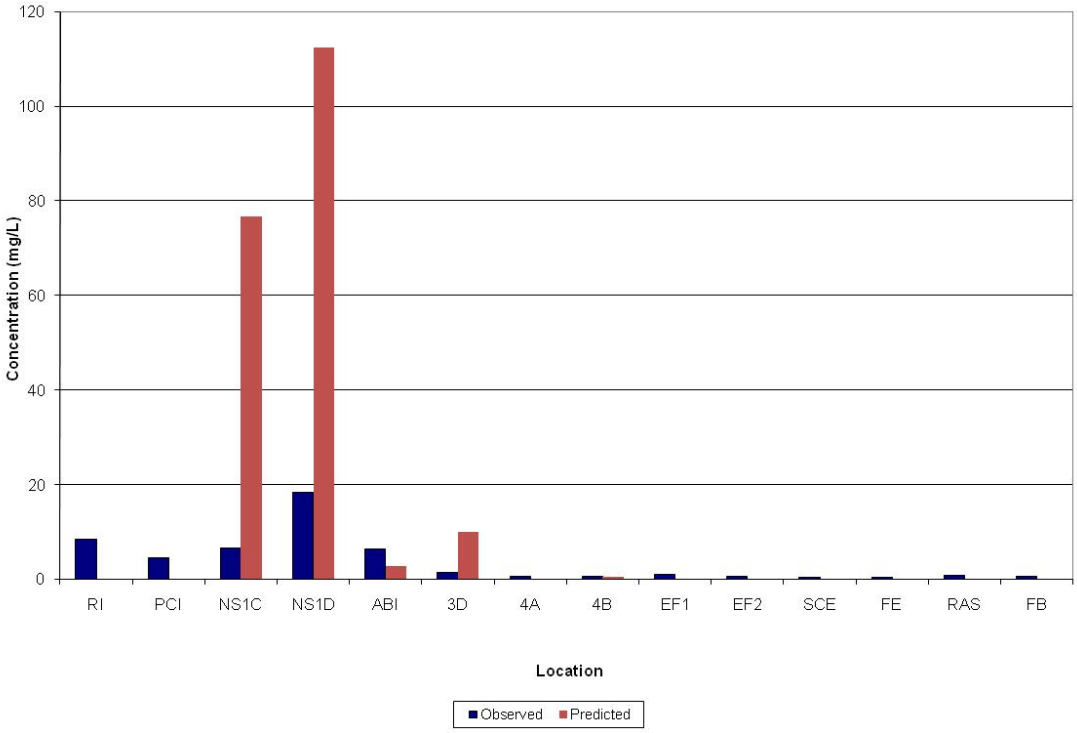


Figure 4-36
Predicted vs. Measured PO_4-P Values – Train C – Day 1 PM

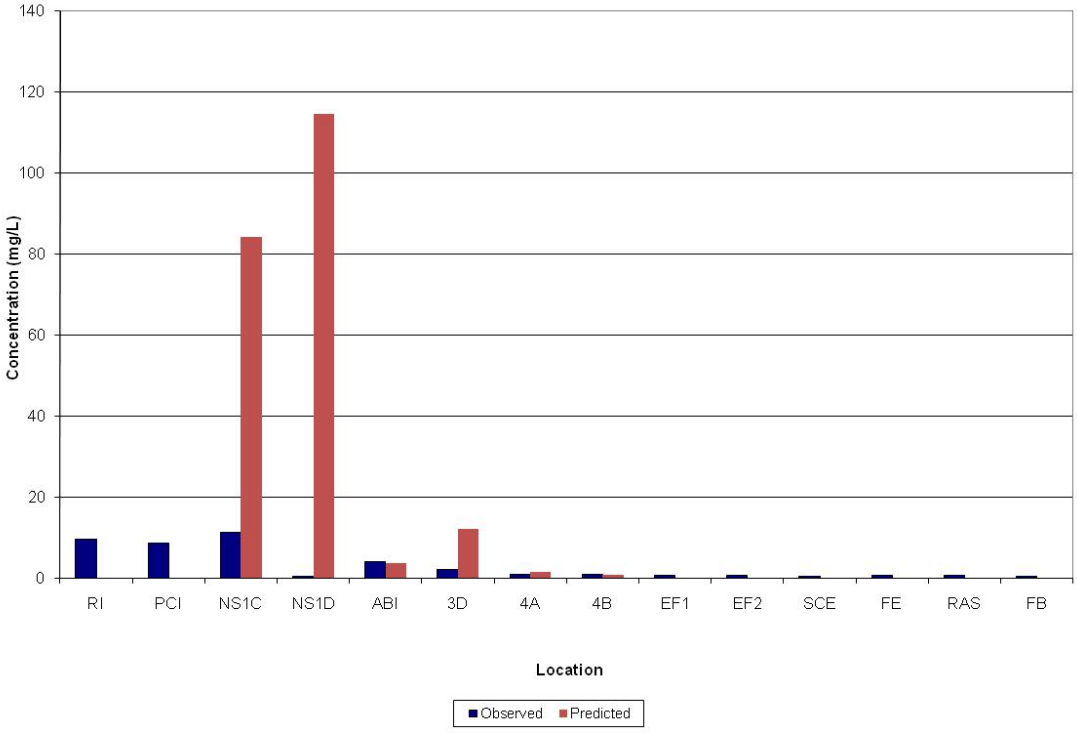


Figure 4-37
Predicted vs. Measured PO_4-P Values – Train C – Day 2 AM

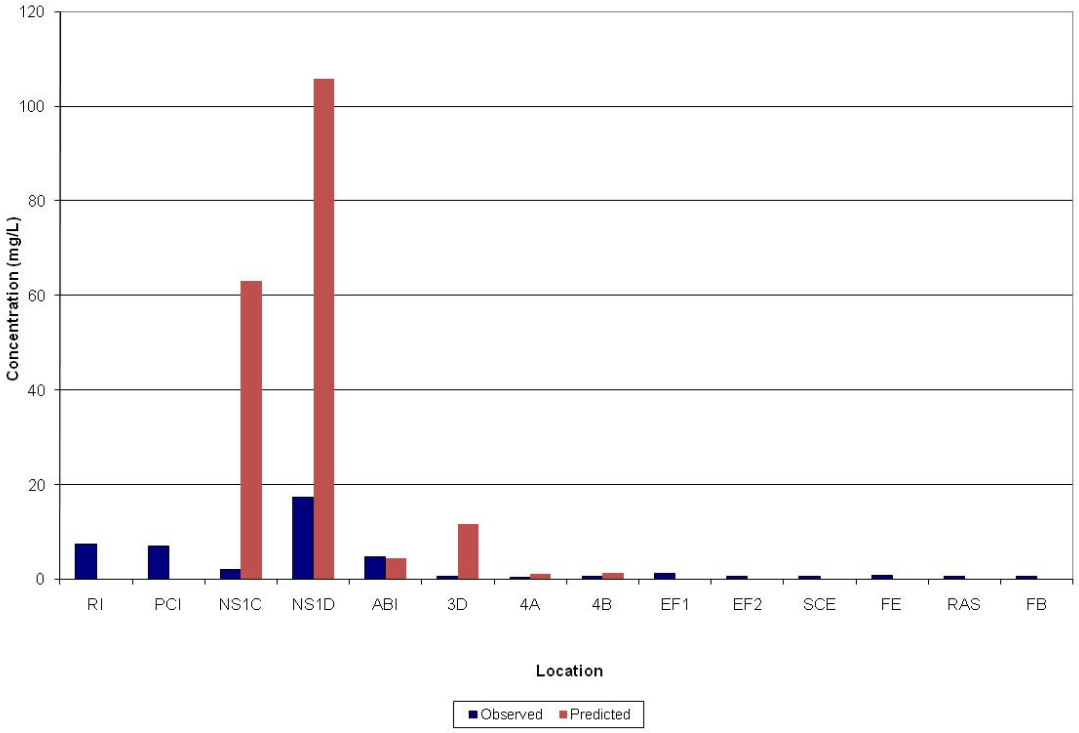
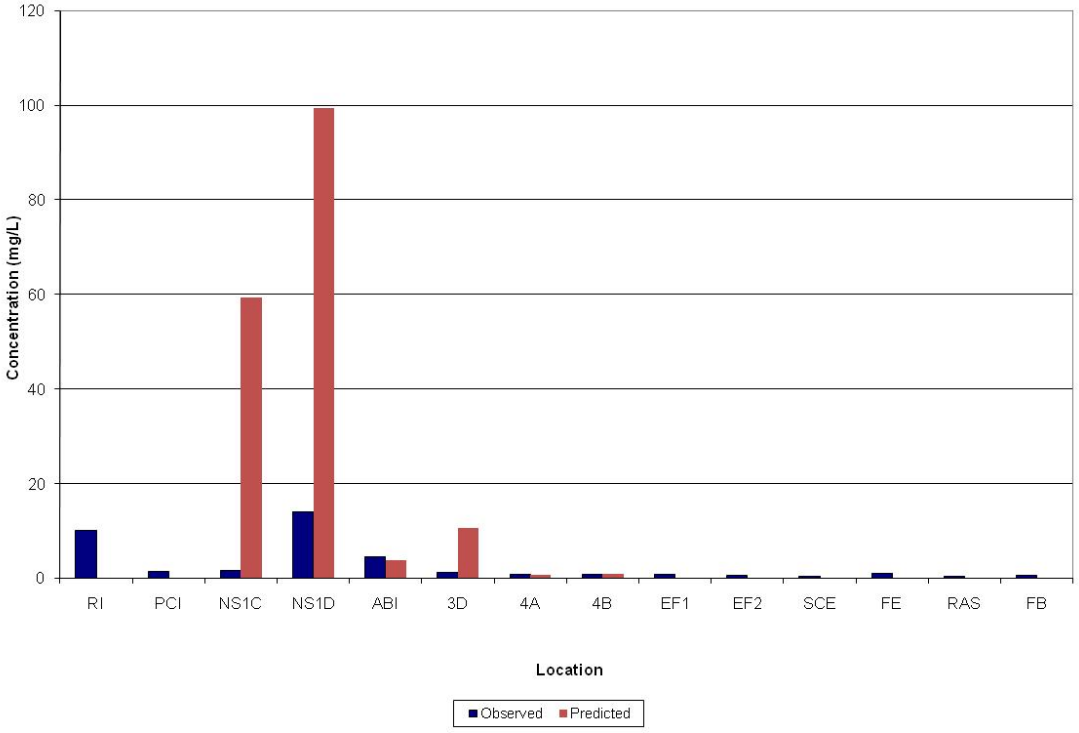


Figure 4-38
Predicted vs. Measured TP Values – Train C – Day 2 PM



4.6 Short-Term Calibration – Plant Effluent Comparison

Figure 4-39 through 4-45 present comparisons of simulated and modeled values of plant effluent parameters. The simulated values match reasonably well with the exception of total phosphorus, were predicted values were lower than observed even without chemical phosphorus removal modeling.

Figure 4-39
Predicted vs. Measured Plant Effluent NH_3 Values

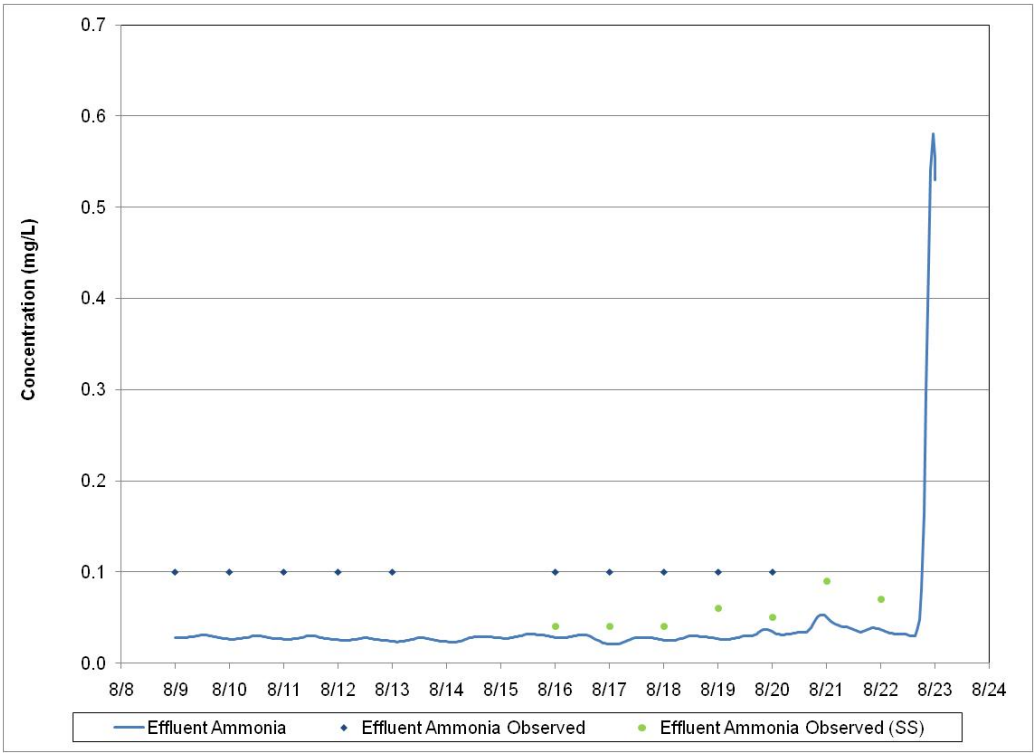


Figure 4-40
Predicted vs. Measured Plant Effluent TKN Values

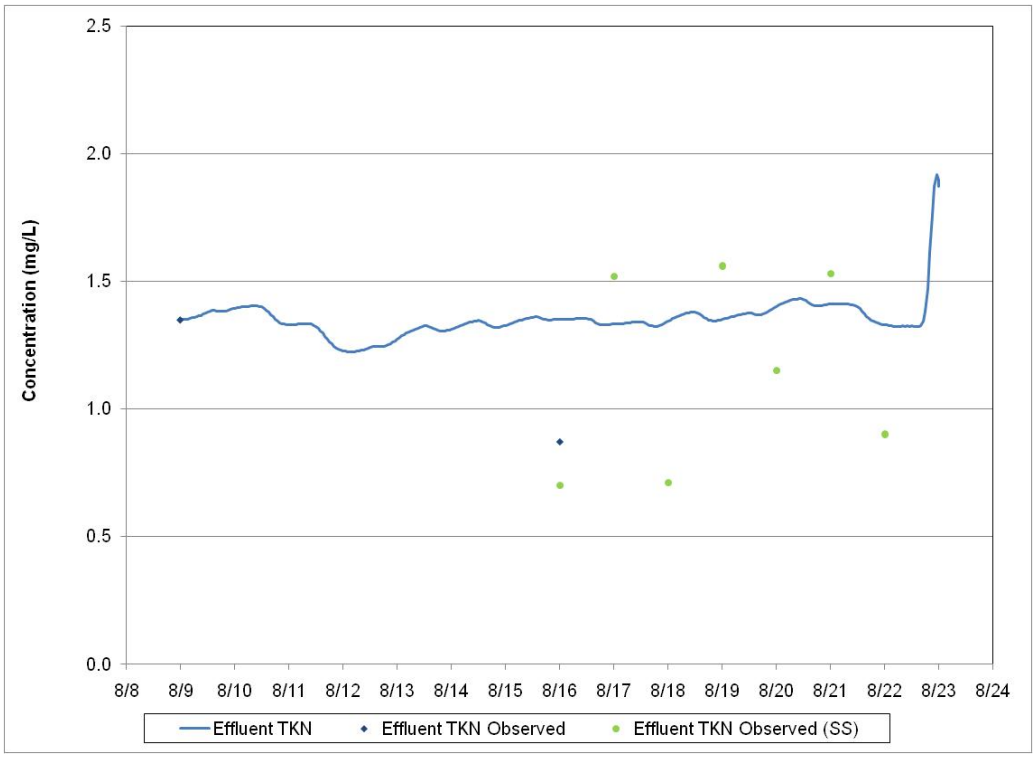


Figure 4-41
Predicted vs. Measured Plant Effluent NO_x Values

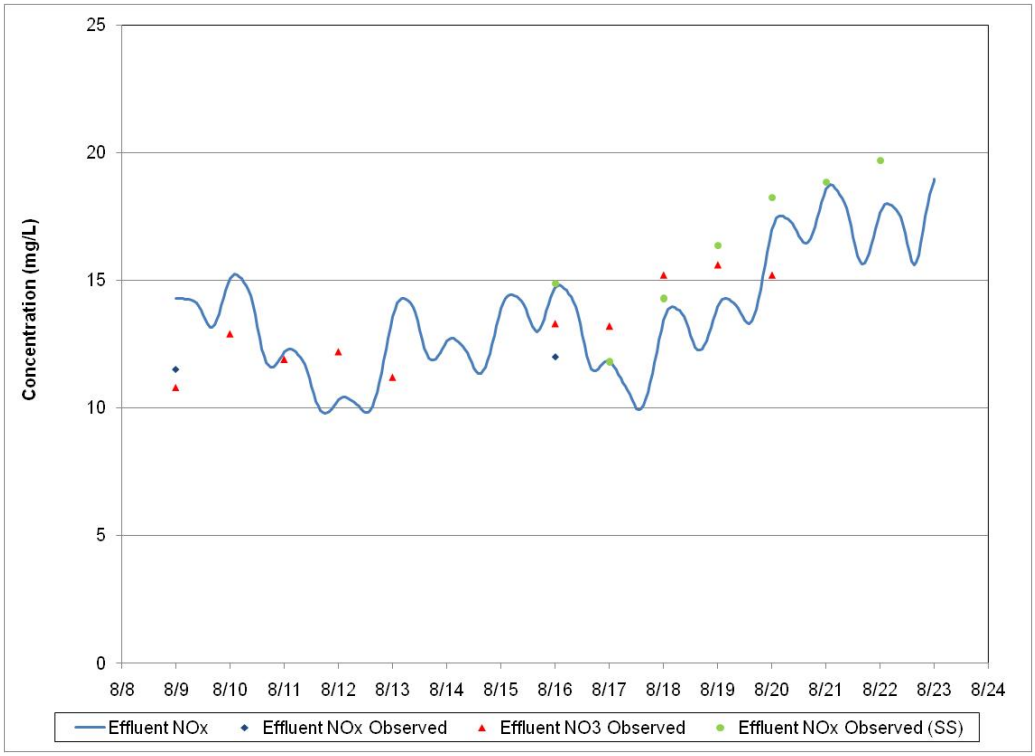


Figure 4-42
Predicted vs. Measured Plant Effluent TN Values

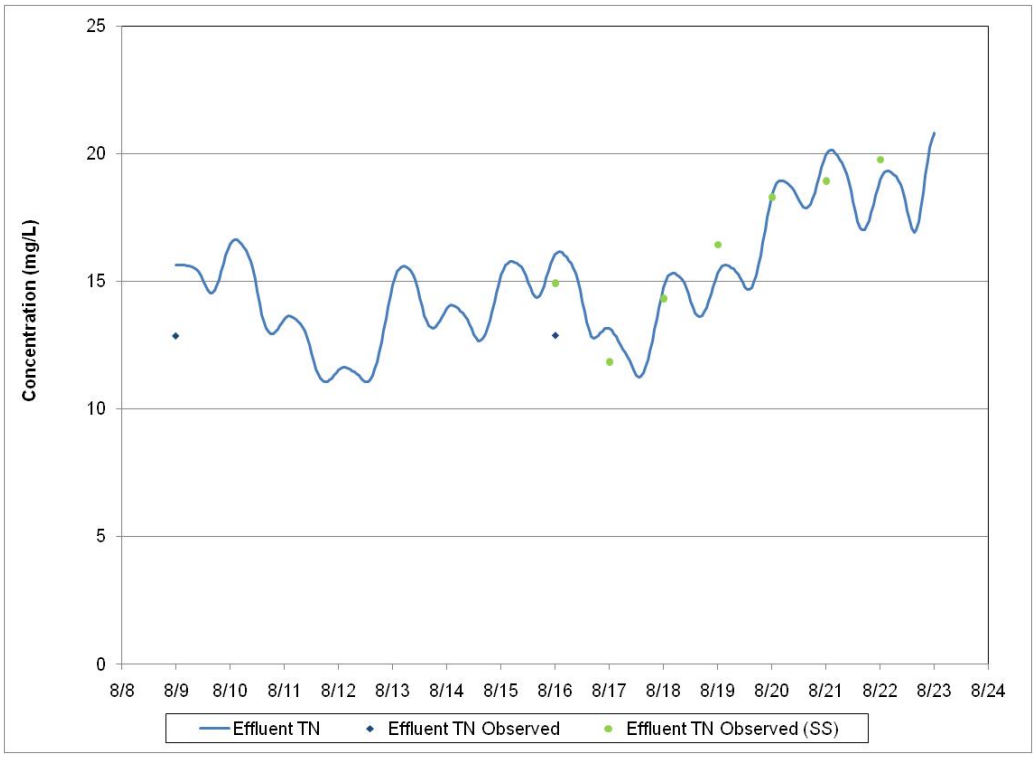


Figure 4-43
Predicted vs. Measured Plant Effluent TP Values

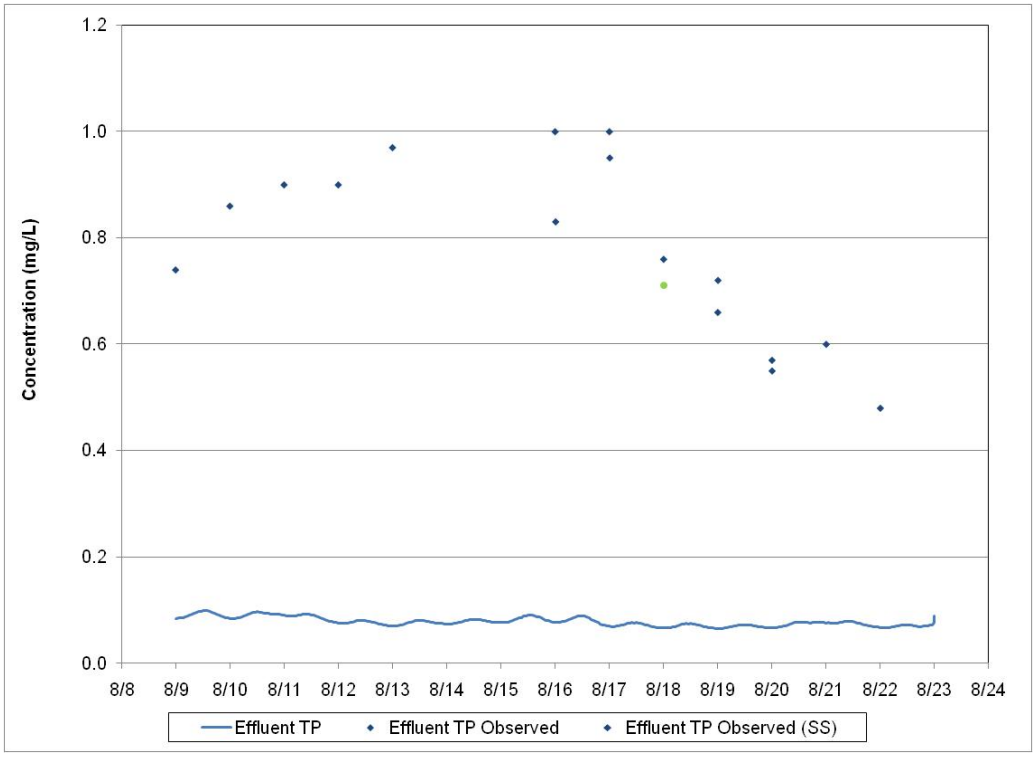


Figure 4-44
Predicted vs. Measured Plant Effluent cBOD Values

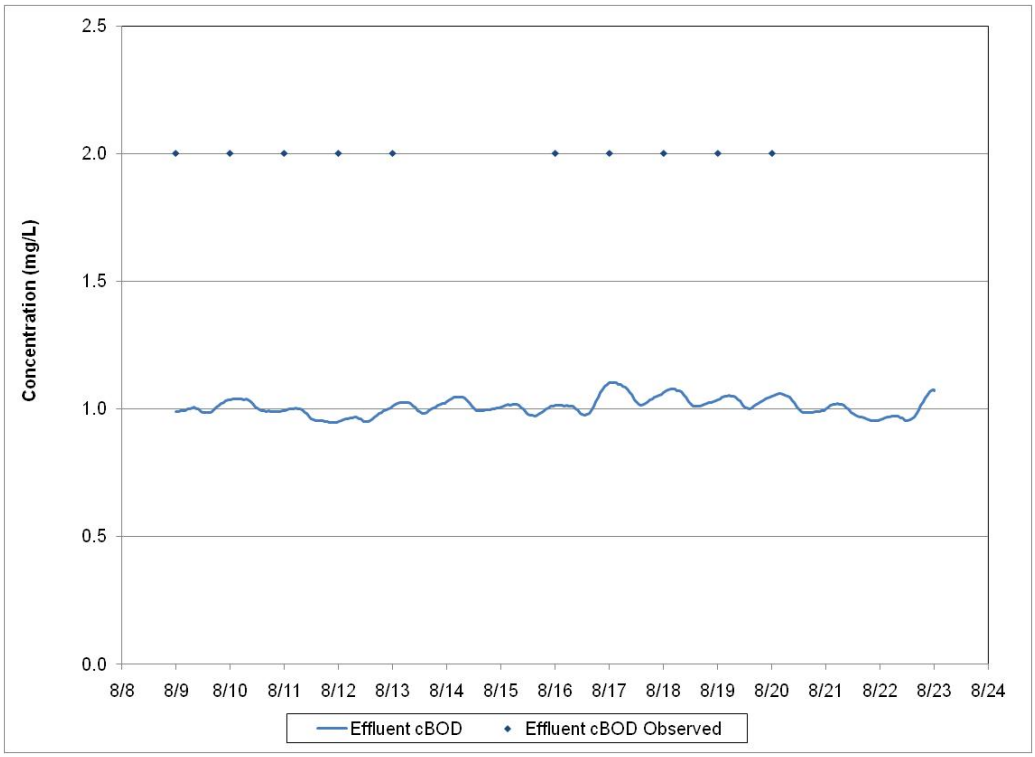
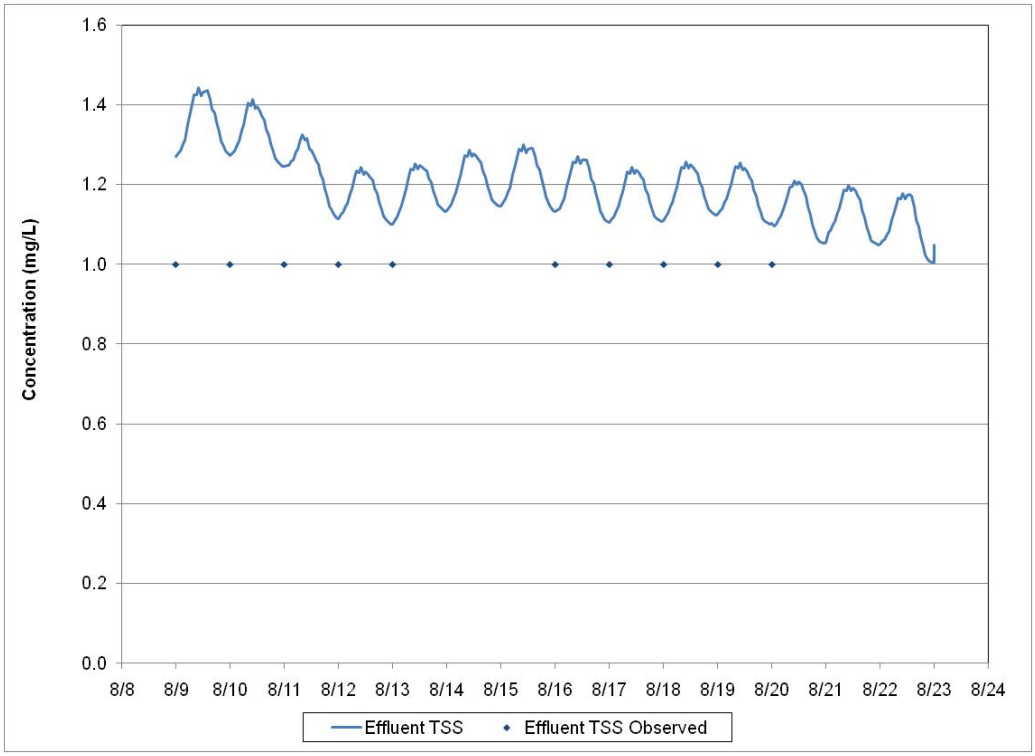


Figure 4-45
Predicted vs. Measured Plant Effluent TSS Values



PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Joe Rohrbacher, Paul Pitt

DATE: October 26, 2009

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum 3-C – Process Simulation Results

1. INTRODUCTION

A calibrated process model of the Mason Farm WWTP was developed using BioWin 3.01 (Envirosim, Ltd.) process simulation software. The model was calibrated based on historical data and the results of special sampling performed in August 2009. The results of the special sampling are summarized in Technical Memorandum No. 3-A, and process model calibration is detailed in Technical Memorandum No. 3-B. These memoranda are appended to this Technical Memorandum.

The process model was used to assess the capacity of the existing activated sludge process to treat the Year 2030 projected flows and loads. In addition simulations were performed to estimate plant performance under maximum month conditions at the current 14.5 mgd permitted design flow. The results of the process modeling were used to establish a load-based approach to secondary treatment capacity determination. The establishment of a maximum load that the existing facility can treat will assist OWASA in determining the timing of future improvements and expansions despite changing influent wastewater characteristics.

2. INFLUENT DESIGN CRITERIA

Projected wastewater flows and loads were developed in Technical Memorandum No. 2. Design influent criteria are provided for two scenarios; current wastewater strength and increased wastewater strength due to water conservation, increased reclaimed water usage and reduced inflow/infiltration. The purpose of providing these two sets of characteristics is to bracket the potential range of wastewater loadings to the plant through 2030. In addition both

the expected demand and higher demand flow projections will be evaluated to provide a range of potential plant loadings in the year 2030 and determine when expansion of capacity and/or improvements to the Mason Farm WWTP can be expected. Tables 1 and 2 present the design influent criteria at 2030 under annual average and maximum month conditions, respectively.

Table 1

Summary of Design Influent Criteria at 2030 – Annual Average Conditions

	Flow (mgd)	Temp. (°C)	BOD_5 (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
Expected Flow, Current WW Strength	10.4	20	210	270	30	40	7.1
Expected Flow, Increased WW Strength	10.4	20	274	365	39	53	9.9
Higher Flow, Current WW Strength	11.3	20	210	270	30	40	7.1
Higher Flow, Increased WW Strength	11.3	20	274	365	39	53	9.9

Table 2

Summary of Design Influent Criteria at 2030 – Maximum Month Conditions

	Flow (mgd)	Temp. (°C)	BOD_5 (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
Expected Flow, Current WW Strength	13.0	13	218	324	31	42	7.4
Expected Flow, Increased WW Strength	13.0	13	285	438	41	55	10.3
Higher Flow, Current WW Strength	14.1	13	218	324	31	42	7.4
Higher Flow, Increased WW Strength	14.1	13	285	438	41	55	10.3

Process simulations were also performed for the current 14.5 mgd design flow condition. Design influent criteria for annual average and maximum month conditions are provided in Table 3.

Table 3

Summary of Design Influent Criteria at 14.5 mgd Design Flow

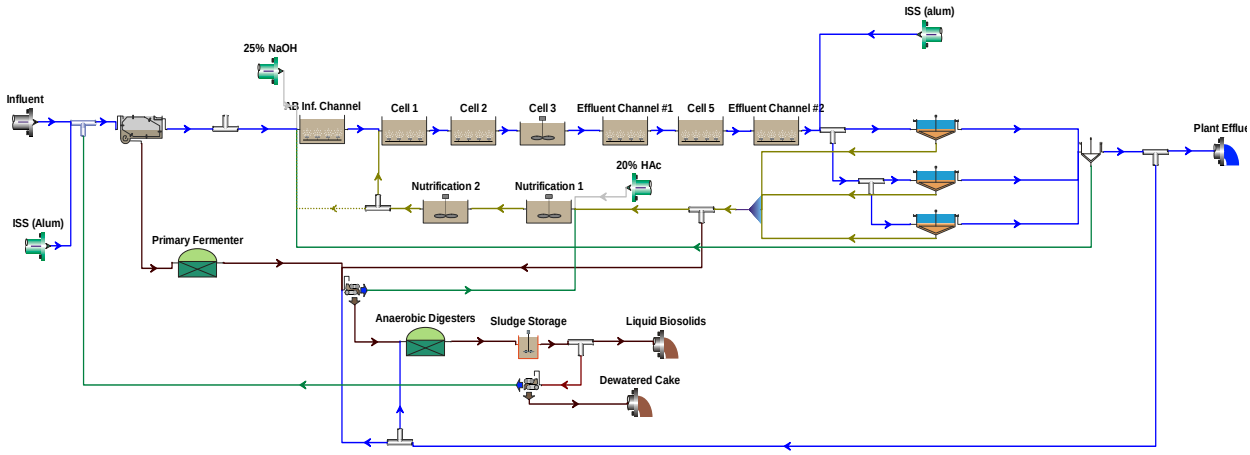
	Flow (mgd)	Temp. (°C)	BOD_5 (mg/L)	TSS (mg/L)	NH3-N (mg/L)	TKN (mg/L)	TP (mg/L)
Annual Average, Current WW Strength	11.6	20	210	270	30	40	7.1
Annual Average, Increased WW Strength	11.6	20	274	365	39	53	9.9
Maximum Month, Current WW Strength	14.5	13	218	324	31	42	7.4
Maximum Month, Increased WW Strength	14.5	13	285	438	41	55	10.3

3. MODEL CONFIGURATION

Figure 1 provides a schematic of the BioWin model used in the process evaluation. The model was setup based on the current 4:3 mode of operation and assumed that all aeration basin and NSL cells were in service. Simulations of annual average conditions assumed anoxic operation of Cell 3 for additional denitrification, as shown in Figure 1. Maximum month simulations assumed operation of all aeration cells aerobically to maintain nitrification under cold weather conditions. The RAS flow was set at 50% of the influent flow to match typical operations. The primary clarifier performance was set at a solids removal rate of 60% to match historical performance. The modeled flow to the secondary clarifiers is based on the predicted flow split summarized in Table 3-15 of Technical Memorandum No. 3.

Figure 1

Process Evaluation BioWin Schematic



Sodium hydroxide feed was adjusted to maintain a pH of 6.6 in Effluent Channel No. 2. Acetic acid (20% solution) was fed at a rate of 100 gallons per day (gpd) per mgd influent flow. Alum requirements were calculated independently of the model to expedite simulation times. An inert suspended solids (ISS) input was entered in the model to simulate the increase in solids production associated with chemical phosphorus removal.

4. SIMULATION RESULTS

Table 4 summarizes the results of the process simulations performed under annual average conditions. The annual average process simulations indicate that full nitrification would be achieved at an aerobic solids retention time (SRT) of 5 days. Average MLSS concentrations assuming current wastewater strength are approximately 3,000 mg/L, which is typical of nutrient removal processes. The process simulations indicated that chemical phosphorus removal would be required to reduce effluent TP concentrations to 0.2 mg/L. Total nitrogen concentrations under this scenario would still be well above the required effluent criteria to meet the current and future TN mass allocations. Substantial denitrification would be required across the existing denitrification filters to meet the mass allocations.

Table 4

Process Simulation Results – Annual Average Conditions

Parameter	Expected Flow Current WW Strength	Expected Flow Increased WW Strength	Higher Flow Current WW Strength	Higher Flow Increased WW Strength
Flow, mgd	10.4	10.4	11.3	11.3
Temperature, C	20	20	20	20
Aerobic SRT, days	5	5	5	5
MLSS Conc., mg/L	2,900	3,700	3,100	4,000
Primary Sludge, lb/d	15,200	20,000	16,600	21,600
WAS Production, lb/d	13,500	17,500	14,700	19,000
RAS Flow, mgd	5.2	5.2	5.65	5.65
Effluent NH3-N, mg/L	< 0.5	< 0.5	< 0.5	< 0.5
Effluent NOx-N, mg/L	16.5	22.0	16.5	22.0
Effluent TKN, mg/L	1.5	1.7	1.5	1.7
Effluent TN, mg/L[1]	18.0	23.7	18.0	23.7
Effluent TP, mg/L[2]	<0.2	<0.2	<0.2	<0.2
Effluent $cBOD_5$, mg/L	< 4	< 4	< 4	< 4
Effluent TSS, mg/L	< 3	< 3	< 3	< 3
25% NaOH Feed, gpd	2,000	4,000	2,100	4,400
20% HAc Feed, gpd	1,000	1,000	1,100	1,100
48% Alum Feed, gpd	800	1,200	900	1,300

[1]Post-filtration assuming no denitrification within filters

[2]Post-filtration assuming chemical precipitation

Table 5 presents the results of the maximum month process simulations. The process simulations indicate that full nitrification would be achieved at an aerobic SRT of 7 days under maximum month conditions. MLSS concentrations at current wastewater strength are at the high end of typical activated sludge design and may lead to clarification issues during high flows. The MLSS concentrations at the increased wastewater strength are excessive, and additional capacity will be required if influent wastewater characteristics strengthen prior to Year 2030.

Table 5

Process Simulation Results – Maximum Month Conditions

Parameter	Expected Flow Current WW Strength	Expected Flow Increased WW Strength	Higher Flow Current WW Strength	Higher Flow Increased WW Strength
Flow, mgd	13.0	13.0	14.1	14.1
Temperature, C	13	13	13	13
Aerobic SRT, days	7	7	7	7
MLSS Conc., mg/L	4,100	5,460	4,500	5,900
Primary Sludge, lb/d	22,000	29,400	23,800	31,800
WAS Production, lb/d	19,500	25,800	21,200	28,000
RAS Flow, mgd	6.5	6.5	7.05	7.05
Effluent NH3-N, mg/L	< 0.5	< 0.5	< 0.5	< 0.5
Effluent NOx-N, mg/L	20.8	27.2	20.8	27.2
Effluent TKN, mg/L	1.7	1.9	1.7	1.9
Effluent TN, mg/L[1]	22.5	29.0	22.5	29.1
Effluent TP, mg/L[2]	0.2	0.2	0.2	0.2
Effluent $cBOD_5$, mg/L	< 4	< 4	< 4	< 4
Effluent TSS, mg/L	< 3	<3	< 3	<3
25% NaOH Feed, gpd	4,000	6,500	4,000	7,000
20% HAc Feed, gpd	1,300	1,300	1,400	1,400
48% Alum Feed, gpd	1,100	1,500	1,200	1,600

[1]Post-filtration assuming no denitrification within filters

[2]Post-filtration assuming chemical precipitation

Table 6 presents the process simulation results at the 14.5 mgd design flow. MLSS solids concentrations may be excessive under current wastewater conditions depending on clarifier performance. The Mason Farm WWTP would have insufficient aeration basin capacity to treat a maximum month flow of 14.5 mgd if influent wastewater characteristics strengthened appreciably from current values.

Table 6

Process Simulation Results – 14.5 mgd Design Flow

Parameter	Annual Avg. Current WW Strength	Annual Avg. Increased WW Strength	Max. Month Current WW Strength	Max. Month Increased WW Strength
Flow, mgd	11.6	11.6	14.5	14.5
Temperature, C	20	20	13	13
Aerobic SRT, days	5	5	7	7
MLSS Conc., mg/L	3,200	4,100	4,600	6,100
Primary Sludge, lb/d	17,000	22,100	24,500	32,600
WAS Production, lb/d	15,100	19,600	21,800	28,700
RAS Flow, mgd	5.8	5.8	7.25	7.25
Effluent NH3-N, mg/L	< 0.5	< 0.5	< 0.5	< 0.5
Effluent NOx-N, mg/L	16.4	22.0	20.8	27.2
Effluent TKN, mg/L	1.5	1.7	1.7	1.9
Effluent TN, mg/L[1]	17.9	23.7	22.5	29.1
Effluent TP, mg/L[2]	0.2	0.2	0.2	0.2
Effluent $cBOD_5$, mg/L	< 4	< 4	< 4	< 4
Effluent TSS, mg/L	< 3	< 3	3	<3
25% NaOH Feed, gpd	2,200	4,500	4,000	7,000
20% HAc Feed, gpd	1,200	1,200	1,500	1,500
48% Alum Feed, gpd	900	1,300	1,200	1,600

[1]Post-filtration assuming no denitrification within filters

[2]Post-filtration assuming chemical precipitation

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Christopher White, Alonso Griborio, Robert Harris

DATE: October 26, 2009

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum 3-D – Clarifier Stress Testing and 2Dc Model Calibration Data

1. INTRODUCTION

Secondary clarifier performance at simulated projected peak design conditions was evaluated on August 11, 13, 25, and 27 (to be referred to as Day 1, Day 2, Day 3, and Day 4, respectively). The data obtained was used for the setup, calibration, and validation of a 2Dc Computational Fluid Dynamics (CFD) model. Because the Mason Farm WWTP (MFWWTP) employs five clarifiers of four different configurations, independent stress tests were conducted for each clarifier configuration. To simulate peak flow conditions, clarifiers were taken out of service and flow was directed to the unit(s) remaining in service. Suspended solids samples were obtained throughout the stress testing and sludge blankets were closely monitored. Each clarifier stress test continued for at least six hours. In addition to secondary clarifier performance, batch tests were conducted to evaluate the settling properties of the sludge.

2. SAMPLING AND PROCEDURES

Stress testing occurred by sending all aeration basin effluent through the clarifier configuration being evaluated. With the exception of the smaller, similarly configured Clarifiers 2 and 3, only one secondary clarifier was online during each day of stress testing. Suspended solids samples were collected on an hourly basis for mixed liquor, return activated sludge, and secondary effluent. Sludge blanket depth for clarifiers in operation was monitored every 30 minutes. During each day of stress testing, the MLSS's SVI was measured twice and the dispersed suspended solids were measured at four locations through the secondary clarification process step (aeration basin effluent, clarifier influent, center well, and effluent weir). Zone settling and compression rates were also evaluated by measuring the settling velocity of various concentrations of sludge. Sludge concentrations were manipulated by mixing different

proportions of RAS, mixed liquor, and secondary effluent. Sludge concentrations were confirmed by evaluating the suspended solids of each batch test. Flocculation properties were also analyzed by agitating mixed liquor for various lengths of time and allowing it to settle for 30 minutes before obtaining a supernatant sample for suspended solids analysis.

During the stress testing of Clarifier 1, the flow to the clarifier was reduced when the flow spilled from one aeration cell into an empty cell. This decreased the actual flow to Secondary Clarifier No. 1 during the stress testing.

3. COLUMN SETTLING TESTS

Settling column tests were performed on during the stress tests on each clarifier.
Figures 3-1 to 3-5 show the settling curves for Clarifier 1. Table 3-1 shows the data from which these figures were generated.

Figures 3-6 to 3-10 show the settling curves for Clarifier 5. Table 3-2 shows the data from which these figures were generated.

Figures 3-11 to 3-15 show the settling curves for Clarifiers 2 and 3. Table 3-3 shows the data from which these figures were generated.

Figures 3-16 to 3-21 show the settling curves for Clarifier 4. Table 3-4 shows the data from which these figures were generated.

Figure 3-1
Day 1, Clarifier 1 - 100% RAS (TSS = 5,345 mg/L)

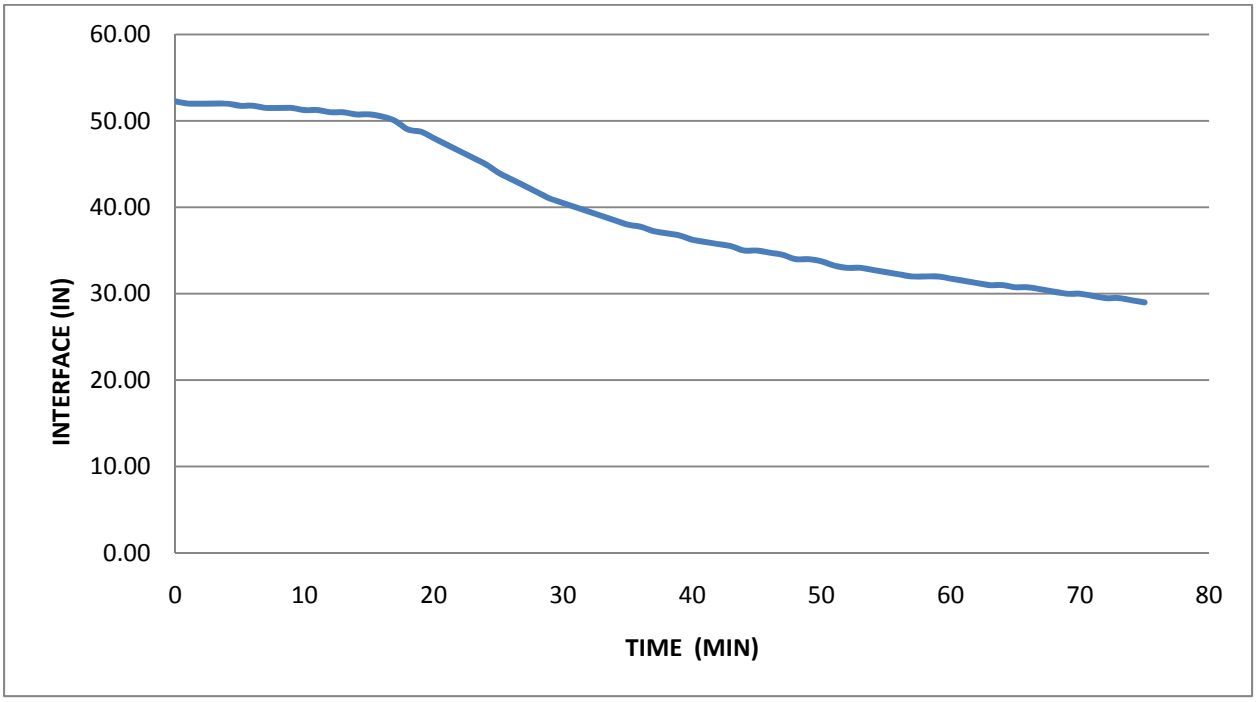


Figure 3-2
Day 1, Clarifier 1 - 100% MLSS (TSS = 2,440 mg/L)

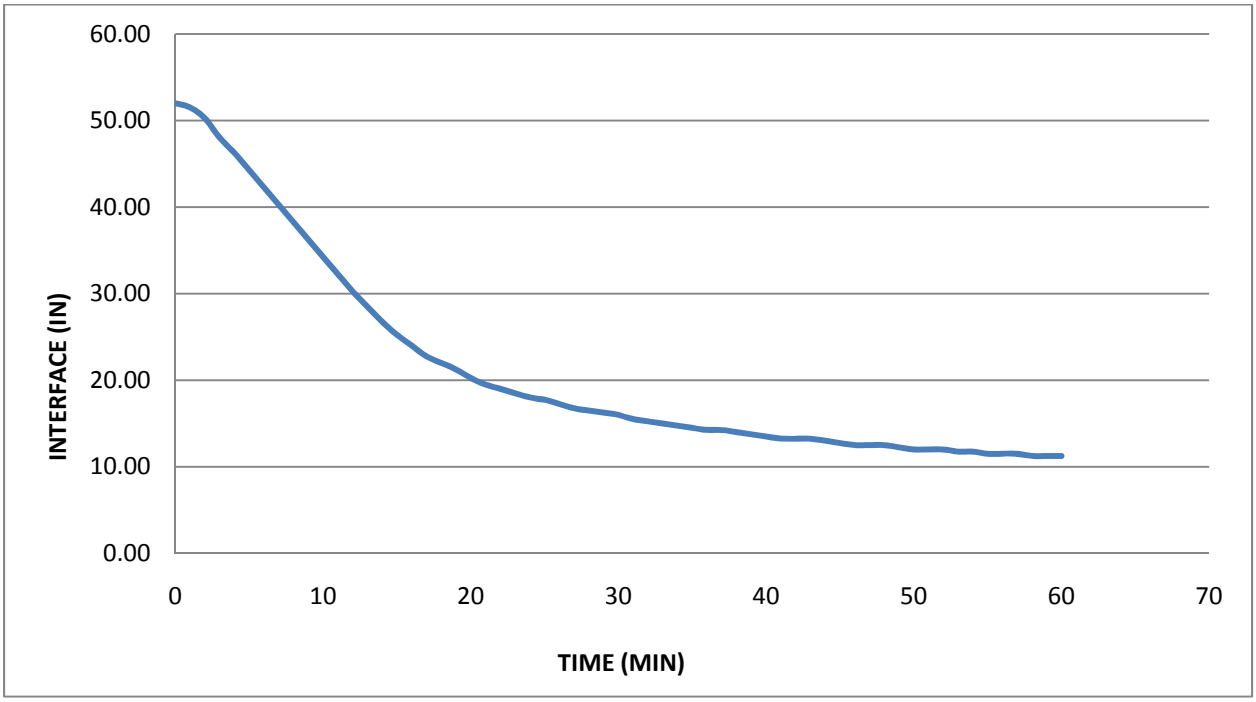


Figure 3-3
Day 1, Clarifier 1 - 66% MLSS, 33% Secondary Effluent (TSS = 1,521 mg/L)

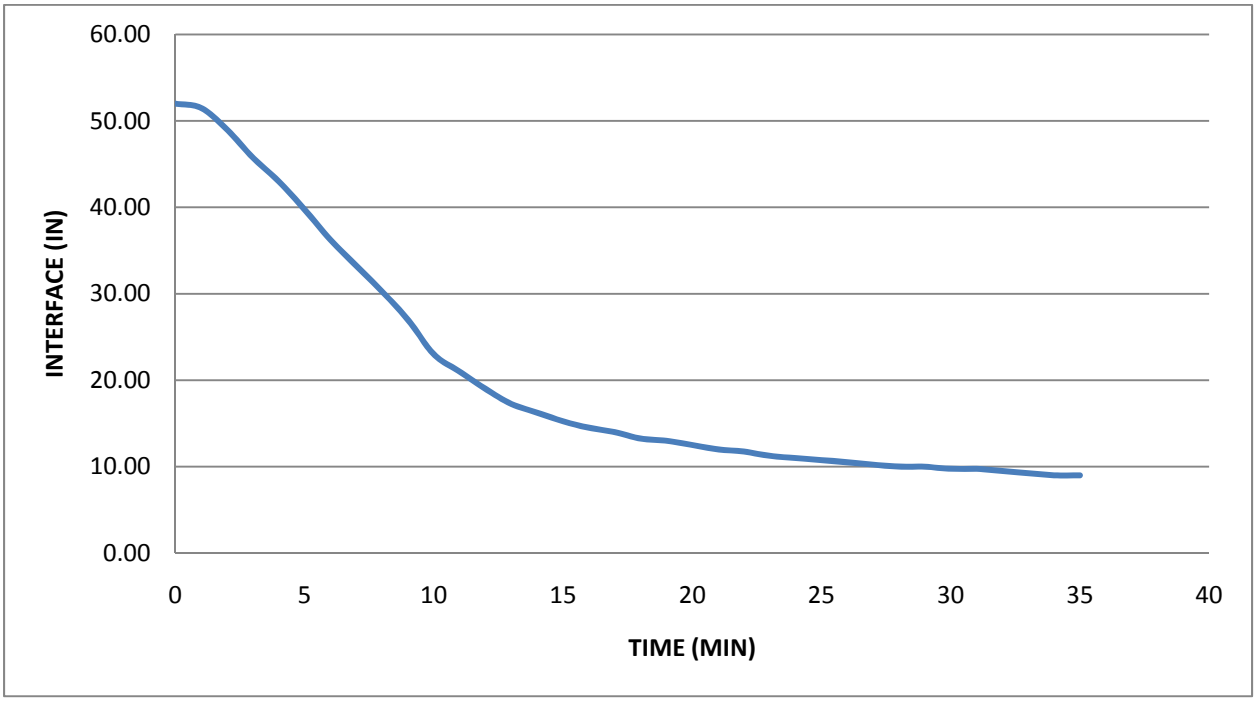


Figure 3-4
Day 1, Clarifier 1 - 33% MLSS, 66% Secondary Effluent (TSS = 770 mg/L)

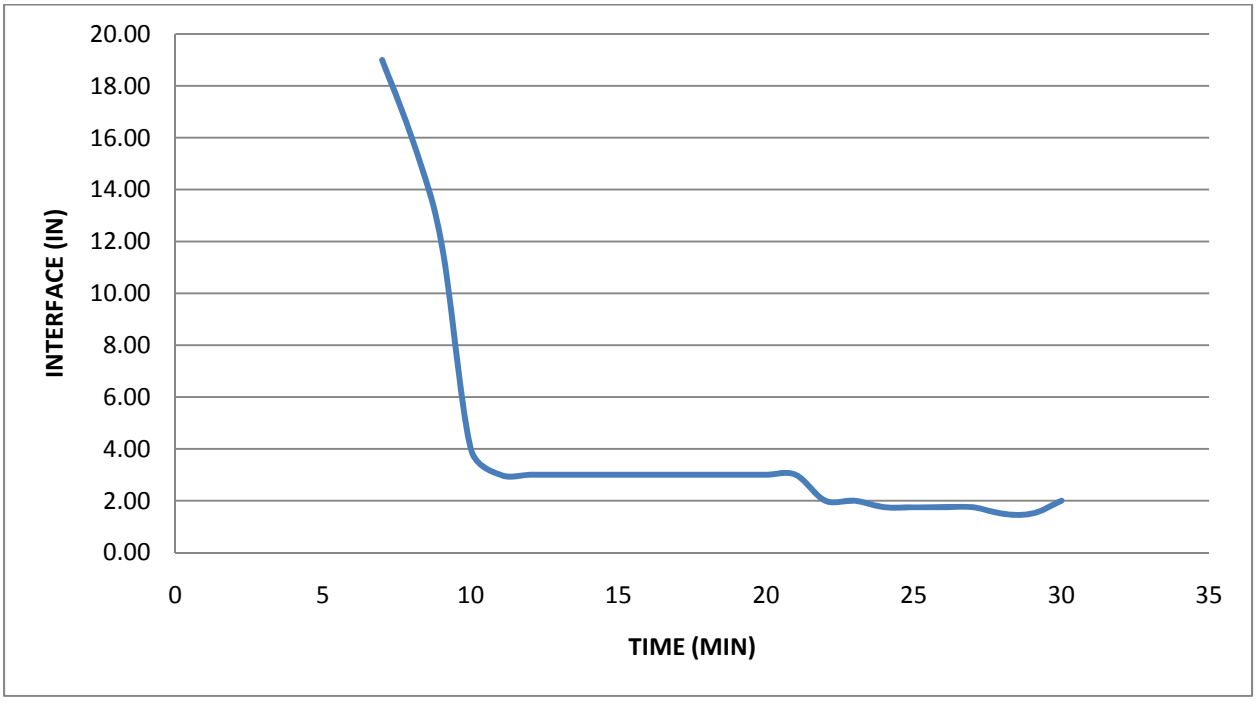


Figure 3-5
Day 1, Clarifier 1 - 50% RAS, 50% MLSS (TSS = 5,640 mg/L)

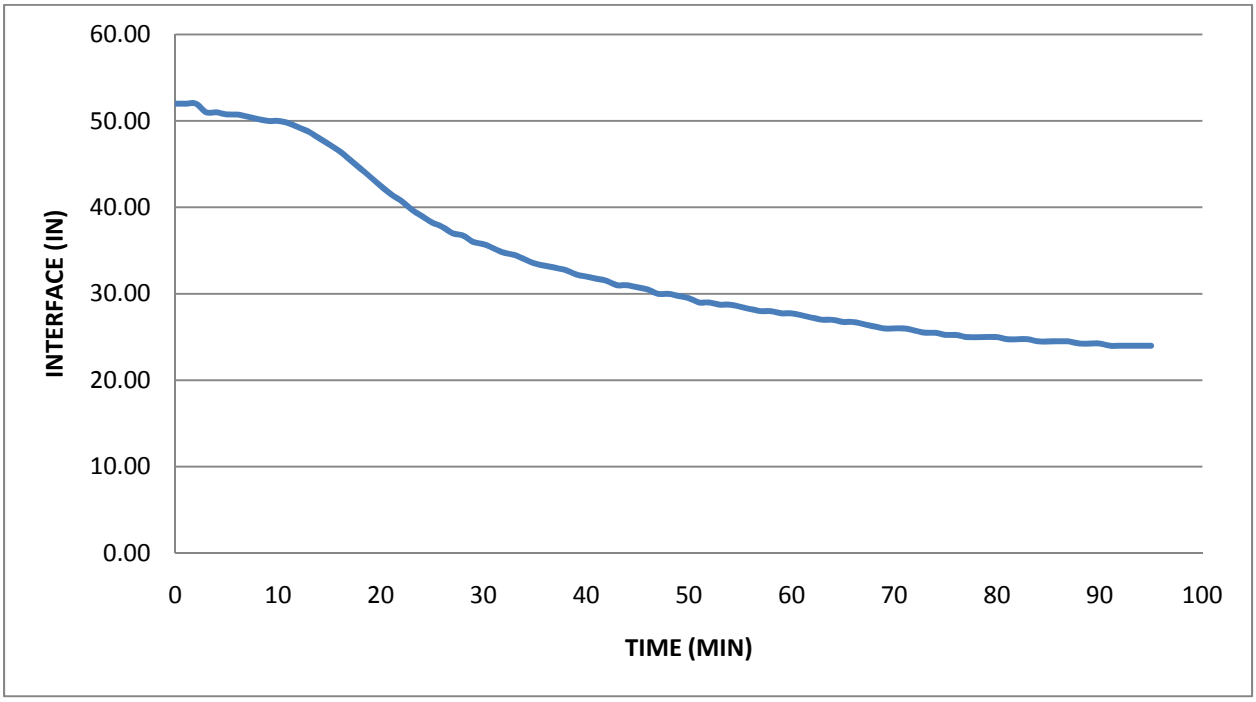


Table 3-1
Day 1 - Clarifier 1 Settling Column Measurements

Time (min)	Clarifier 1 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
0	52.25	52.00	52	HAZY	52
1	52.00	51.50	51.5	HAZY	52
2	52.00	50.25	49	HAZY	52
3	52.00	48.00	45.75	HAZY	51
4	52.00	46.25	43	HAZY	51
5	51.75	44.25	39.75	HAZY	50.75
6	51.75	42.25	36.25	HAZY	50.75
7	51.50	40.25	33.25	18.75	50.5
8	51.50	38.25	30.25	21.75	50.25
9	51.50	36.25	27	25.00	50
10	51.25	34.25	23	29.00	50
11	51.25	32.25	21	31.00	49.75
12	51.00	30.25	19	33.00	49.25
13	51.00	28.50	17.25	34.75	48.75

Time (min)	Clarifier 1 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
14	50.75	26.75	16.25	35.75	48
15	50.75	25.25	15.25	36.75	47.25
16	50.50	24.00	14.5	37.50	46.5
17	50.00	22.75	14	38.00	45.5
18	49.00	22.00	13.25	38.75	44.5
19	48.75	21.25	13	39.00	43.5
20	48.00	20.25	12.5	39.50	42.5
21	47.25	19.50	12	40.00	41.5
22	46.50	19.00	11.75	40.25	40.75
23	45.75	18.50	11.25	40.75	39.75
24	45.00	18.00	11	41.00	39
25	44.00	17.75	10.75	41.25	38.25
26	43.25	17.25	10.5	41.50	37.75
27	42.50	16.75	10.25	41.75	37
28	41.75	16.50	10	42.00	36.75
29	41.00	16.25	10	42.00	36
30	40.50	16.00	9.75	42.25	35.75
31	40.00	15.50	9.75		35.25
32	39.50	15.25	9.5		34.75
33	39.00	15.00	9.25		34.5
34	38.50	14.75	9		34
35	38.00	14.50	9		33.5
36	37.75	14.25			33.25
37	37.25	14.25			33
38	37.00	14.00			32.75
39	36.75	13.75			32.25
40	36.25	13.50			32
41	36.00	13.25			31.75
42	35.75	13.25			31.5
43	35.50	13.25			31
44	35.00	13.00			31
45	35.00	12.75			30.75
46	34.75	12.50			30.5
47	34.50	12.50			30
48	34.00	12.50			30
49	34.00	12.25			29.75

Time (min)	Clarifier 1 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
50	33.75	12.00			29.5
51	33.25	12.00			29
52	33.00	12.00			29
53	33.00	11.75			28.75
54	32.75	11.75			28.75
55	32.50	11.50			28.5
56	32.25	11.50			28.25
57	32.00	11.50			28
58	32.00	11.25			28
59	32.00	11.25			27.75
60	31.75	11.25			27.75
61	31.50				27.5
62	31.25				27.25
63	31.00				27
64	31.00				27
65	30.75				26.75
66	30.75				26.75
67	30.50				26.5
68	30.25				26.25
69	30.00				26
70	30.00				26
71	29.75				26
72	29.50				25.75
73	29.50				25.5
74	29.25				25.5
75	29.00				25.25
76					25.25
77					25
78					25
79					25
80					25
81					24.75
82					24.75
83					24.75
84					24.5
85					24.5

Time (min)	Clarifier 1 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
86					24.5
87					24.5
88					24.25
89					24.25
90					24.25
91					24
92					24
93					24
94					24
95					24

Figure 3-6
Day 2, Clarifier 5 - 10% MLSS (TSS = 2,470 mg/L)

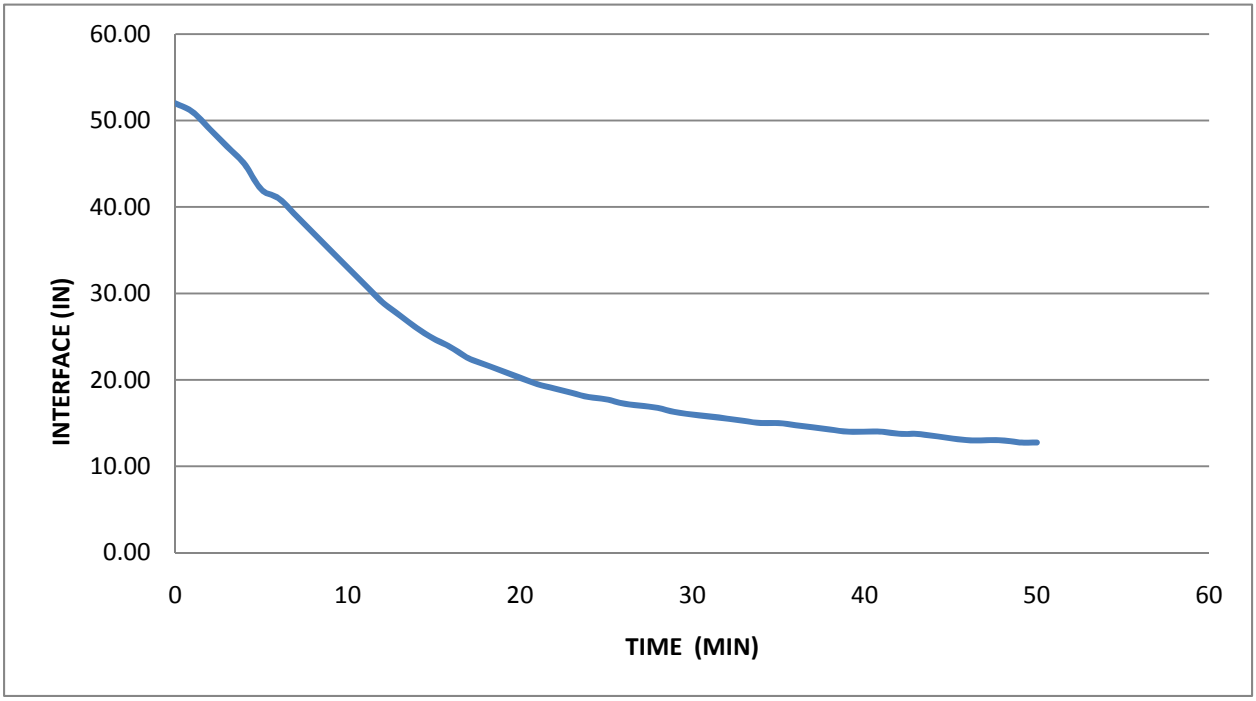


Figure 3-7
Day 2, Clarifier 5 - 66% MLSS, 33% Secondary Effluent (TSS = 1,810 mg/L)

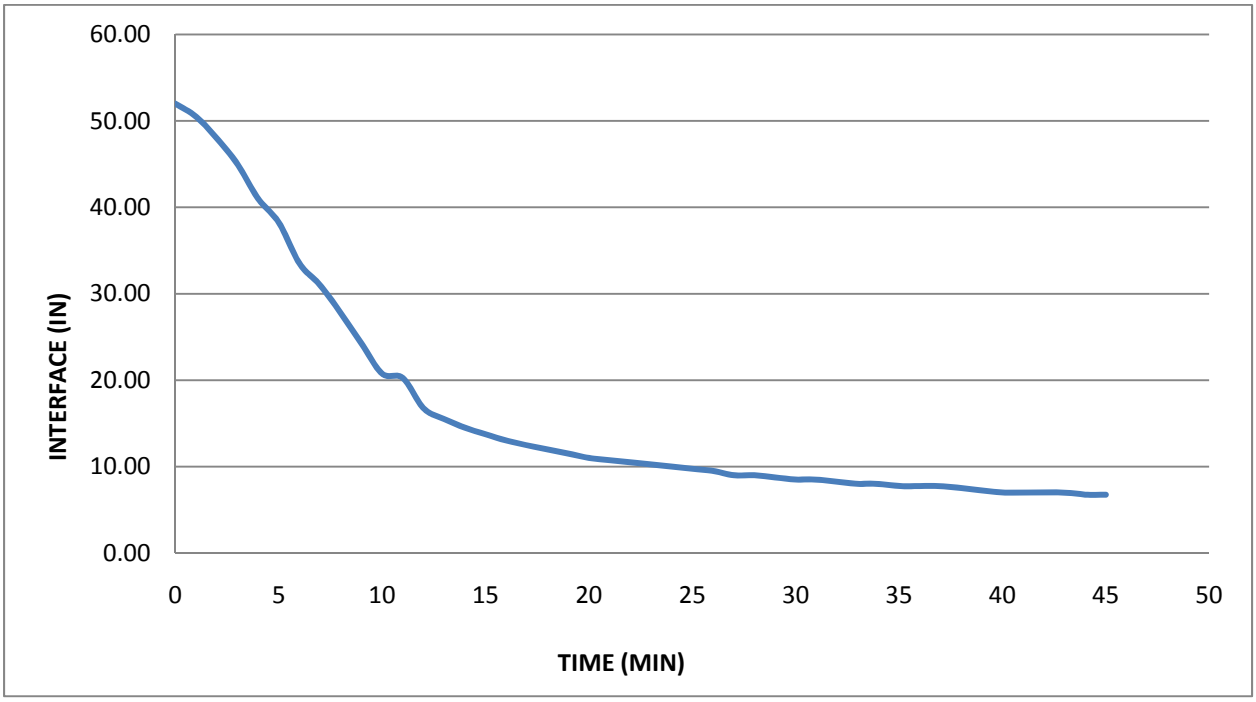


Figure 3-8
Day 2, Clarifier 5 - 33% MLSS, 66% Secondary Effluent (TSS = 707 mg/L)

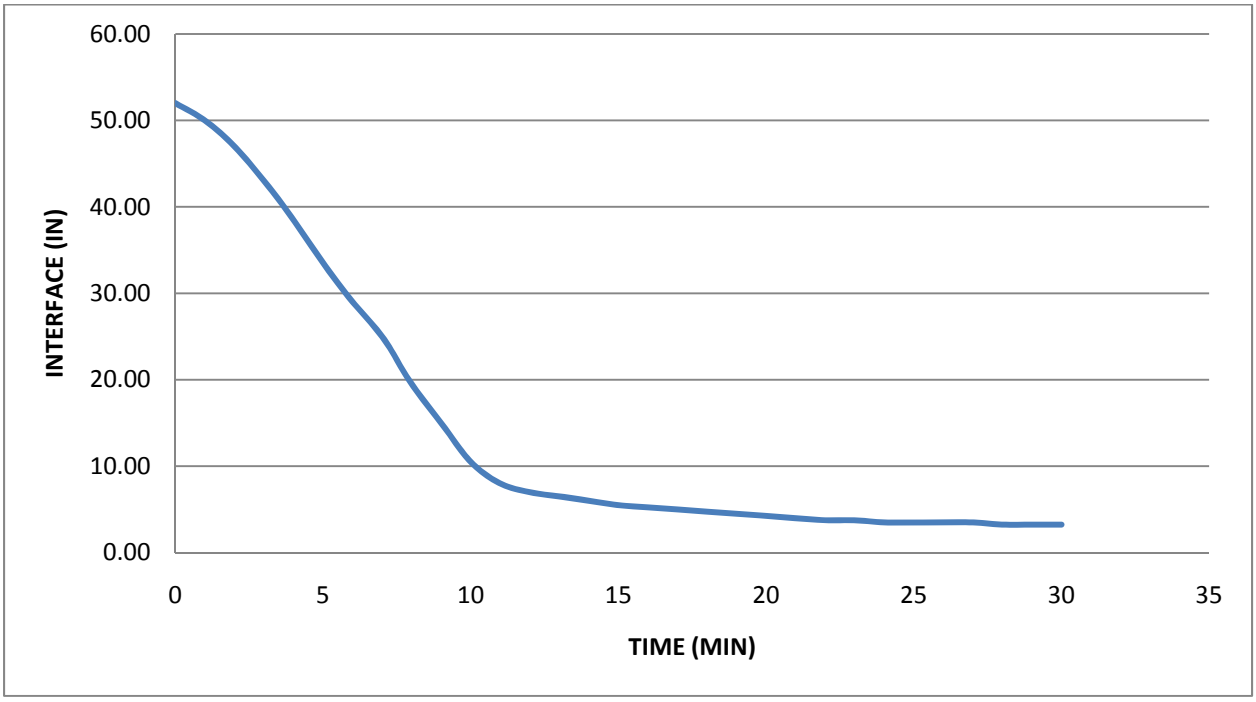


Figure 3-9
Day 2, Clarifier 5 - 100 % RAS (TSS = 10,175 mg/L)

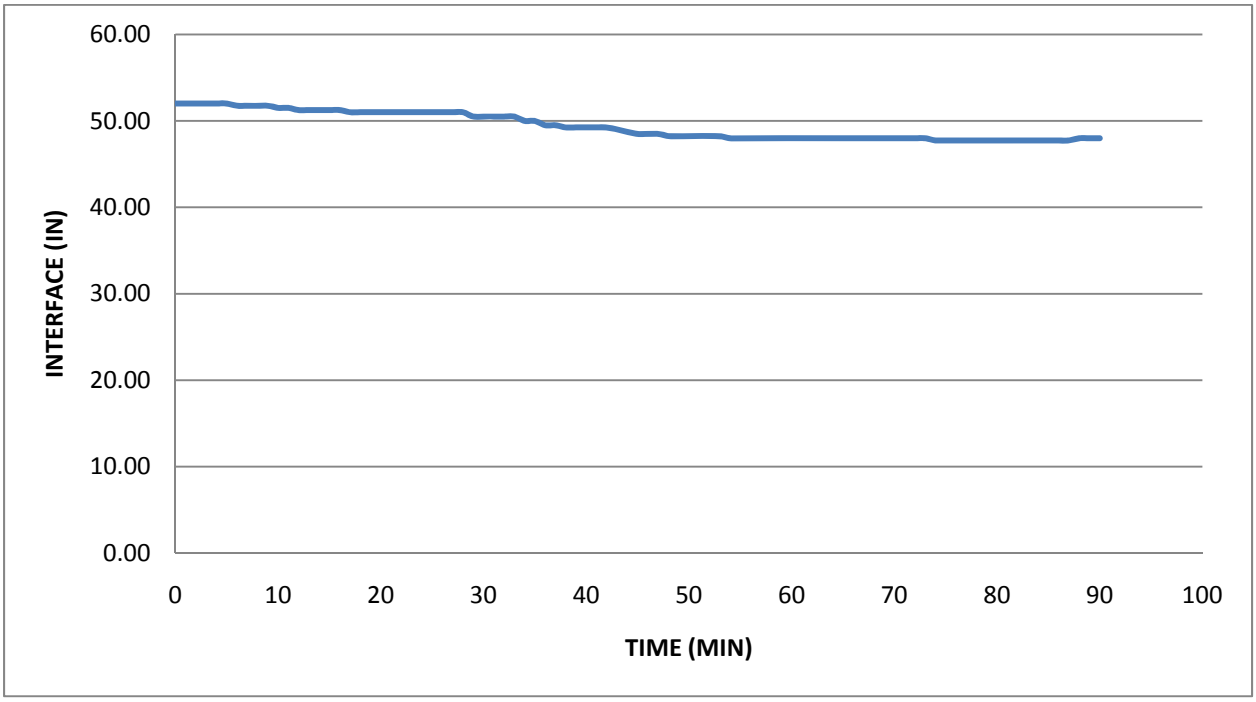


Figure 3-10
Day 2, Clarifier 5 - 50% RAS, 50% MLSS (TSS = 8,640 mg/L)

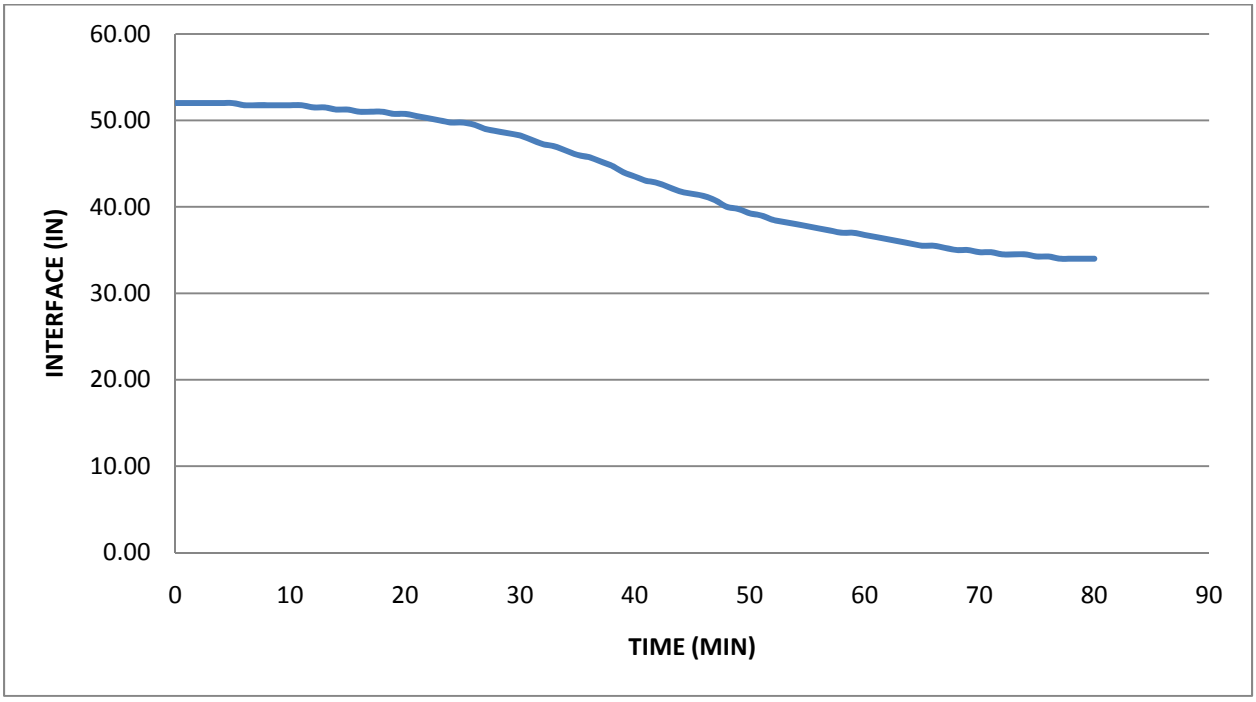


Table 3-2
Day 2 - Clarifier 5 Settling Column Measurements

Time (min)	Clarifier 5 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
0	52.00	52.00	52.00	52.00	52.00
1	52.00	51.00	50.50	50.00	52.00
2	52.00	49.00	48.00	47.00	52.00
3	52.00	47.00	45.00	43.00	52.00
4	52.00	45.00	41.00	38.50	52.00
5	52.00	42.00	38.25	33.50	52.00
6	51.75	41.00	33.50	29.00	51.75
7	51.75	39.00	31.00	25.00	51.75
8	51.75	37.00	27.75	19.50	51.75
9	51.75	35.00	24.25	15.00	51.75
10	51.50	33.00	20.75	10.50	51.75
11	51.50	31.00	20.25	8.00	51.75
12	51.25	29.00	16.75	7.00	51.50
13	51.25	27.50	15.50	6.50	51.50
14	51.25	26.00	14.50	6.00	51.25
15	51.25	24.75	13.75	5.50	51.25
16	51.25	23.75	13.00	5.25	51.00
17	51.00	22.50	12.50	5.00	51.00
18	51.00	21.75	12.00	4.75	51.00
19	51.00	21.00	11.50	4.50	50.75
20	51.00	20.25	11.00	4.25	50.75
21	51.00	19.50	10.75	4.00	50.50
22	51.00	19.00	10.50	3.75	50.25
23	51.00	18.50	10.25	3.75	50.00
24	51.00	18.00	10.00	3.50	49.75
25	51.00	17.75	9.75	3.50	49.75
26	51.00	17.25	9.50	3.50	49.50
27	51.00	17.00	9.00	3.50	49.00
28	51.00	16.75	9.00	3.25	48.75
29	50.50	16.25	8.75	3.25	48.50
30	50.50	16.00	8.50	3.25	48.25
31	50.50	15.75	8.50		47.75
32	50.50	15.50	8.25		47.25
33	50.50	15.25	8.00		47.00

Time (min)	Clarifier 5 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
34	50.00	15.00	8.00		46.50
35	50.00	15.00	7.75		46.00
36	49.50	14.75	7.75		45.75
37	49.50	14.50	7.75		45.25
38	49.25	14.25	7.50		44.75
39	49.25	14.00	7.25		44.00
40	49.25	14.00	7.00		43.50
41	49.25	14.00	7.00		43.00
42	49.25	13.75	7.00		42.75
43	49.00	13.75	7.00		42.25
44	48.75	13.50	6.75		41.75
45	48.50	13.25	6.75		41.50
46	48.50	13.00			41.25
47	48.50	13.00			40.75
48	48.25	13.00			40.00
49	48.25	12.75			39.75
50	48.25	12.75			39.25
51	48.25				39.00
52	48.25				38.50
53	48.25				38.25
54	48.00				38.00
55	48.00				37.75
56	48.00				37.50
57	48.00				37.25
58	48.00				37.00
59	48.00				37.00
60	48.00				36.75
61	48.00				36.50
62	48.00				36.25
63	48.00				36.00
64	48.00				35.75
65	48.00				35.50
66	48.00				35.50
67	48.00				35.25
68	48.00				35.00
69	48.00				35.00

Time (min)	Clarifier 5 Interface (in)				
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE	50% MLSS, 50% RAS
70	48.00				34.75
71	48.00				34.75
72	48.00				34.50
73	48.00				34.50
74	47.75				34.50
75	47.75				34.25
76	47.75				34.25
77	47.75				34.00
78	47.75				34.00
79	47.75				34.00
80	47.75				34.00
81	47.75				
82	47.75				
83	47.75				
84	47.75				
85	47.75				
86	47.75				
87	47.75				
88	48				
89	48				
90	48				

Figure 3-11
Day 3, Clarifiers 2 & 3 - 100% RAS (TSS = 7,300 mg/L)

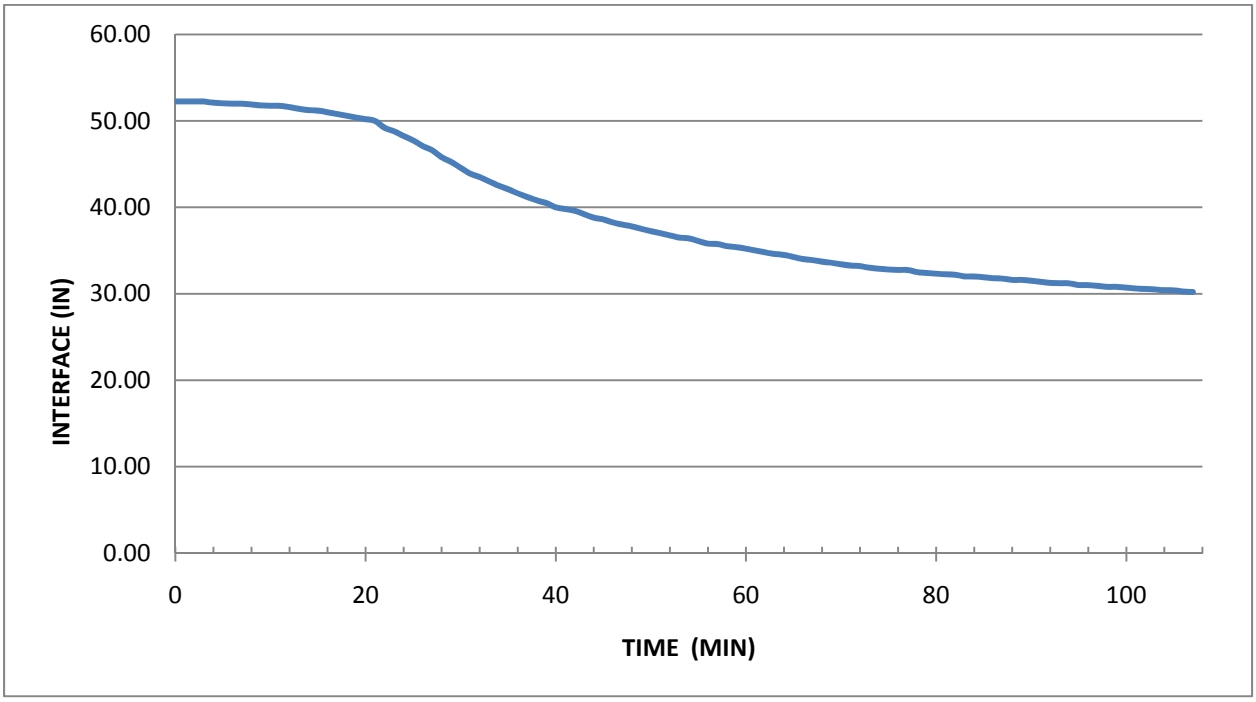


Figure 3-12
Day 3, Clarifiers 2 & 3 - 100% MLSS (TSS = 2,950 mg/L)

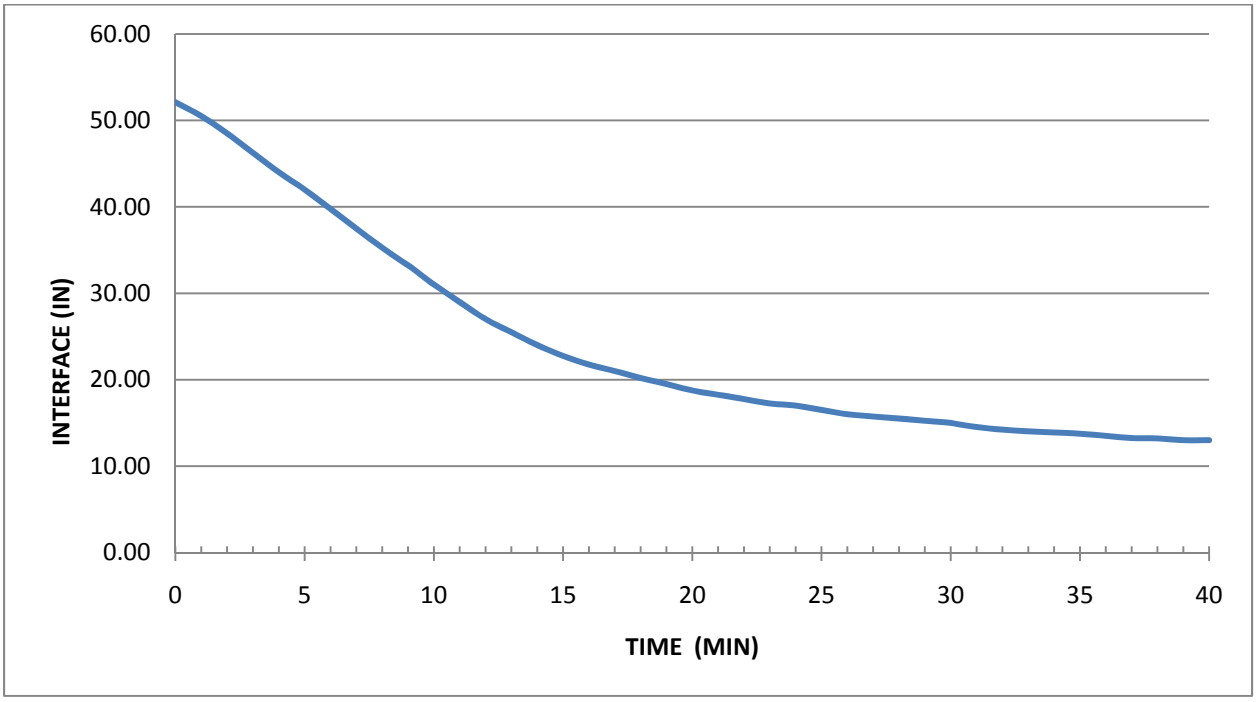


Figure 3-13
Day 3, Clarifiers 2 & 3 - 50% RAS, 50% MLSS (TSS = 6,480 mg/L)

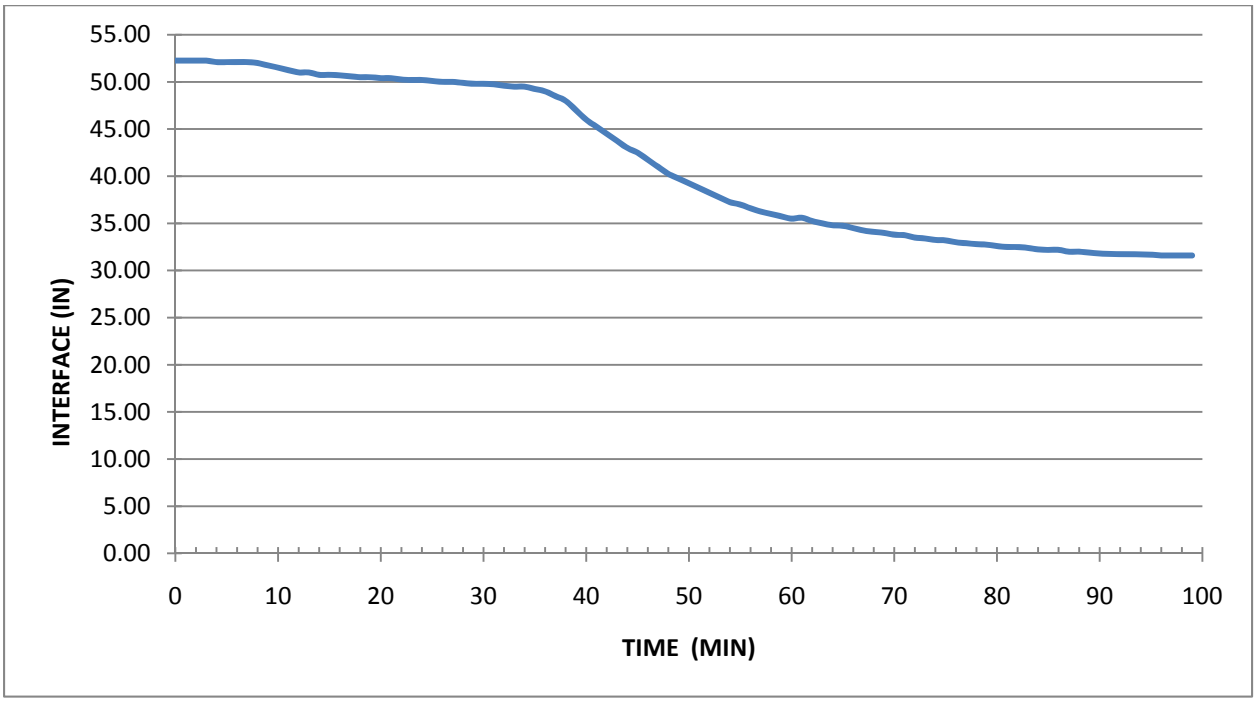


Figure 3-14
Day 3, Clarifiers 2 & 3 - 66% MLSS, 33% Secondary Effluent (TSS = 1,640 mg/L)

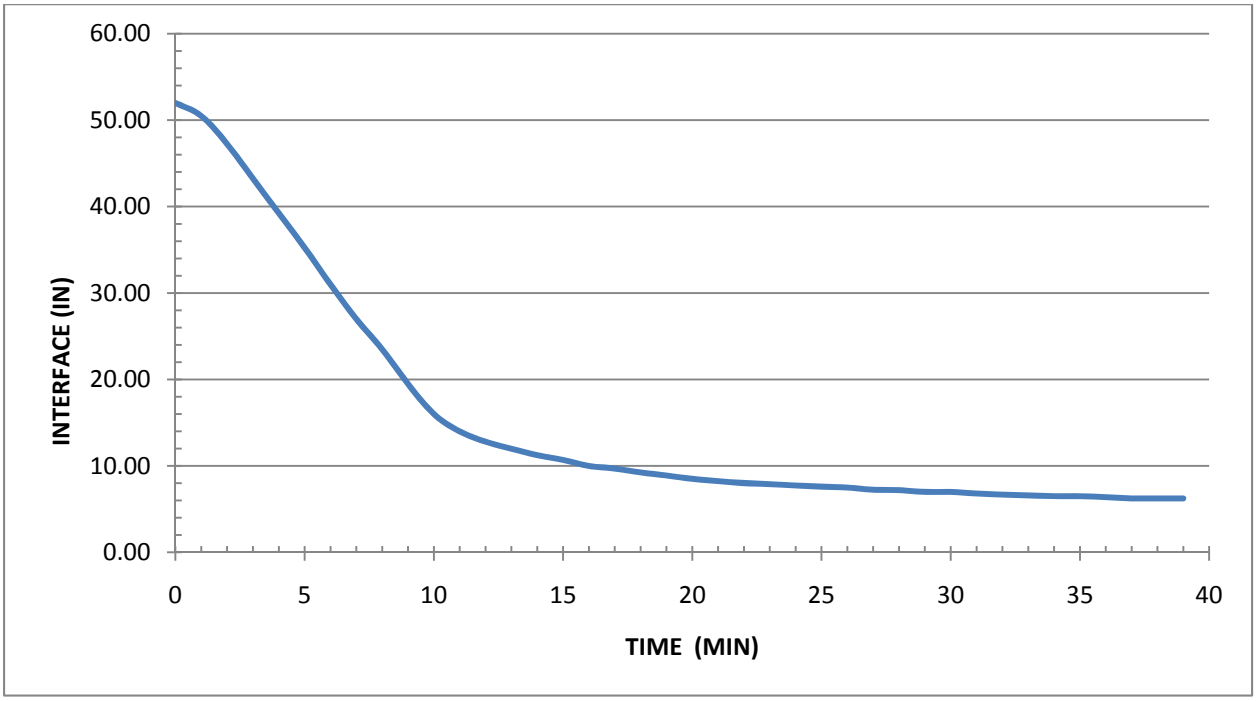


Figure 3-15
Day 3, Clarifiers 2 & 3 - 33% MLSS, 66% Secondary Effluent (TSS = 825 mg/L)

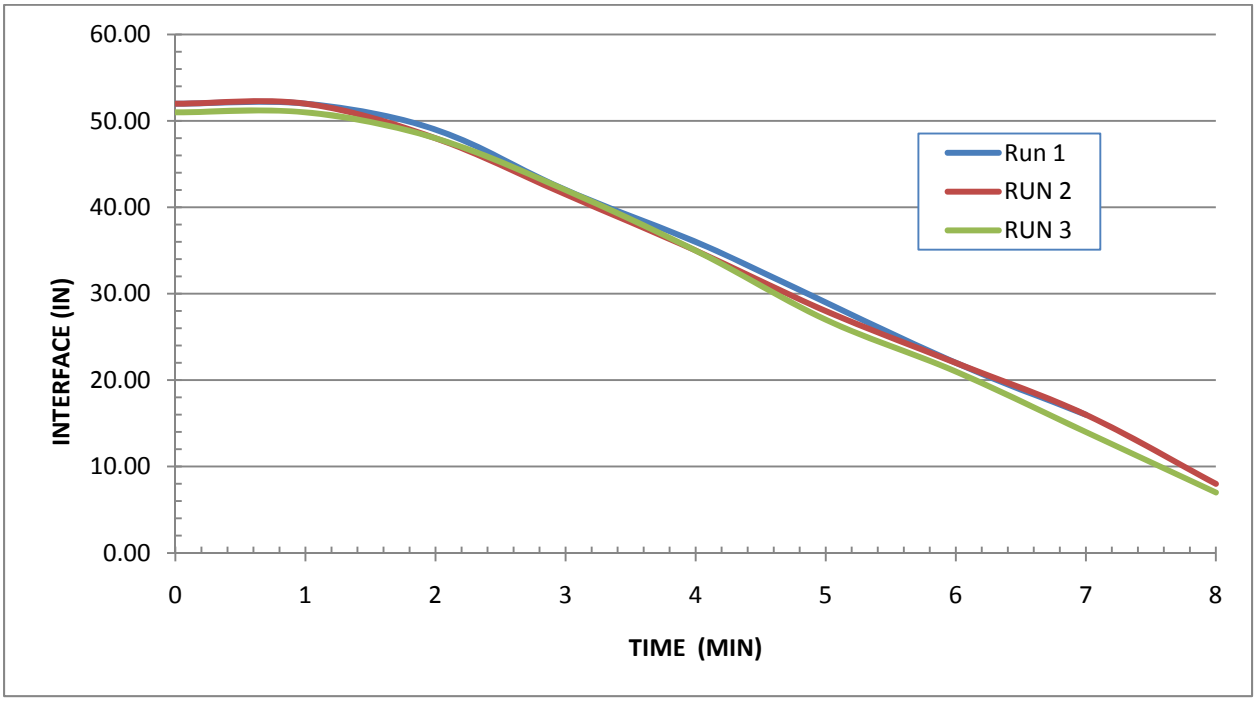


Table 3-3
Day 3 - Clarifiers 2 and 3 Settling Column Measurements

Time (min)	Clarifiers 2 and 3 Interface (in)						
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE			50% MLSS, 50% RAS
				Run 1	Run 2	Run 3	
0	52.25	52.10	52.00	52.00	52.00	51.00	52.25
1	52.25	50.50	50.50	52.00	52.00	51.00	52.25
2	52.25	48.50	47.25	49.00	48.00	48.00	52.25
3	52.25	46.25	43.25	42.00	41.50	42.00	52.25
4	52.10	44.00	39.25	36.00	35.00	35.00	52.10
5	52.05	42.00	35.25	29.00	28.00	27.00	52.10
6	52.00	39.75	31.00	22.00	22.00	21.00	52.10
7	52.00	37.50	27.00	16.00	16.00	14.00	52.10
8	51.90	35.25	23.50		8.00	7.00	52.00
9	51.80	33.25	19.50				51.75
10	51.75	31.00	16.00				51.50
11	51.75	29.00	14.00				51.25
12	51.60	27.00	12.80				51.00

Time (min)	Clarifiers 2 and 3 Interface (in)						
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE			50% MLSS, 50% RAS
				Run 1	Run 2	Run 3	
13	51.40	25.50	12.00				51.00
14	51.25	24.00	11.25				50.75
15	51.20	22.75	10.70				50.75
16	51.00	21.75	10.00				50.70
17	50.80	21.00	9.70				50.60
18	50.60	20.20	9.25				50.50
19	50.40	19.50	8.90				50.50
20	50.20	18.75	8.50				50.40
21	50.00	18.25	8.25				50.40
22	49.20	17.75	8.00				50.25
23	48.80	17.25	7.90				50.20
24	48.25	17.00	7.75				50.20
25	47.75	16.50	7.60				50.10
26	47.10	16.00	7.50				50.00
27	46.60	15.75	7.25				50.00
28	45.80	15.50	7.20				49.90
29	45.25	15.25	7.00				49.80
30	44.58	15.00	7.00				49.80
31	43.90	14.50	6.80				49.75
32	43.50	14.25	6.70				49.60
33	43.00	14.00	6.60				49.50
34	42.50	13.90	6.50				49.50
35	42.10	13.75	6.50				49.25
36	41.60	13.50	6.40				49.00
37	41.20	13.25	6.25				48.50
38	40.80	13.20	6.25				48.00
39	40.50	13.00	6.25				47.00
40	40.00	13.00					46.00
41	39.80						45.25
42	39.60						44.50
43	39.20						43.75
44	38.80						43.00
45	38.60						42.50
46	38.25						41.75
47	38.00						41.00

Time (min)	Clarifiers 2 and 3 Interface (in)						
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE			50% MLSS, 50% RAS
				Run 1	Run 2	Run 3	
48	37.80						40.25
49	37.50						39.75
50	37.25						39.25
51	37.00						38.75
52	36.75						38.25
53	36.50						37.75
54	36.40						37.25
55	36.10						37.00
56	35.80						36.60
57	35.75						36.25
58	35.50						36.00
59	35.40						35.75
60	35.20						35.50
61	35.00						35.60
62	34.80						35.25
63	34.60						35.00
64	34.50						34.80
65	34.25						34.75
66	34.00						34.50
67	33.90						34.25
68	33.70						34.10
69	33.60						34.00
70	33.40						33.80
71	33.25						33.75
72	33.20						33.50
73	33.00						33.40
74	32.90						33.25
75	32.80						33.20
76	32.75						33.00
77	32.75						32.90
78	32.5						32.80
79	32.4						32.75
80	32.32						32.60
81	32.25						32.50
82	32.2						32.50

Time (min)	Clarifiers 2 and 3 Interface (in)						
	100% RAS	100% MLSS	66% MLSS, 33% SE	33% MLSS, 66% SE			50% MLSS, 50% RAS
				Run 1	Run 2	Run 3	
83	32						32.40
84	32						32.25
85	31.9						32.20
86	31.8						32.20
87	31.75						32.00
88	31.6						32.00
89	31.6						31.90
90	31.5						31.80
91	31.3875						31.75
92	31.25						31.75
93	31.22						31.75
94	31.2						31.70
95	31						31.70
96	31						31.60
97	30.9						31.60
98	30.8						31.60
99	30.8						31.60
100	30.7						
101	30.6						
102	30.55						
103	30.5						
104	30.4						
105	30.4						
106	30.25						
107	30.2						

Figure 3-16
Day 4, Clarifier 4 - 100% MLSS (TSS = 5,450 mg/L)

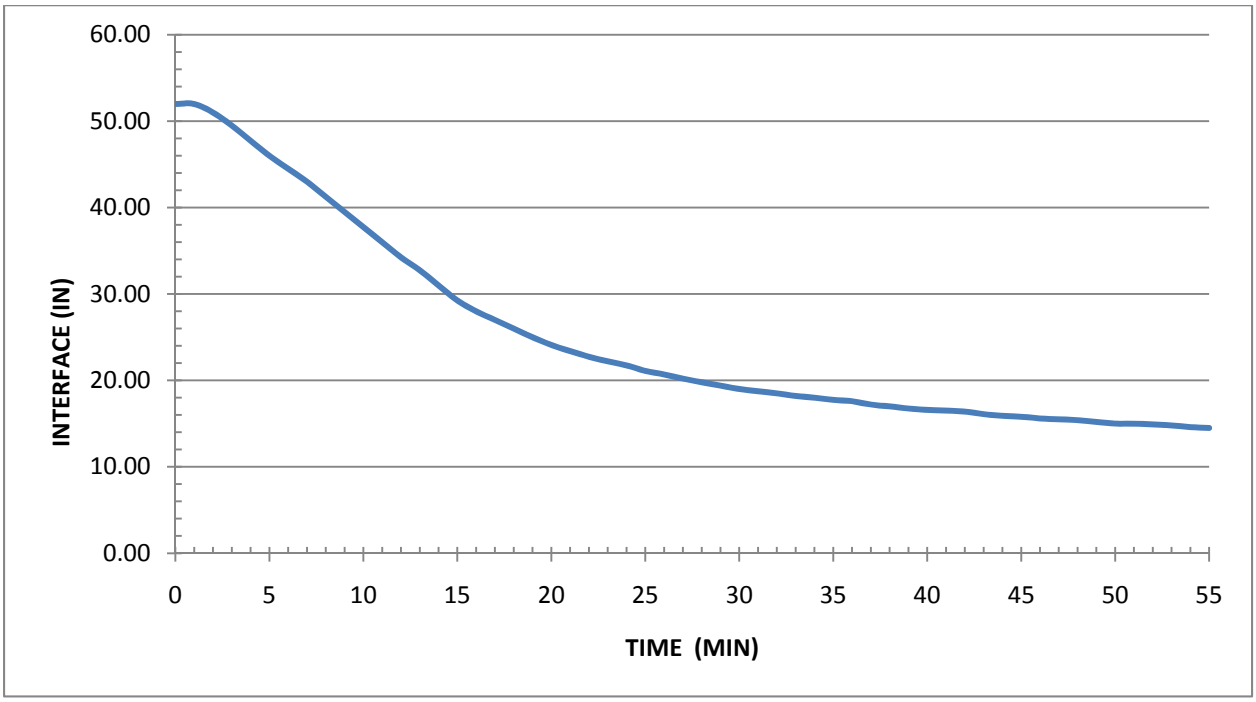


Figure 3-17
Day 4, Clarifier 4 - 100% RAS (TSS = 6,280 mg/L)

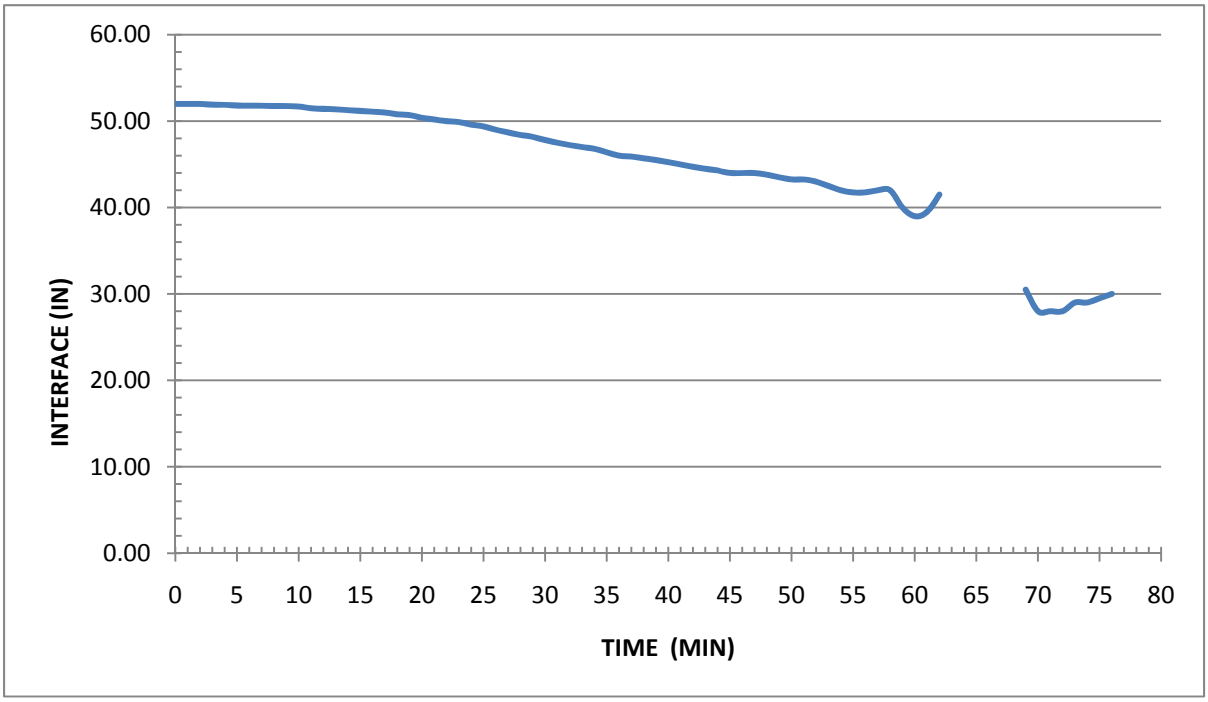


Figure 3-18
Day 4, Clarifier 4 - 33% MLSS, 66% RAS (TSS = 5,300 mg/L)

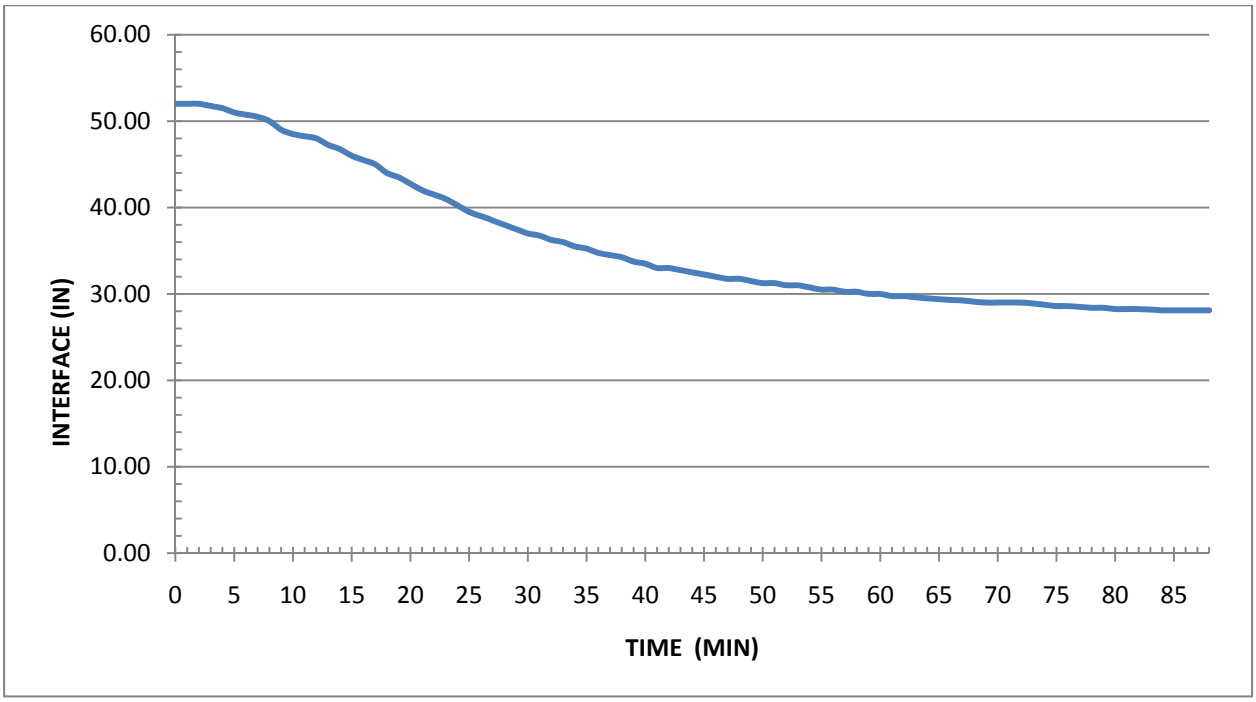


Figure 3-19
Day 4, Clarifier 4 - 66% MLSS, 33% Secondary Effluent (TSS = 2,400 mg/L)

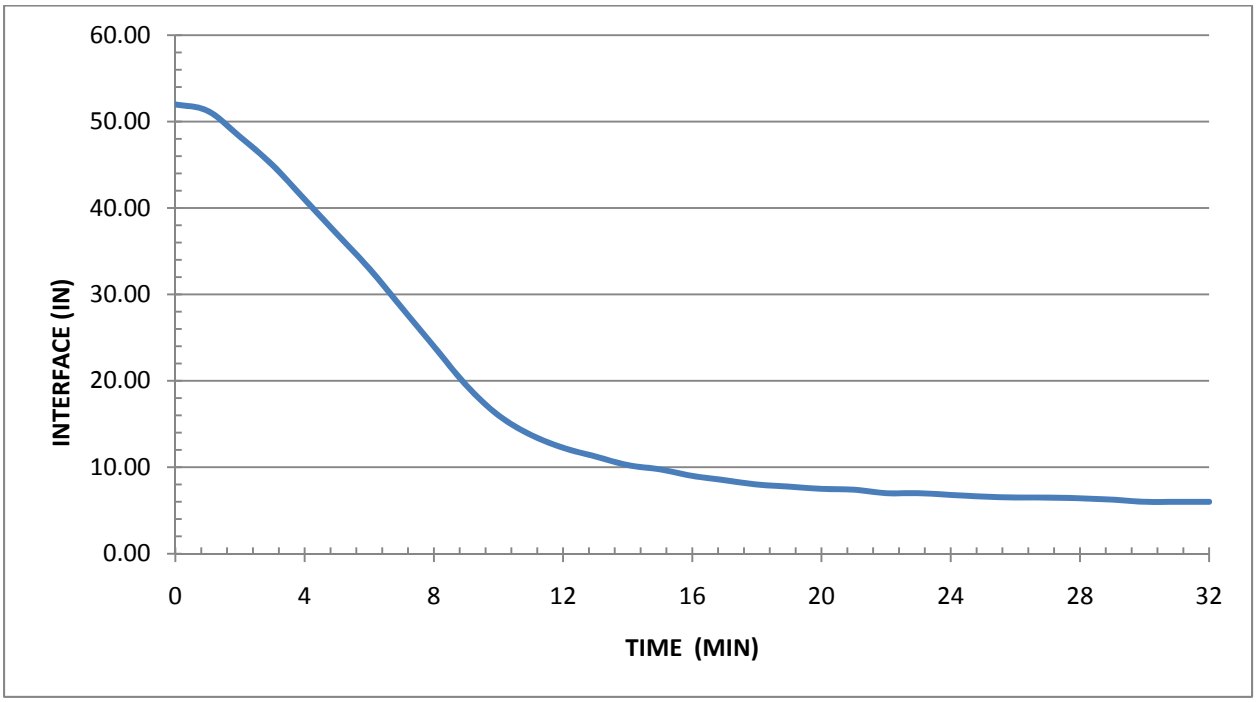


Figure 3-20

Day 4, Clarifier 4 - 50% MLSS, 50% Secondary Effluent (TSS = 1,170 mg/L)

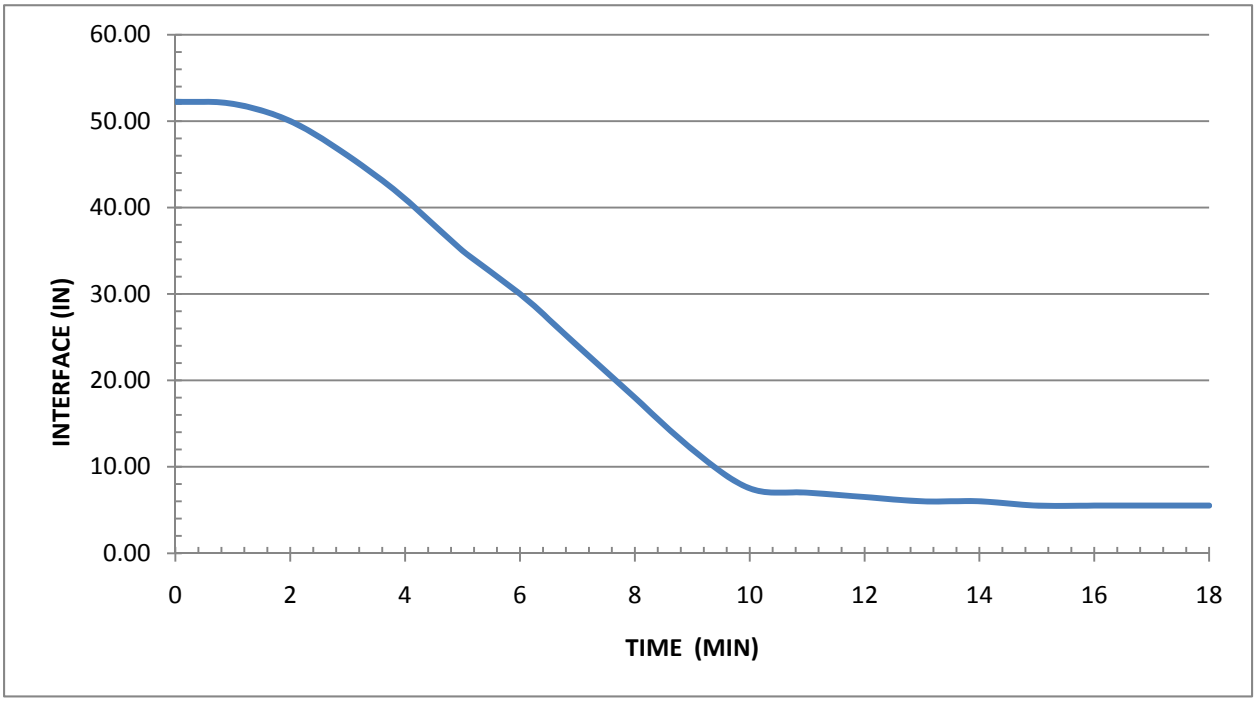


Figure 3-21

Day 4, Clarifier 4 - 66% MLSS, 33% RAS (TSS = 4,040 mg/L)

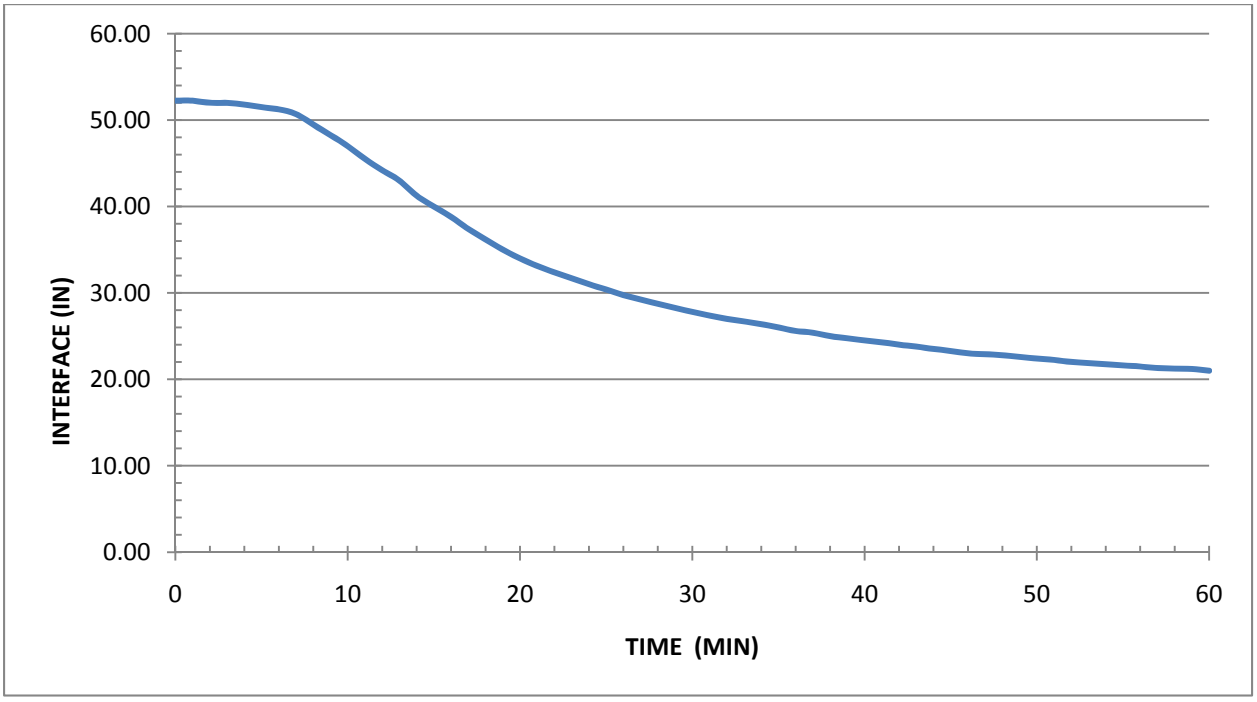


Table 3-4
Day 4 - Clarifier 4 Settling Column Measurements

Time (min)	Clarifier 4 Interface (in)					
	100% RAS	100% MLSS	66% MLSS, 33% SE	66% MLSS, 33% RAS	33% MLSS, 66% RAS	50% MLSS, 50% SE
0	52.00	52.00	52.00	52.25	52.00	52.25
1	52.00	52.00	51.25	52.25	52.00	52.00
2	52.00	51.00	48.25	52.00	52.00	50.00
3	51.90	49.50	45.00	52.00	51.75	46.00
4	51.90	47.75	41.00	51.80	51.50	41.00
5	51.80	46.00	37.00	51.50	51.00	35.00
6	51.80	44.50	33.00	51.25	50.75	30.00
7	51.80	43.00	28.50	50.70	50.50	24.00
8	51.75	41.25	24.00	49.50	50.00	18.00
9	51.75	39.50	19.50	48.25	49.00	12.00
10	51.70	37.75	16.00	47.00	48.50	7.50
11	51.50	36.00	13.75	45.50	48.25	7.00
12	51.40	34.25	12.25	44.20	48.00	6.50
13	51.40	32.75	11.25	43.00	47.25	6.00
14	51.25	31.00	10.25	41.25	46.75	6.00
15	51.20	29.25	9.75	40.00	46.00	5.50
16	51.10	28.00	9.00	38.80	45.50	5.50
17	51.00	27.00	8.50	37.40	45.00	5.50
18	50.80	26.00	8.00	36.20	44.00	5.50
19	50.70	25.00	7.75	35.00	43.50	
20	50.40	24.10	7.50	34.00	42.75	
21	50.20	23.40	7.40	33.10	42.00	
22	50.00	22.75	7.00	32.40	41.50	
23	49.90	22.20	7.00	31.70	41.00	
24	49.60	21.75	6.80	31.00	40.25	
25	49.40	21.10	6.60	30.40	39.50	
26	49.00	20.70	6.50	29.75	39.00	
27	48.70	20.20	6.50	29.25	38.50	
28	48.40	19.80	6.40	28.75	38.00	
29	48.20	19.40	6.25	28.25	37.50	
30	47.80	19.00	6.00	27.80	37.00	
31	47.50	18.75	6.00	27.4	36.75	
32	47.25	18.50	6.00	27	36.25	
33	47.00	18.20		26.7	36	

Time (min)	Clarifier 4 Interface (in)					
	100% RAS	100% MLSS	66% MLSS, 33% SE	66% MLSS, 33% RAS	33% MLSS, 66% RAS	50% MLSS, 50% SE
34	46.80	18.00		26.4	35.5	
35	46.40	17.75		26	35.25	
36	46.00	17.60		25.6	34.75	
37	45.90	17.20		25.4	34.5	
38	45.70	17.00		25	34.25	
39	45.50	16.75		24.75	33.75	
40	45.25	16.60		24.5	33.5	
41	45.00	16.50		24.3	33	
42	44.70	16.40		24	33	
43	44.50	16.10		23.8	32.75	
44	44.30	15.90		23.5	32.5	
45	44.00	15.80		23.3	32.25	
46	44.00	15.60		23	32	
47	44.00	15.50		22.9	31.75	
48	43.80	15.40		22.8	31.75	
49	43.50	15.20		22.6	31.5	
50	43.25	15.00		22.4	31.25	
51	43.25	15.00		22.25	31.25	
52	43.00	14.90		22	31	
53	42.50	14.80		21.9	31	
54	42.00	14.60		21.75	30.75	
55	41.75	14.50		21.6	30.5	
56	41.75			21.5	30.5	
57	42.00			21.3	30.25	
58	42.00			21.25	30.25	
59	40.00			21.2	30	
60	39.00			21	30	
61	39.50				29.75	
62	41.50				29.75	
63					29.6	
64					29.5	
65					29.4	
66					29.3	
67					29.25	
68					29.1	
69	30.50				29	

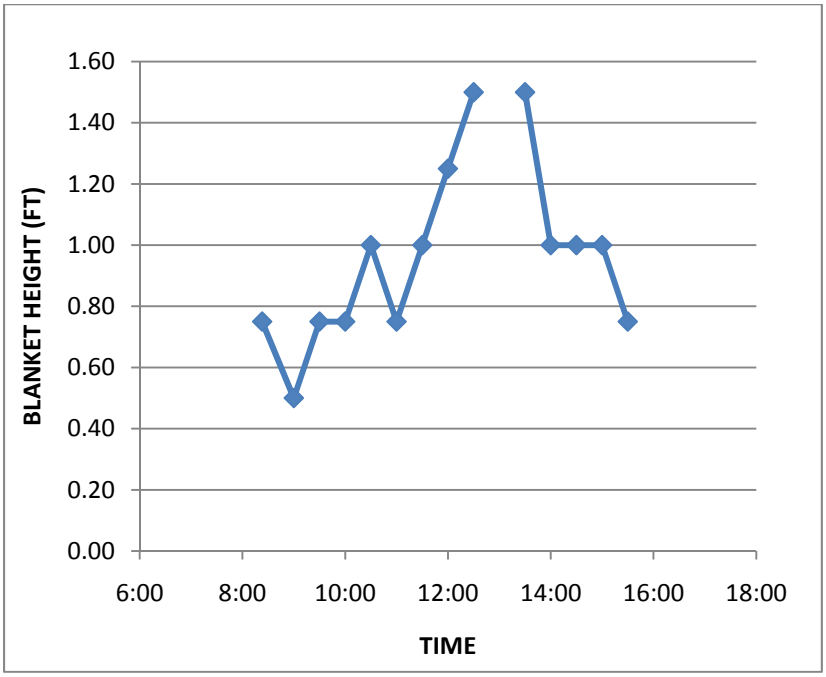
Time (min)	Clarifier 4 Interface (in)					
	100% RAS	100% MLSS	66% MLSS, 33% SE	66% MLSS, 33% RAS	33% MLSS, 66% RAS	50% MLSS, 50% SE
70	28.00				29	
71	28.00				29	
72	28.00				29	
73	29.00				28.9	
74	29.00				28.75	
75	29.50				28.6	
76	30				28.6	
77					28.5	
78					28.4	
79					28.4	
80					28.25	
81					28.25	
82					28.25	
83					28.2	
84					28.1	
85					28.1	
86					28.1	
87					28.1	
88					28.1	

4. STRESS TESTING

Clarifier blanket levels were sampled during the stress tests at intervals of approximately half an hour. Figure 4-1 shows the blanket levels recorded in Clarifier 1. Figure 4-2 shows the blanket levels recorded in Clarifier 5. Figure 4-3 shows the blanket levels recorded in Clarifier 2. The graph in Figure 4-3 shows the blanket depth with only the thickened sludge and the blanket depth with the thickened sludge and the cloudy layer of solids above it. Figures 4-4 and 4-5 shows these same blanket recordings for Clarifiers 2 and 4, respectively.

Figure 4-1

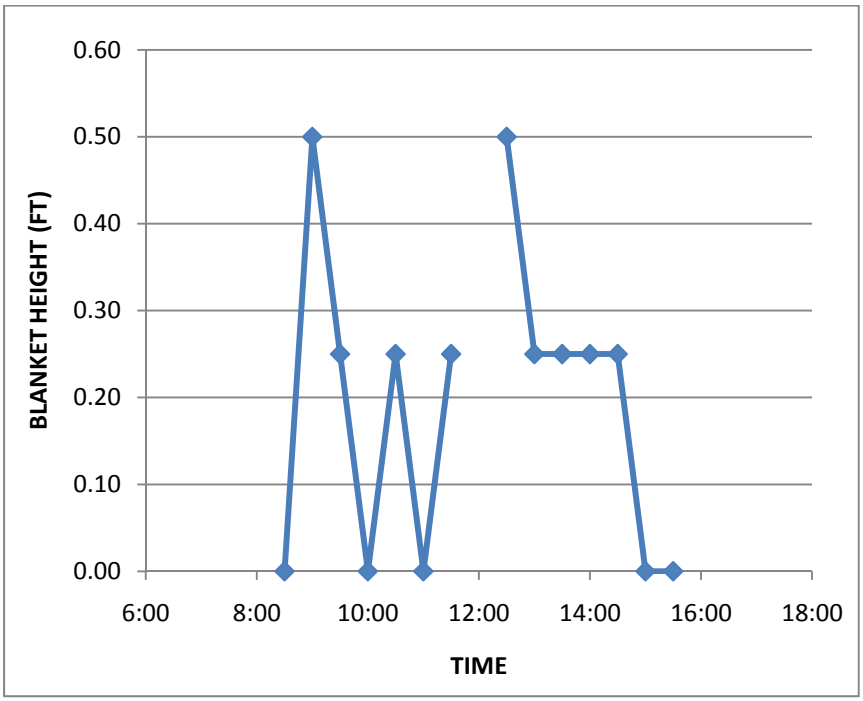
Clarifier 1 Blanket Depth



TIME	SLUDGE DEPTH (ft)
8:23	0.75
9:00	0.50
9:30	0.75
10:00	0.75
10:30	1.00
11:00	0.75
11:30	1.00
12:00	1.25
12:30	1.50
13:00	
13:30	1.50
14:00	1.00
14:30	1.00
15:00	1.00
15:30	0.75

Figure 4-2

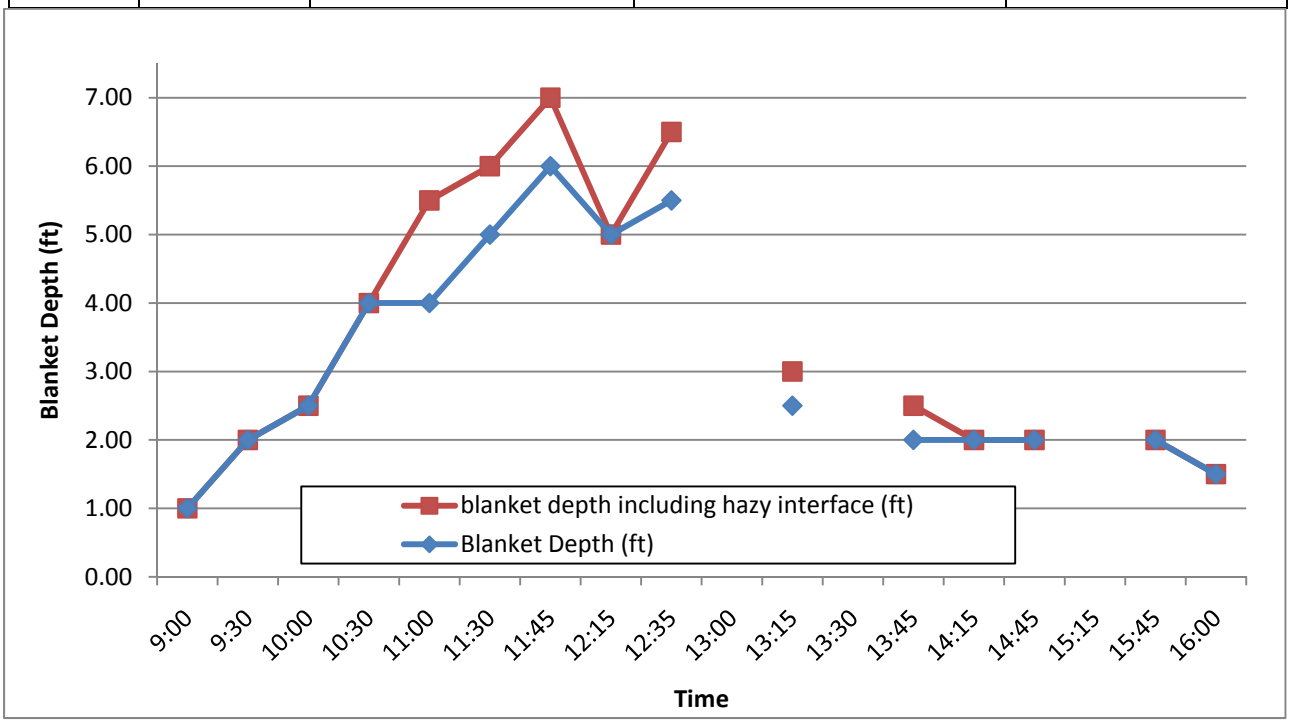
Clarifier 5 Blanket Depth



TIME	DEPTH (ft)	FLUME HT (in)
8:30	0.00	9.5
9:00	0.50	9.0
9:30	0.25	10.0
10:00	0.00	10.5
10:30	0.25	11.0
11:00	0.00	11.5
11:30	0.25	11.5
12:00		
12:30	0.50	11.5
13:00	0.25	11.0
13:30	0.25	11.5
14:00	0.25	10.5
14:30	0.25	10.5
15:00	0.00	11.0
15:30	0.00	10.5

Figure 4-3
Clarifier 2 Blanket Depth

TIME	Blanket Depth (ft)	Blanket depth including hazy interface (ft)	Clar. 2 water level in effluent box (in from top)	Clar. 2 flume water depth (in)
9:00	1.00	1.00	56.00	13.0
9:30	2.00	2.00	52.00	16.0
10:00	2.50	2.50	49.00	17.0
10:30	4.00	4.00	49.00	17.0
11:00	4.00	5.50	48.00	16.5
11:30	5.00	6.00	48.00	17.0
11:45	6.00	7.00		
12:15	5.00	5.00	48.00	16.5
12:35	5.50	6.50		
13:00			54.00	15.0
13:15	2.50	3.00		
13:30				14.5
13:45	2.00	2.50		
14:15	2.00	2.00	56.00	14.0
14:45	2.00	2.00		
15:15				14.5
15:45	2.00	2.00		15.0
16:00	1.50	1.50		



. **Figure 4-4**

Clarifier 3 Blanket Depth

TIME	Blanket Depth (ft)	Blanket depth including hazy interface (ft)	Clar 3 water level in effluent box (in from top)	Clar 3 flume water depth (in)
9:00	0.75	0.75	50.00	13.0
9:30	1.00	3.00	46.00	16.0
10:00	3.00	3.00	44.00	17.0
10:30	4.50	4.50	44.00	17.0
11:00	4.50	5.50	43.00	17.5
11:30	6.00	6.50	44.00	18.0
11:45	5.50	6.50		
12:15	5.00	5.00	44.00	17.0
12:35	5.50	6.50		
13:00			48.00	15.0
13:15	4.00	4.50		
13:30				14.5
13:45	4.00	4.00		
14:15	3.50	4.00	51.00	14.0
14:45	4.00	4.00		
15:15				14.5
15:45	3.00	3.00		15.0
16:00	2.50	3.00		

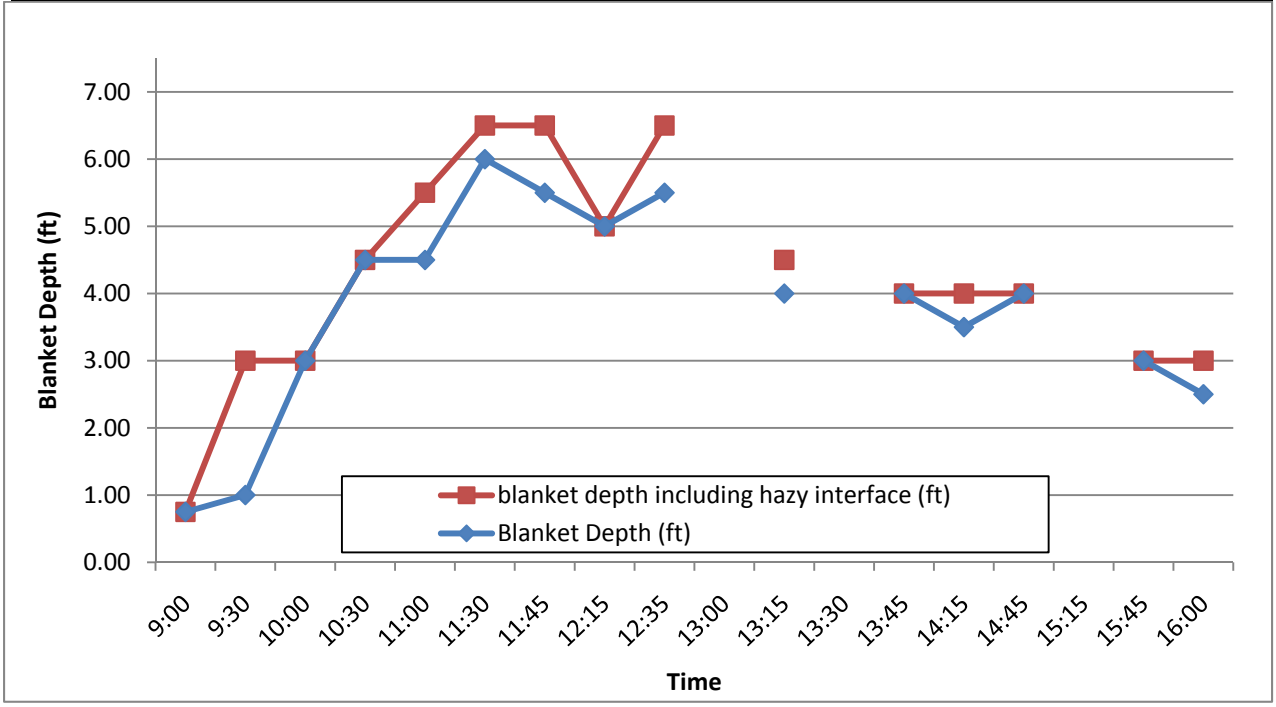


Figure 4-5
Clarifier 4 Blanket Depth

TIME	Measurement at outer wall		Measurement halfway to center		Clar 2 flume water depth (in)
	Blanket Depth (ft)	Depth including hazy interface (ft)	Blanket Depth (ft)	Depth including hazy interface (ft)	
8:30	-	-	-	-	6.0
8:45	-	-	-	-	
9:00	0.00	5.50	-	-	13.5
9:45	0.00	5.00	3.00	5.00	14.5
10:00	0.50	3.00	4.00	5.00	15.0
10:30	5.00	5.00	4.00	4.50	14.5
11:00	4.00	5.00	5.00	5.00	15.5
11:30	6.00	6.50	6.00	6.50	15.5
12:00	6.00	6.50	5.50	6.00	15.5
12:30	6.00	6.50	6.50	7.00	15.0
13:00	6.50	7.00	5.00	5.50	15.0
13:45	5.00	5.50	5.00	5.50	13.5
14:00					
14:15	4.50	5.00	3.00	4.00	13.5
15:00	3.00	5.00	3.00	4.00	15.5

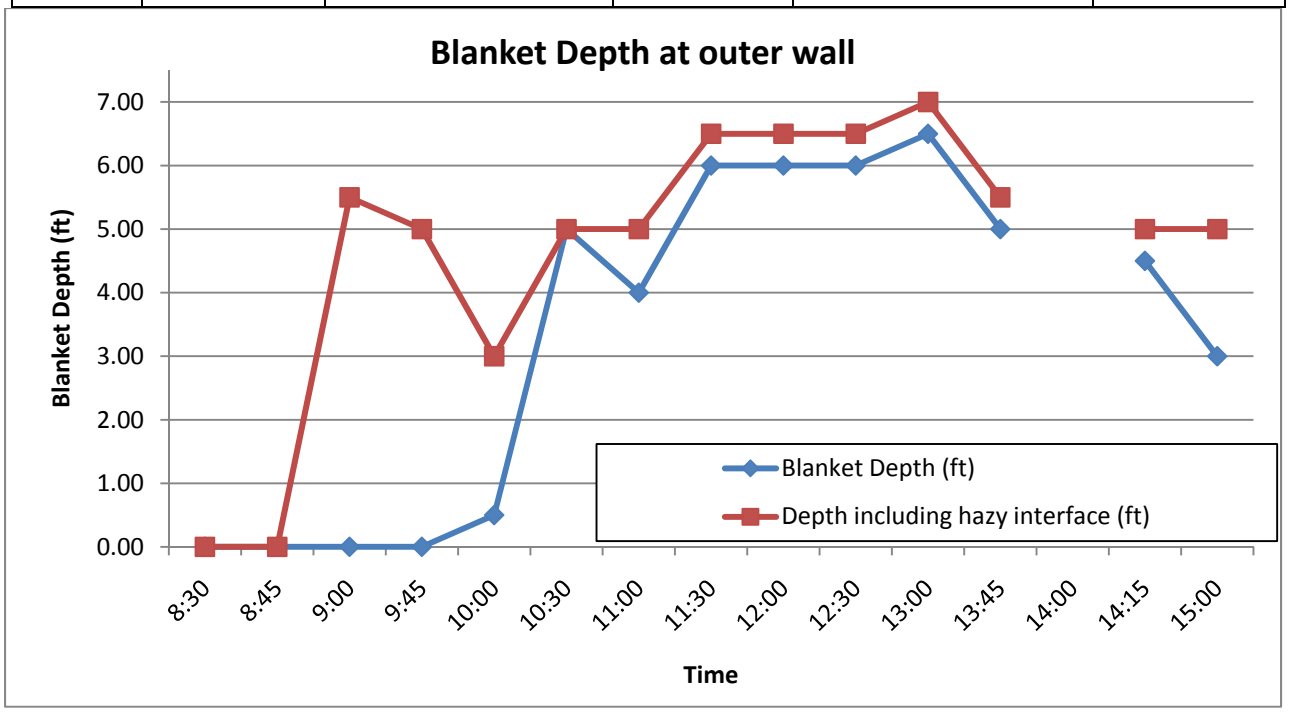


Figure 4-5 (cont)
Clarifier 4 Blanket Depth

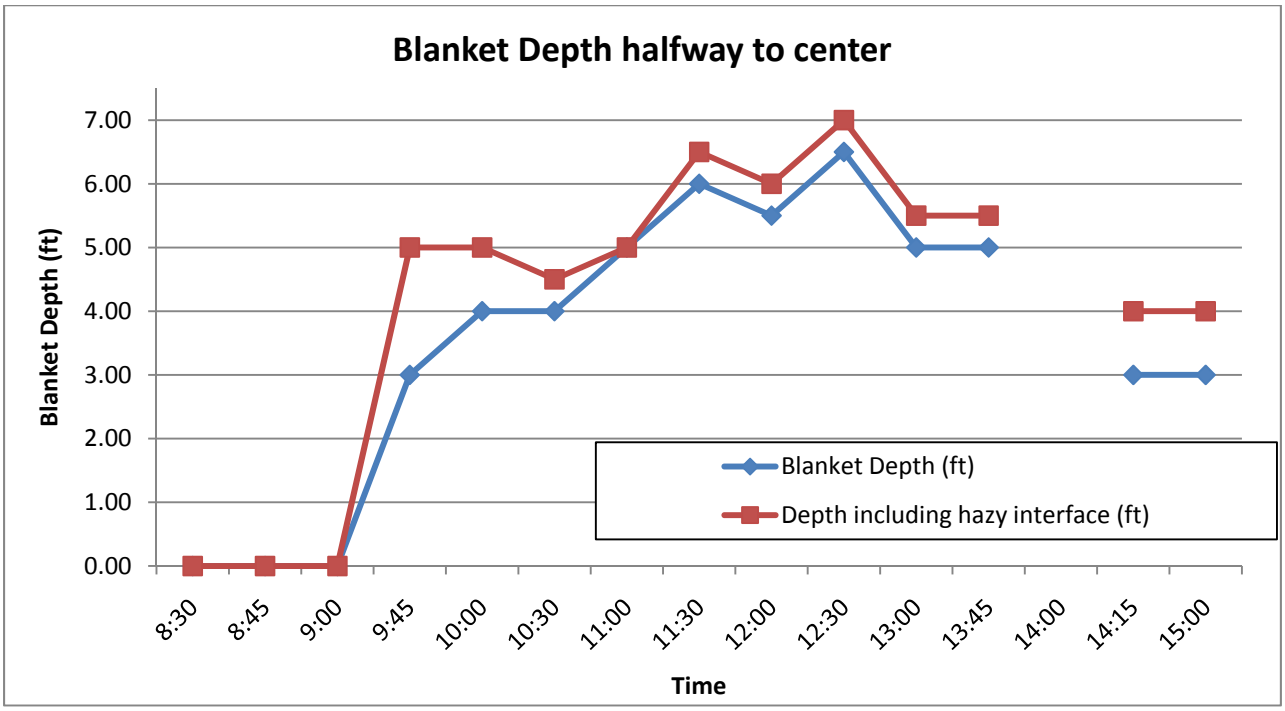


Figure 4-6
Clarifier MLSS Concentrations during Stress Testing

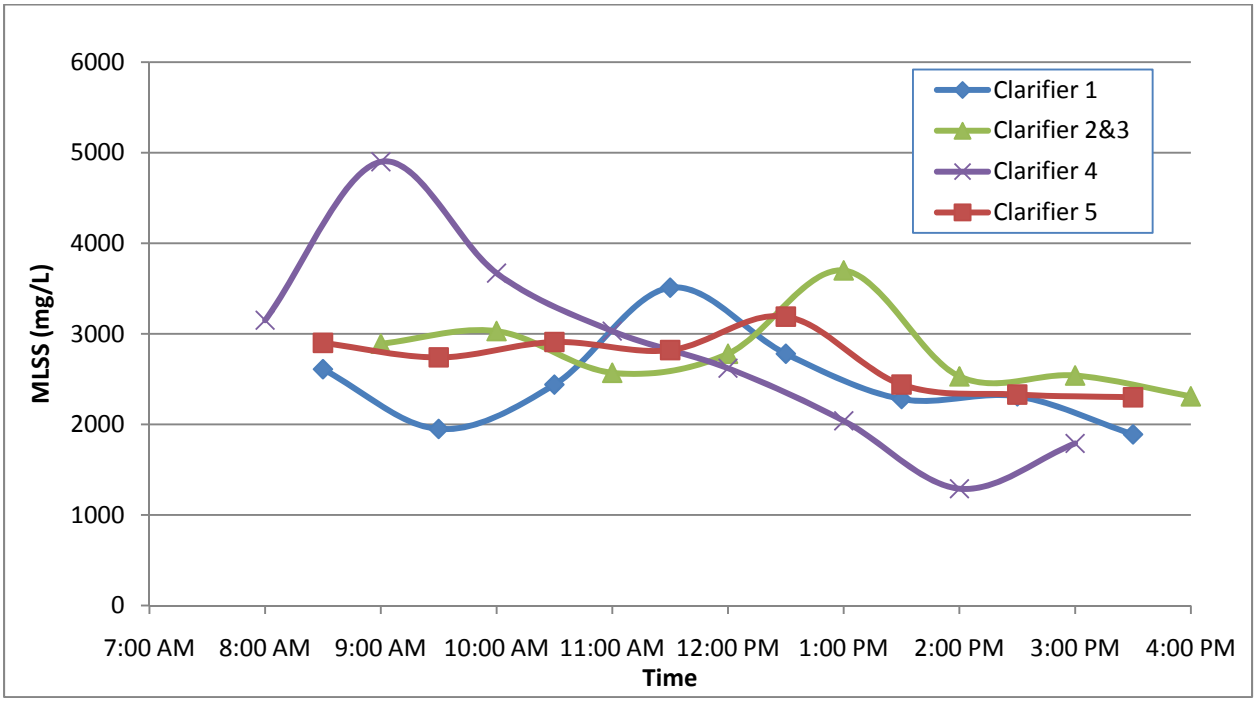


Figure 4-7
Clarifier ESS Concentrations during Stress Testing

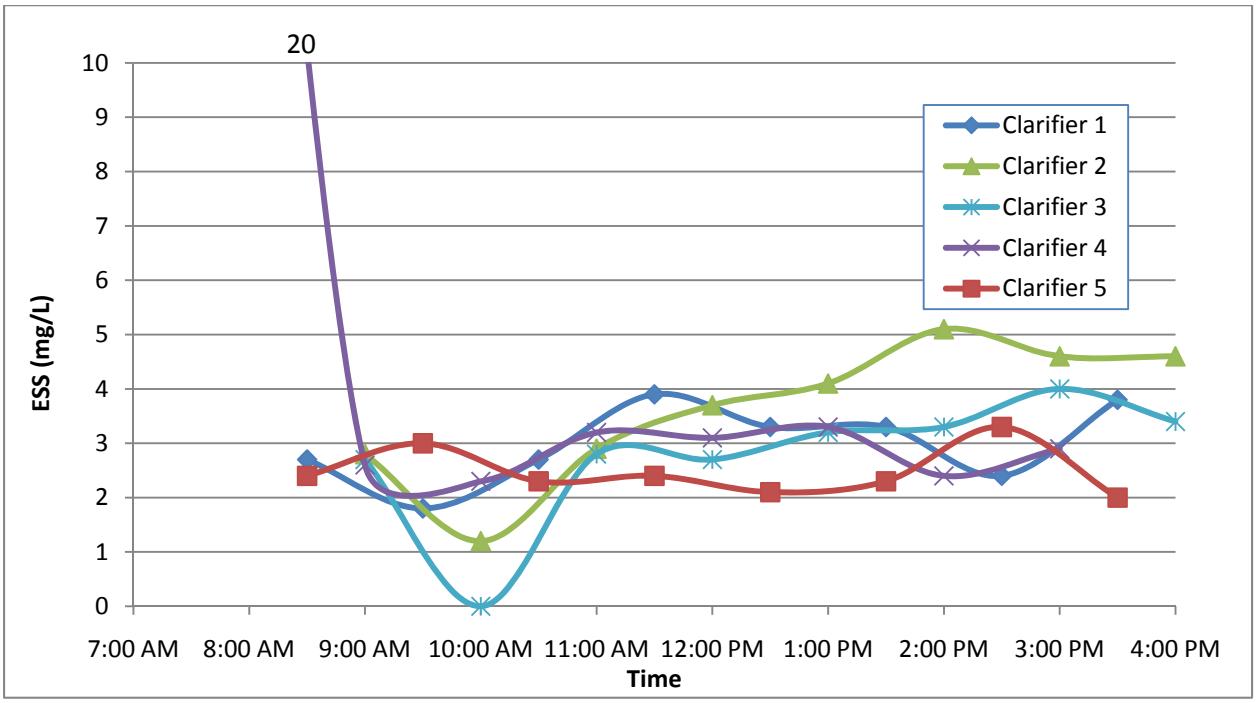
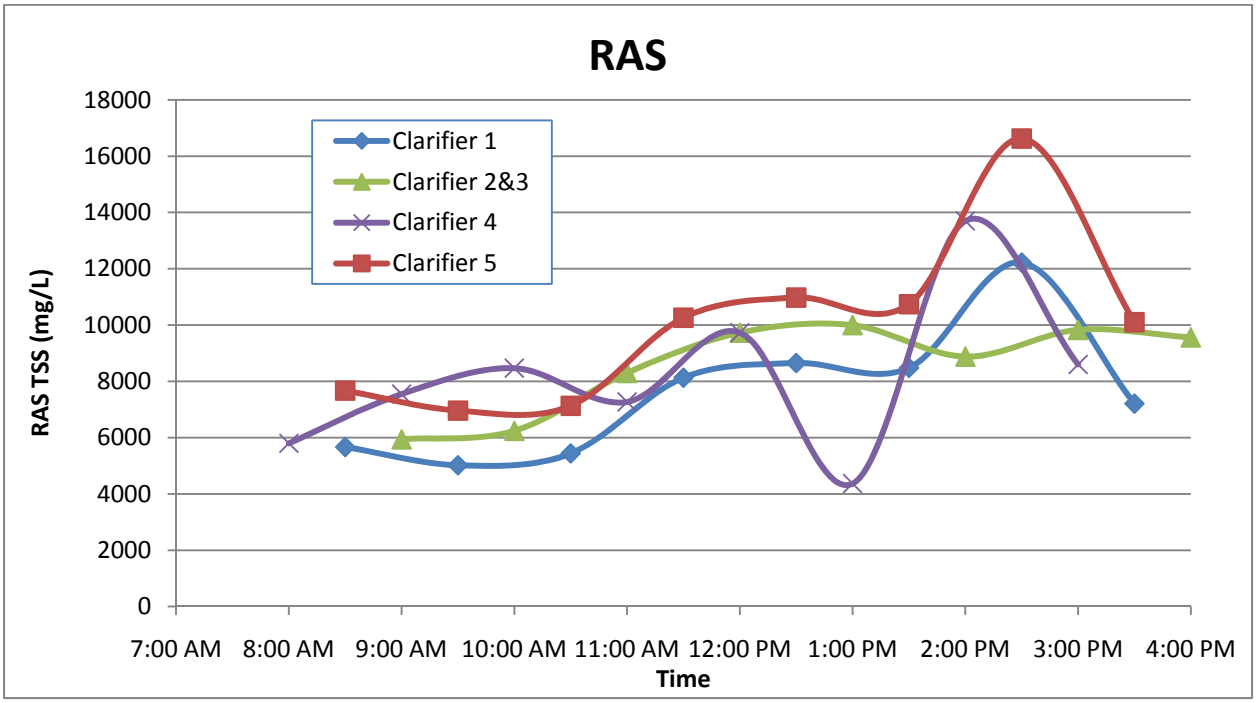


Figure 4-8
Clarifier ESS Concentrations during Stress Testing



5. SETTLING VELOCITIES

The zone settling velocity and the compression rate are both simulated in the model using exponential equations similar to the Vesilind equation. Each individual model has two settling parameters: V_o and *K* for zone settling and V_c and k_c for the compression rate.

$V_s = V_o e^{-KX}$ Hindered Threshold (*Xh*) < *X* ≤ Compression Threshold

$V_s = V_c e^{-k_c X}$ *X* > Compression Threshold

The two parameters of the exponential equations are determined in a batch settling test (column settling test). The settling velocity is measured following the procedure described in the Standard Method 2710 E for the evaluation of the Zone Settling Rate (APHA, 1999). The batch settling tests were performed using a 5-ft tall 6-inch in diameter settling column provided with a stirring mechanism to minimize the wall effect. In order to obtain the settling velocity as a function of solids concentration, the batch settling tests were conducted using different sludge concentrations each day (see Figures 3-1 to 3-21). The resulting settling velocities were subsequently utilized to estimate the settling parameter V_o and *K* from the exponential equation. In this modeling effort, the values of *Vc* and k_c were assumed equal to *Vo* and *K.* **Figures 5-1** to **5-4** show the settling velocity data collected each sampling day and the settling coefficients values are summarized in **Table 5-1**.

Table 5-1
Settling Velocities determined from Column Settling Tests

Day	1	2	3	4
Date	8/11/2009	8/13/2009	8/25/2009	8/27/2009
V_o (ft/H)	28.4	27.2	36.3	45.0
K (L/g)	0.359	0.328	0.366	0.416

Figure 5-1
Field Data and Fitted Exponential Equation for Zone Settling – Day 1

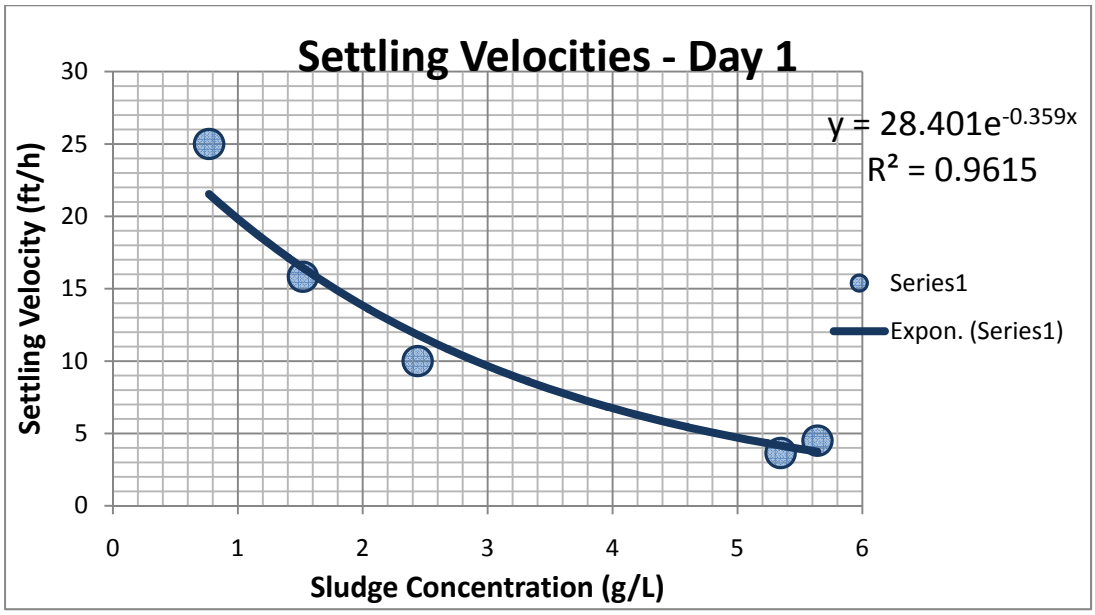


Figure 5-2
Field Data and Fitted Exponential Equation for Zone Settling – Day 2

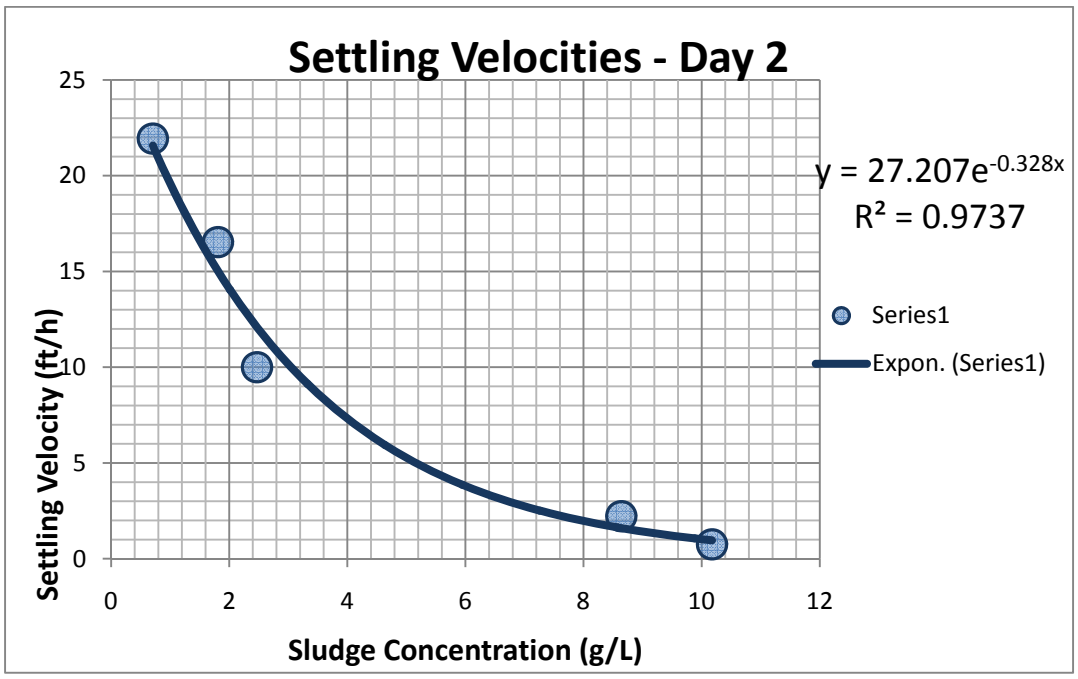


Figure 5-3
Field Data and Fitted Exponential Equation for Zone Settling – Day 3

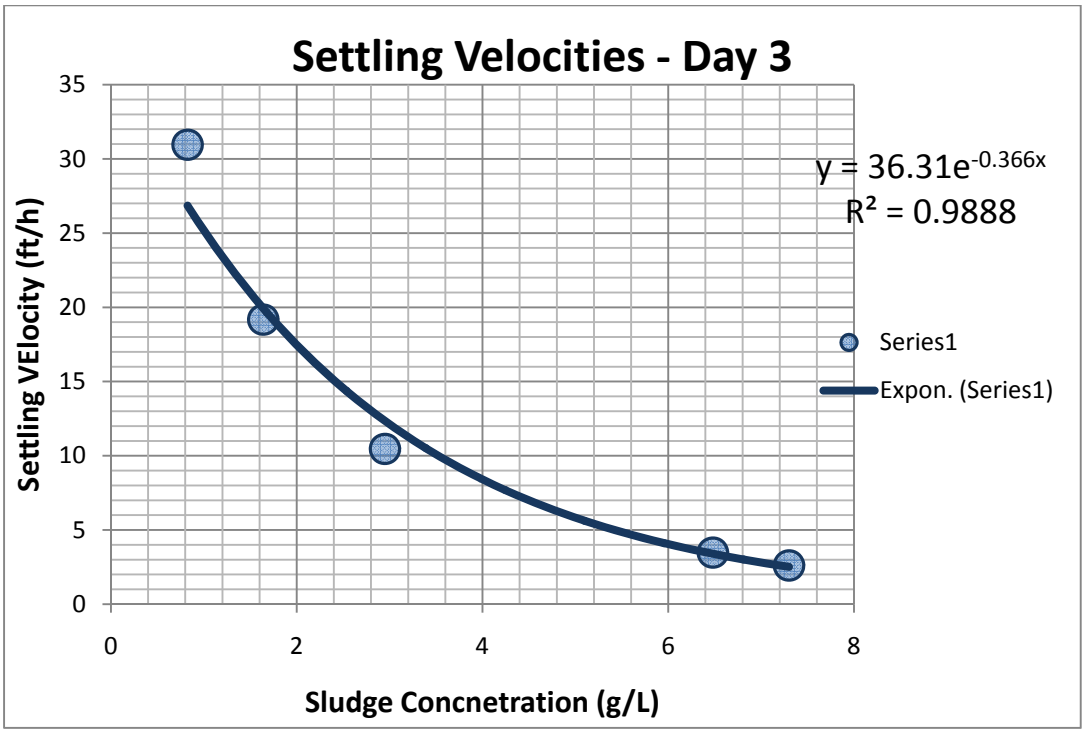
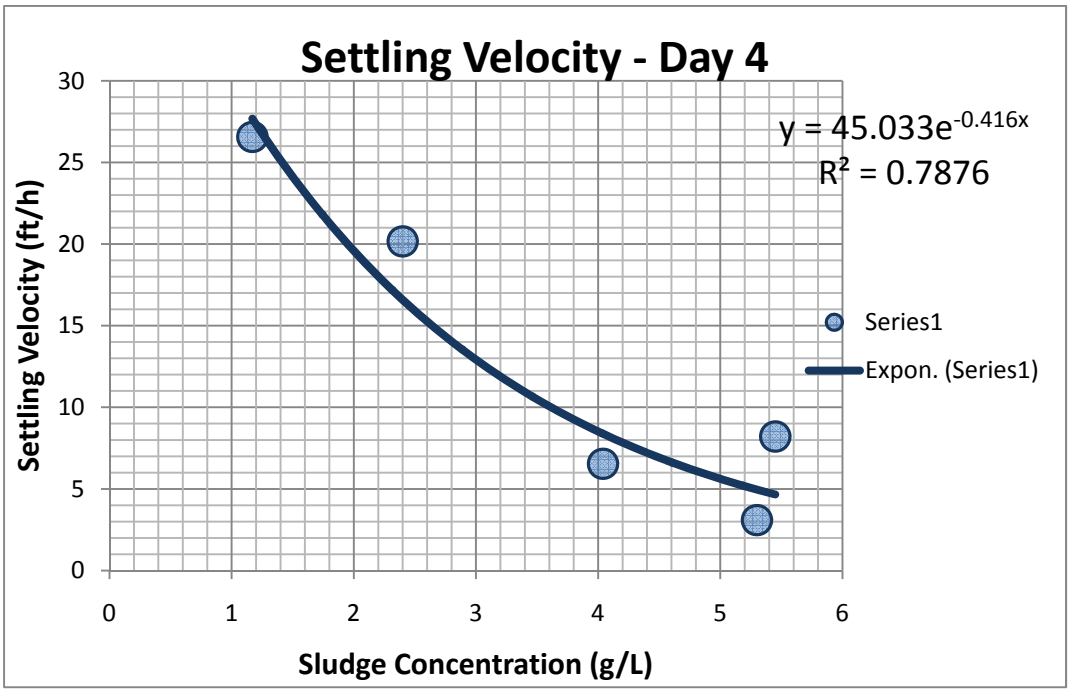


Figure 5-4
Field Data and Fitted Exponential Equation for Zone Settling – Day 4



PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Joe Rohrbacher, Alonso Griborio

DATE: October 26, 2009
Revised February 15, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum 3-E – Wet Weather Treatment Analysis

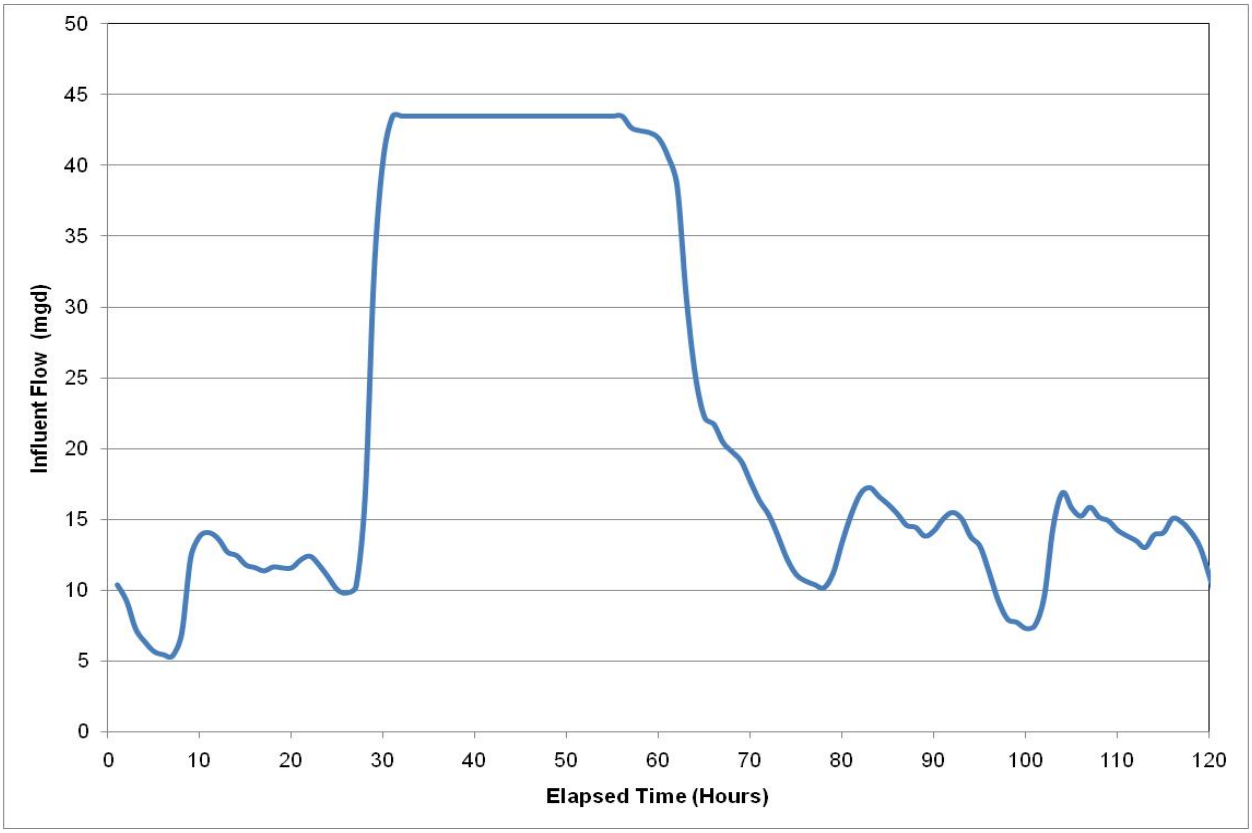
1. INTRODUCTION

It is very important to maintain nitrification and prevent solids washout during wet weather events so that effluent quality does not deteriorate. A loss of nitrification could lead to prolonged effluent quality violations as it may take the nitrifying bacteria a significant time to recover, particularly during cold weather. Solids washout will lead to increased effluent solids and associated particulate nitrogen and phosphorus and cause the loss of biomass, reducing treatment capability. A wet weather analysis of the existing secondary treatment facilities was performed to determine the capacity of the Mason Farm WWTP to process peak wet weather flows.

A design storm event was developed and presented in Technical Memorandum No. 2 and serves as the basis for the wet weather analysis of existing secondary treatment facilities (see Figure 1). BioWin version 3.0.1 process simulation software was employed to model the design storm through the biological treatment facilities. In addition, the Clarifier 2Dc computation fluid dynamics (CFD) clarifier modeling program was implemented to simulate the effects of the increased hydraulic flows and solids loading rate on the existing secondary clarifiers. Dynamic analyses were performed using these two models to determine effluent quality changes, particularly in regards to ammonia and effluent suspended solids concentrations, over the duration of a five-day period including a 24-hour sustained peak flow event. The two models were coupled to one another by applying the BioWin predicted MLSS concentrations during the design storm to the Clarifier 2Dc model to accurately represent the solids loading rates on the

secondary clarifiers.

Figure 1
Design Storm Hydrograph



2. BIOLOGICAL PROCESS EVALUATION

The Year 2030 maximum month conditions assuming current wastewater strengths were used as the initial condition for the wet weather analysis. Note that all aeration basin cells are operated aerobically under this condition. The secondary clarifier flow distribution estimated during the hydraulic evaluation provided in Technical Memorandum No. 1 was used in the wet weather analysis. Influent wastewater concentrations were adjusted to maintain a fixed load to the process through the storm event, since wet weather flows will dilute the strength of the wastewater but not necessarily reduce the overall mass load of contaminants.

Wet weather evaluations were performed for several scenarios including:

1. No solids storage assumed in the secondary clarifiers. A blanket depth of 5% of the clarifier depth (BioWin default) was used.

2. Solids storage was assumed in the secondary clarifiers to simulate the reduction in MLSS to the clarifiers observed during clarifier stress testing during peak flow events. A blanket depth of 5 feet in each clarifier was used.

3. No solids storage assumed in the secondary clarifiers. Primary effluent flows above 28 mgd were directed to Aeration Basin Cell 5.

4. Solids storage assumed in the secondary clarifiers. Primary effluent flows above 28 mgd were directed to Aeration Basin Cell 5.

In order to model the impact of the blanket depth with the ideal clarifier element, an initial steady state run assuming storage in the clarifiers was performed. The total SRT, including clarifier and NSL cell volume, was held at the same value as for the no blanket evaluations. Since a portion of the biomass is shifted to the clarifiers, the analyses where solids storage was assumed started from a lower initial MLSS concentration. It is also important to note that storing solids in the clarifier may decrease the aerobic SRT and reduce nitrification efficiency. The process model was used to assess these impacts.

Each scenario was evaluated assuming a constant RAS rate of 41% of the influent flow throughout the storm event. This RAS rate is based on the ratio of the maximum firm RAS capacity (18 mgd) to the peak hour flow (43.5 mgd).

Figure 2 presents the predicted effluent ammonia concentration for each simulation. The storage of solids in the clarifier blanket reduced the modeled aerobic retention time, which increased effluent ammonia concentrations. The model predicted that step feed of flows above 28 mgd resulted in a 36 hour period of elevated ammonia concentrations (1 to 2 mg/L), which stabilized once step feed was discontinued Step feed of flows above 28 mgd is not expected to impact compliance with the current monthly effluent ammonia limits.

Figure 2

Wet Weather Evaluation – Predicted Effluent Ammonia Concentrations

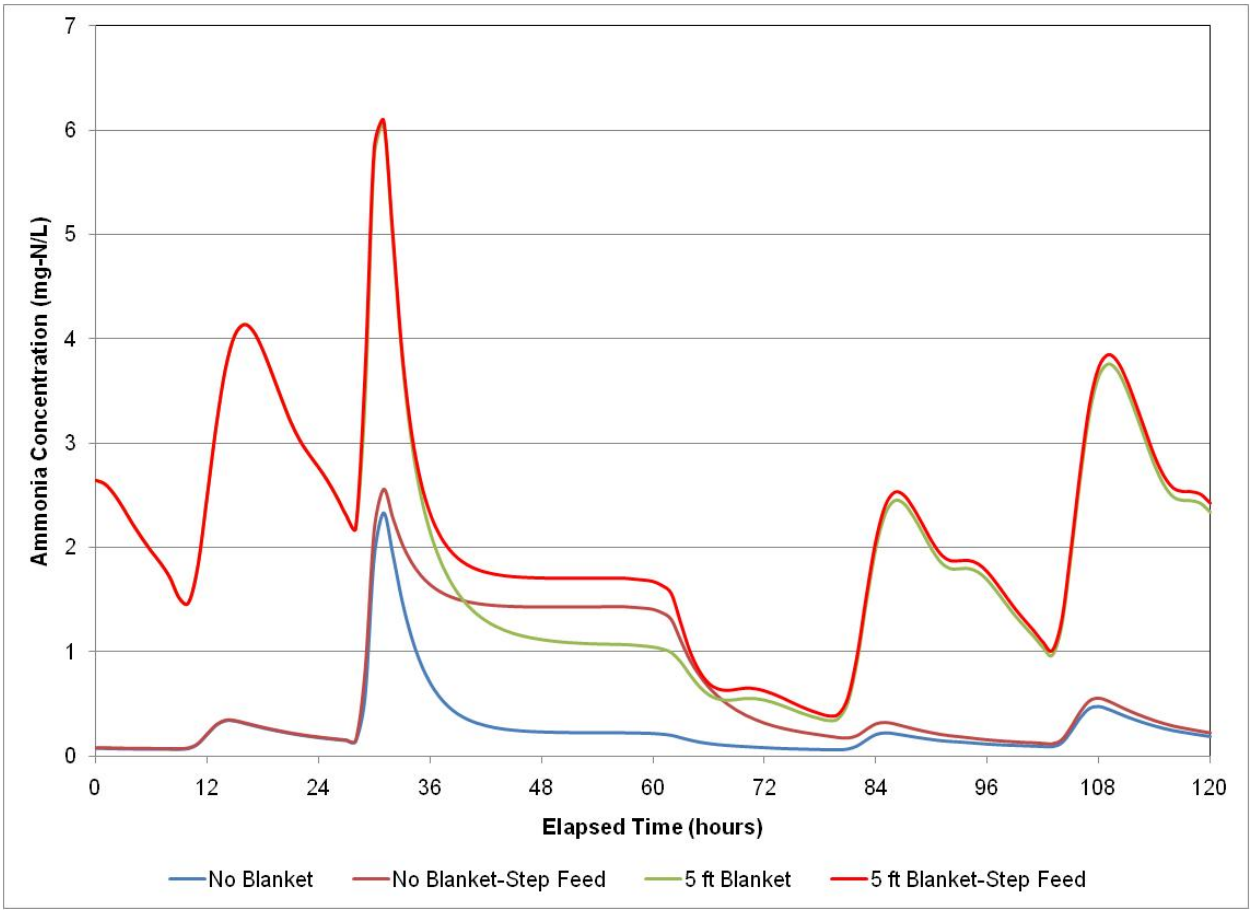
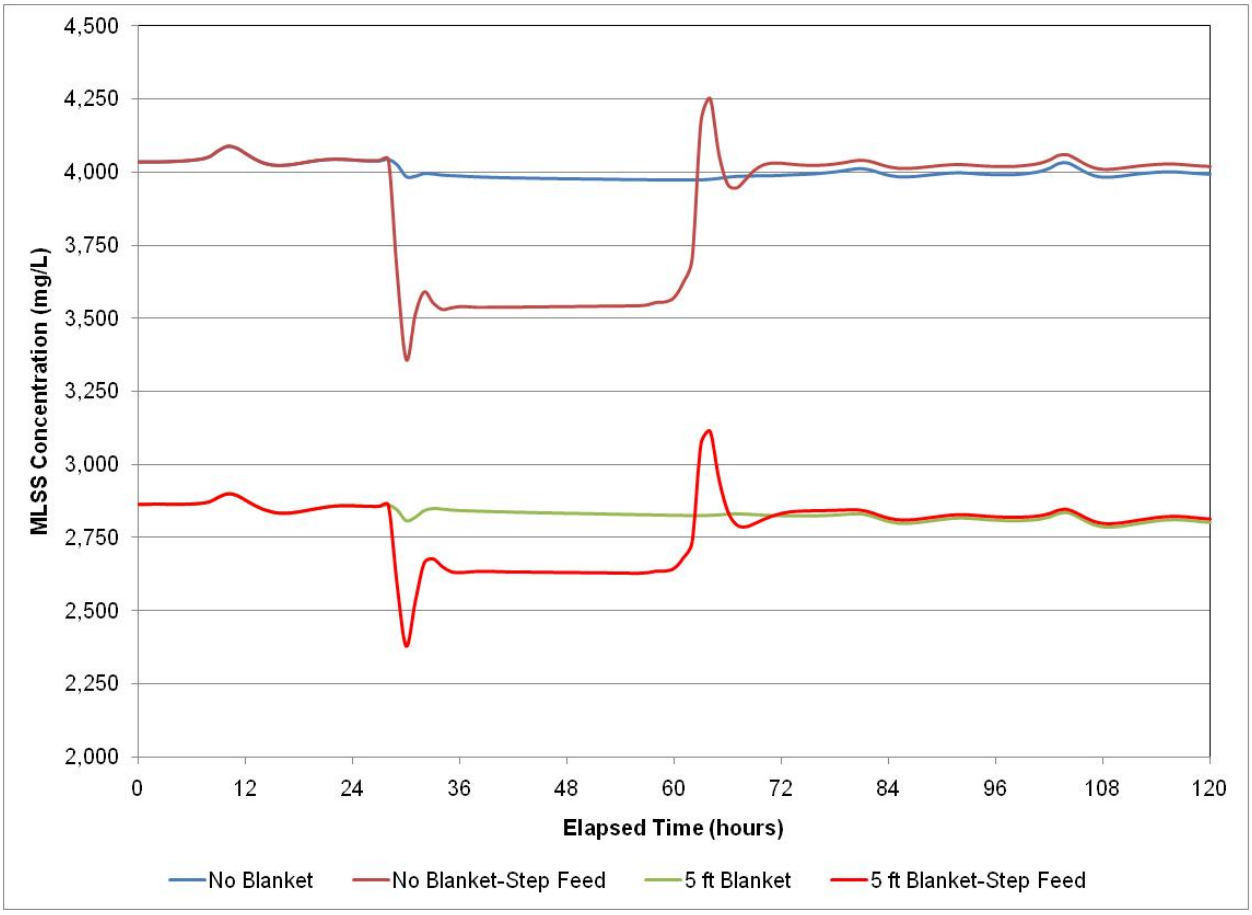


Figure 3 displays the predicted effluent MLSS concentrations through the 5-day period. Assuming solids storage in the secondary clarifiers significantly reduced the MLSS and solids loading to the clarifiers during peak flow events as observed in the field. Step feed of primary effluent also reduced secondary clarifier solids loading rates significantly as solids are stored in the upstream portion (prior to Cell 5) of the aeration basin.

Figure 3

Wet Weather Evaluation – Predicted MLSS Concentrations



3. CFD MODELING BACKGROUND

The CFD model used in this project is a quasi-three-dimensional clarifier model developed at the University of New Orleans (McCorquodale et. al. 2005, Griborio and McCorquodale 2006). The model, commonly referred as 2Dc, includes a set of governing equations that are based on the following principles: (1) continuity or conservation of fluid volume; (2) conservation of momentum; (3) conservation of mass of solids; (4) conservation of thermal energy; (5) a modified mixing length turbulence closure scheme; (6) non-Newtonian flow related to the solids ratio; (7) flocculation due to the rate of dissipation of turbulent kinetic energy, velocity gradients, differential settling and filtration; and (8) discrete, zone and compression settling.

4. MODEL SETUP AND CALIBRATION

Four different CFD models were set up for the Mason Farm WWTP secondary clarifiers: (1) Secondary Clarifier 1; (2) Secondary Clarifier 2 and 3; (3) Secondary Clarifier 4 and (4) Secondary Clarifier 5. The models were set up based on available clarifier design information and were calibrated based on field data gathered over a four day period. Initial data were collected on August 11th and 13th, 2009, and additional data were collected on August 25th and 27th, 2009. The field procedure included the simulation of peak flow conditions by stress testing of the clarifiers. The clarifiers were stress tested by taking units out of service and re-directing flow to the remaining units. In general, the field data collected for model calibration included:

- Influent flows
- RAS flows
- MLSS
- RAS suspended solids concentration
- Sludge blanket height
- Secondary clarifier effluent suspended solids
- Settling properties of the sludge: zone settling and compression rate
- Flocculation properties of the sludge
- Flocculated suspended solids (FSS) and dispersed suspended solids (DSS)
- SVI

The stress testing of the secondary clarifier and the data gathered for the calibration of the 2Dc models are summarized in Technical Memorandum No. 3 – Appendix D "Clarifier Stress Testing and 2Dc Model Calibration Data."

In order to proceed with the CFD modeling of the secondary clarifiers, some key assumptions were made. The major assumptions used to complete the CFD modeling are:

- It is assumed that the distribution of Mixed Liquor Suspended Solids (MLSS) to the secondary clarifiers in service is uniform.

- Only secondary sedimentation is modeled. The effluent suspended solids (ESS) reported in this document refers to the ESS of the secondary clarifiers (unless indicated otherwise).

- The model does not include the effects of temperature and atmospheric heat exchange on clarifier performance.

- It is assumed that the secondary clarifier suction mechanism is in good condition and is capable of retrieving the sludge at the RAS rate and sludge blanket concentration predicted by the model. It is assumed that all other structural and mechanical components of the secondary clarifiers are in good conditions.

- It is assumed that secondary clarifiers have a RAS flow capacity equal to the capacity presented in Table 3-13 in Technical Memorandum No. 3.

5. CAPACITY AND PERFORMANCE ASSESSMENT – SLUDGE VOLUME INDEX

The capacity and performance of the secondary clarifiers were evaluated for different sludge volume index (SVI) values using the MLSS results predicted in Figure 3. The core of the 2Dc settling function is the Vesilind equation (or exponential equation) which is based on two kinetics coefficients: Vo and K. These settling coefficients were measured in the field along with the SVI values. During the field visit the settleability of the sludge was good, as indicated by the SVI, Vo and K values presented in Table 1. However, is typical for a wastewater treatment plant to experience periods of poor settling conditions normally represented by higher SVIs. Since recent SVI data were not available for the facility, SVIs of 120 mL/g and 150 mL/g were selected to assess the clarifier capacity and performance under poorer settling conditions that those observed during the field visit. The Vo and K parameters for these SVIs were estimated using

published relationships. The SVI, Vo and K used for the clarifier capacity evaluation are summarized in Table 1**.**

Table 1

Settling Properties Used for Clarifier Evaluation

Description	SVI (mL/g)	Vo (ft/h)	K (L/g)
Field Measured 8/25/2009 –Good Settling	90	36.3	0.366
Assumed SVI – Average Settling(1)	120	33.9	0.445
Assumed SVI – Poor Settling(1)	150	31.2	0.544

[1]The Vo and K values were estimated using the Ekama & Marais and Wahlberg & Keinath relationships

6. CAPACITY AND PERFORMANCE ASSESSMENT - RESULTS

The secondary clarifier capacities were evaluated using the theoretical and predicted flow distribution presented in Tables 3-14 and 3-15 in Technical Memorandum No. 3 for a 43.5 mgd peak flow event. Tables 2, 3, 4 and 5 summarize the results 24 hours into the peak flow event for the theoretical flow distribution for Clarifiers 1, 2 and 3, 4 and 5, respectively. A RAS rate of 41% of the effluent flow from each clarifier is assumed.

The hydraulic modeling performed for Technical Memorandum No. 1 results indicates that Secondary Clarifiers 2 and 5 will experience higher loading conditions due to submergence of the influent flume. At a 43.5 mgd peak flow, the flow to Secondary Clarifier 2 will be 5.22 mgd instead of the theoretical flow of 4.83 mgd and the flow to Secondary Clarifier 5 will be 15.66 mgd instead of the theoretical flow of 14.5 mgd. The remaining clarifiers will experience lower flow than the theoretical flow based on flume throat widths. The simulations of Secondary Clarifiers 2 and 5 where repeated using these higher loading conditions, and the results are summarized in Tables 6 and 7.

Table 2

Secondary Clarifier 1 – Capacity Assessment Results – Theoretical Flow Distribution

Flow (mgd)	SOR (gpd/ft2)	MLSS (mg/L)	SLR (lbs/d/ft2)	SVI (mL/g)	ESS (mg/L)	RAS TSS (mg/L)	Observation
9.66	854	4,000	40.3	90	< 15	12,910	
				120	Fail	10,950	
				150	Fail	9,000	
		3,500	35.3	120	< 25	11,060	High blanket
				150	Fail	9,070	
		2,840	28.6	120	< 15	9,655	
				150	< 15	9,111	High blanket
		2,640	26.6	120	< 15	8,990	
				150	< 15	8,950	

Table 3

Secondary Clarifier 2&3 – Capacity Assessment Results – Theoretical Flow Distribution

Flow (mgd)	SOR (gpd/ft2)	MLSS (mg/L)	SLR (lbs/d/ft2)	SVI (mL/g)	ESS (mg/L)	RAS TSS (mg/L)	Observation
4.83	851	4,000	40.2	90	< 20	12,900	
				120	Fail	11,040	
				150	Fail	9,030	
		3,500	35.2	120	< 25	11,070	Very high blanket
				150	Fail	9,060	
		2,840	28.5	120	< 15	9,650	
				150	< 20	9,030	Very high blanket
		2,640	26.5	120	< 15	8,990	
				150	< 15	8,900	

Table 4

Secondary Clarifier 4 – Capacity Assessment Results – Theoretical Flow Distribution

Flow (mgd)	SOR (gpd/ft2)	MLSS (mg/L)	SLR (lbs/d/ft2)	SVI (mL/g)	ESS (mg/L)	RAS TSS (mg/L)	Observation
9.66	1,016	4,000	48.0	90	< 15	12,950	
				120	Fail	10,900	
				150	Fail	8,800	
		3,500	42.0	120	< 15	10,950	High blanket
				150	Fail	8,900	
		2,840	34.1	120	< 15	9,660	
				150	< 15	8,960	
		2,640	31.7	120	< 15	9,000	
				150	< 15	9,020	

Table 5

Secondary Clarifier 5 – Capacity Assessment Results – Theoretical Flow Distribution

Flow (mgd)	SOR (gpd/ft2)	MLSS (mg/L)	SLR (lbs/d/ft2)	SVI (mL/g)	ESS (mg/L)	RAS TSS (mg/L)	Observation
14.49	911	4,000	43.0	90	< 15	12,890	
				120	Fail	11,040	
				150	Fail	8,840	
		3,500	37.6	120	< 15	11,040	High blanket
				150	Fail	8,800	
		2,840	30.5	120	< 15	9,660	
				150	< 15	9,040	High blanket
		2,640	28.4	120	< 15	8,900	
				150	< 15	8,900	

Table 6

Secondary Clarifier 2 – Capacity Assessment Results – Predicted Flow Distribution

Flow (mgd)	SOR (gpd/ft2)	MLSS (mg/L)	SLR (lbs/d/ft2)	SVI (mL/g)	ESS (mg/L)	RAS TSS (mg/L)	Observation
5.22	920	4,000	43.4	90	< 20	12,885	Very high blanket
				120	Fail		
				150	Fail		
		3,500	38.0	90	< 15	11,870	
				120	Fail	10,810	
				150	Fail		
		2,840	30.8	120	< 15	9,650	
				150	Fail	8,810	
		2,640	28.7	120	< 15	8,990	
				150	< 20	8,780	High blanket

Table 7

Secondary Clarifier 5 – Capacity Assessment Results – Predicted Flow Distribution

Flow (mgd)	SOR (gpd/ft2)	MLSS (mg/L)	SLR (lbs/d/ft2)	SVI (mL/g)	ESS (mg/L)	RAS TSS (mg/L)	Observation
15.66	985	4,000	46.5	90	< 15	12,900	
				120	Fail		
				150	Fail		
		3,500	40.7	120	< 30	11,040	Very high blanket
				150	Fail		
		2,840	33.0	120	< 20	9,500	
				150	< 25	9,040	Very high blanket
		2,640	30.7	120	< 15	8,900	
				150	< 15	8,880	

7. Reduced Peak Flow Evaluation

Additional process and clarifier modeling was performed based on limiting peak sustained influent flows to 40 mgd. The modeling indicates that the existing clarifiers have sufficient capacity to treat a MLSS concentration up to 4,000 mg/L at a peak sustained flow of 40 mgd assuming average settling conditions. Step feed of primary effluent flows above 28 mgd under the 14.5 mgd maximum month design conditions will limit MLSS concentrations to 4,000 mg/L, and the existing secondary clarifiers have sufficient capacity up to the current permitted 14.5 mgd design flow provided that peak sustained flows to secondary treatment are limited to 40 mgd.

8. SECONDARY CLARIFIER EVALUATION SUMMARY

The results for the secondary clarifier evaluation are summarized as follows:

Theoretical Flow Distribution

- The secondary clarifiers can treat the 43.5 mgd peak flow at a MLSS concentration of 4,000 mg/L assuming a SVI of 90 mL/g (good settling).

- The MLSS concentration needs to be reduced to approximately 2,800 mg/L to effectively treat the 43.5 mgd peak flow at a SVI of 120 mL/g (average settling). Operation at a MLSS concentration of 3,500 mg/L, which can be provided through step feed operation, results in high blanket levels that could result in clarifier failure if the peak flows are sustained for more than 24 hours.

- The MLSS concentration needs to be reduced to approximately 2,600 mg/L to effectively treat the 43.5 mgd peak flow at a SVI of 150 mL/g (poor settling). The process modeling indicated a reduction in MLSS concentration to approximately 2,600 mg/L by providing a five-foot sludge blanket and step feed.

Predicted Flow Distribution

- The predicted flow distribution limits the capacity of Secondary Clarifier 2 , which

becomes the limiting unit in the secondary clarifier system.

- The secondary clarifiers can treat the 43.5 mgd peak flow at a MLSS concentration of 3,500 mg/L assuming a SVI of 90 mL/g (good settling). Operation at a MLSS concentration of 4,000 mg/L results in very high blanket levels that could result in clarifier failure if the peak flows are sustained for more than 24 hours.

- The MLSS concentration needs to be reduced to approximately 2,800 mg/L to effectively treat the 43.5 mgd peak flow at a SVI of 120 mL/g (average settling). Clarifier failure is predicted at MLSS concentrations of 3,500 mg/L for Secondary Clarifier No. 2.

- The MLSS concentration needs to be reduced to less than 2,600 mg/L to effectively treat the 43.5 mgd peak flow at a SVI of 150 mL/g (poor settling). Operation at a MLSS concentration of approximately 2,600 mg/L results in high blanket levels that could result in clarifier failure if the peak flows are sustained for more than 24 hours.

Reduced Peak Flow Evaluation

- The MLSS concentration needs to be reduced to approximately 4,000 mg/L to effectively treat a 40 mgd peak flow at a SVI of 120 mL/g (average settling).

  The existing secondary clarifiers have sufficient capacity at the current 14.5 mgd design flow provided peak sustained flows are limited to 40 mgd and solids settleability is average or better.

8. REFERENCES

Griborio, A., and McCorquodale, J.A (2006). Optimum Design of your Center Well: Use of a CFD Model to Understand the Balance between Flocculation and Improved Hydrodynamics. *Proceedings Water Environment Federation 79th Annual Conference & Exposition*, Dallas., Oct. pp. 263 - 280.

McCorquodale, J.A., Griborio, A., and Georgiou, I. (2005). A Public Domain Settling Tank Model. *Proceedings Water Environment Federation 78th Annual Conference & Exposition*, Washington, D.C., Oct. 29 – Nov. 2, pp. 2546-2561.

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Joe Rohrbacher, Paul Pitt, Sarah Lothman

DATE: January 18, 2010
Revised February 15, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum 5A– Carbon Management

1. INTRODUCTION

The final Jordan Lake nutrient rules require that the Mason Farm WWTP meet effluent annual mass limits for Total Nitrogen (TN) and Total Phosphorus (TP) equivalent to performance at concentrations of 3.0 mg/l and 0.23 mg/l, respectively, at the 14.5 mgd maximum month capacity. Supplemental carbon addition to the existing denitrification filters will be required in order to achieve meet the required TN mass load allocation. The Mason Farm WWTP currently feeds acetic acid (20% by weight) to the nutrified sludge (NSL) basin to supplement fermentate to promote biological phosphorus removal. In addition, acetic acid can be fed to the denitrification filters (when operating in denite mode) as a carbon source for denitrification. There are several chemicals that can be fed at the Mason Farm WWTP as a carbon substrate. This memorandum will evaluate the application of various supplemental carbon sources including methanol, acetic acid, and glycerin compounds to the denitrification filters at the Mason Farm WWTP.

2. SUPPLEMENTAL CARBON ALTERNATIVES FOR DENITRIFICATION

Denitrification is the reduction of nitrate and nitrite to nitrogen gas and is the reaction employed to biologically remove nitrogen from wastewater streams. In general the half reaction for denitrification is:

$$1/5\ NO_3^- + 6/5\ H^+ + e^- \rightarrow 1/10\ N_{2\ (g)} + 3/5\ H_2O$$

Denitrifying bacteria are facultative aerobes, meaning they have the capacity to grow in the presence or absence of oxygen. When available, aerobic respiration is favored because more energy is released from oxygen than nitrate when oxidizing the same electron donor. This means denitrifiers must be in an anoxic, or at least oxygen limited, environment to achieve denitrification or else aerobic respiration will be preferred.

To complete the half reaction above, an electron donor must be selected. In general, there are two types of electron donors: organic and inorganic. Organic donors include glucose, methanol, ethanol, acetate, or any other simple organic molecule. Organic compounds employ heterotrophic denitrifiers to complete the reaction. These compounds are the focus of this memorandum. Inorganic donors include hydrogen gas (H_2) and elemental sulfur (S^0). These substrates are consumed by denitrifying autotrophs, which use inorganic carbon, such as CO_2, as their carbon source. The use of inorganic electron donors is not widely used in wastewater treatment and is not discussed in this memorandum. An electron donor is required to reduce nitrate (the electron acceptor) and complete the redox reaction. All substrates used to complete this reaction are electron donors, and a carbon donor refers to an organic electron donor.

For wastewater applications, supplemental carbon donors are generally added to enhance biological nitrate removal in anoxic regions of the treatment process. Traditionally, carbon donors are added in the secondary anoxic cell of biological nutrient removal tanks or in denitrification filters, where the availability of oxygen is limited and nitrate and nitrite concentrations are relatively high. In addition, acetate has been added to anaerobic zones for enhanced biological phosphorus removal efficiency. The following carbon donors were investigated for use at the Mason Farm WWTP:

- Acetic Acid
- Methanol
- Glycerin Compounds

Other carbon donors exist but were eliminated from evaluation based on their high cost or limited availability. These products include ethanol and sugar products such as corn syrup and igh-fructose corn syrup waste products.

2.1 ACETIC ACID

Acetic acid and sodium acetate solutions can be used to provide external carbon for denitrification. Acetic acid addition is commonly provided upstream of anaerobic basins to aid in biological phosphorus removal. The Mason Farm WWTP currently feeds a 20% by weight solution of acetic acid to the NSL cells for this purpose. Acetic acid is typically available as 56 percent and 20 percent strength (by weight) solutions, whereas sodium acetate can be purchased in a dry form or as sodium acetate solution (10 to 30 percent by weight). Sodium acetate was eliminated from consideration due to its high cost in comparison to acetic acid. Chemical properties of 20 percent and 56 percent acetic acid solution are included in Table 5A-1.

Denitrification using acetic acid does not require a specialist population of organisms like methanol does. Additionally, significantly greater growth rates of denitrifying organisms have been observed with acetate in comparison to methanol (Dold, et. al., 2007 and Mokhayeri, et. al., 2006). The required COD to nitrate ratio for acetate addition was found to be 3.5 mg COD per mg NO_3-N in bench scale studies performed by Mokhayeri, et. al. (2006).

Acetic acid (56 percent solution) is classified as a high health and moderate fire hazard and is very corrosive to many materials. Exposure to the skin or eyes must be avoided. Acetic acid storage tanks for a 56 percent solution should be constructed of 316L stainless steel. Storage facilities must be carefully designed to avoid the introduction of water into the storage tank, which could cause an explosion and rupture the storage tank. Acetic acid (20 percent solution) poses less health risks than the 56 percent solution and is non-flammable. Storage tanks designed for a 20 percent solution should be constructed of stainless steel, polyethylene, or fiberglass reinforced plastic (FRP). Existing bulk storage tanks are constructed of FRP and are not suitable for storage of 56% acetic acid. Table 5A-2 summarizes the recommended materials for acetic acid feed facilities.

Estimated costs of 56 percent and 20 percent acetic acid solution are $3.17 and $0.83 per gallon, respectively. Since the majority of acetic acid is produced through the carbonylation of methanol, acetic acid is subject to the same price and supply volatility as methanol.

2.2 METHANOL

Methanol is currently the most established supplemental carbon donor used for denitrification in wastewater treatment facilities. However, methanol is a highly flammable liquid, and a recent high profile methanol explosion case, resulting in the deaths of two utility workers in Daytona Beach, Florida, has resulted in utilities reevaluating the use of methanol from a health and safety aspect. In addition, price and supply volatility have created an uncertainty in the long-term sustainability of methanol use.

Methanol is a clear, colorless, flammable liquid with a characteristic pungent odor, and is generally supplied with a purity of 99.9 percent. Table 5A-1 summarizes the chemical properties of methanol.

Methanol is typically produced through reacting natural gas with steam in the presence of a catalyst to form synthesis gases (carbon dioxide, carbon monoxide and hydrogen). The synthesis gas reacts in the presence of another catalyst to form methanol. Coal, wood, methane and other resources may also be used to generate synthesis gas for methanol production. There are ten operating methanol plants in the United States that collectively produce over 1.3 billion gallons (Methanol Institute 2010).

Methanol is typically dosed to the post-anoxic zone of four and five-stage biological nutrient removal plants and/or to tertiary deep bed denitrification filters. Normal heterotrophic bacteria, which denitrify in the pre-anoxic zones using available carbon in the influent wastewater and in post-anoxic zones using endogenous carbon, cannot utilize methanol as a carbon donor. A specialist population of methanol-utilizing-bacteria known as methylotrophs must be present to successfully denitrify using methanol addition. An acclimation period of a few weeks to months is often necessary following initial application of methanol to provide for sufficient methylotrophs for effective denitrification. Recent research indicates that these organisms are more temperature dependent than previously thought and that the methylotrophic growth rate slows considerably in cold weather. This low growth rate coupled with a strong temperature dependency underscores the significance of providing for sufficient post-anoxic solids retention times to prevent methylotroph washout (Dold, et. al., 2007).

Typical methanol feed rates are 3 to 4 lbs of methanol per lb of nitrate removed. Methanol feed is commonly flow paced, and in denitrification filters or post anoxic zones the dose is adjusted frequently according to the nitrate concentrations in filter or anoxic zone influent and effluent. Methanol feed should be carefully monitored and controlled to prevent overdosing which can cause high effluent BOD.

One advantage that methanol has over other alternative carbon sources is that it is proven in the denitrification filter addition application. Recent trials using alternate carbon sources have identified difficulties in the denitrification filter application due to high dissolved oxygen (D.O.) in the filter influent causing rapid growth of aerobic biomass in the top of the filter media and filter walls. The growth of this aerobic biomass increases filter fouling and backwash frequency. Filter fouling and increased backwash frequency due to aerobic biomass growth is not a concern with methanol. Hazen and Sawyer recently assisted the City of Raleigh, NC in piloting a glycerin product applied to denitrification filters. Results showed a large majority of the glycerin was being used by the excess influent D.O. concentration.

Methanol storage and use requires significant precautions and safety equipment. Safety issues are further addressed in Section 4. The EPA Nitrogen Control Manual recommends that storage tanks have the capacity to store methanol for two to four weeks to minimize problems due to delays in methanol delivery.

GLYCERIN COMPOUNDS

One of the latest carbon donors under investigation is glycerin compounds, many of which are byproducts from biodiesel production. Biodiesel is commonly made by chemically altering an organic oil, either vegetable oil or animal fat, through the use of a catalyst and an alcohol. The chemical reaction that occurs through this process breaks down the oil molecules and replaces the glycerin portion of the molecule with an alcohol molecule. The result is a separated solution with glycerin on the bottom and biodiesel on top. The glycerin is usually further processed to recover any residual methanol. The different glycerin compounds discussed in this section contain approximately 60 to 75 percent glycerin and 0.06 to 6 percent methanol.

Glycerin can be purchased directly from a biodiesel manufacturer or from a chemical supplier. The exact chemical composition and properties of these glycerin products are not available and will vary between batches. Brenntag Southeast, located in North Carolina, sells both a crude and a refined glycerin product. If glycerin is produced from animal fat, an acid stabilizer is added to keep the fatty acids from coming out of solution and congealing. The glycerin contains up to 5 percent acid which lowers the pH. The Brenntag glycerin fraction is typically 60 to 70 percent and the remainder is made up mostly of Material Organic Non Glycerin (MONG), ash and water. MONG consists of unidentified organic compounds that are waste products of biodiesel production.

Piedmont Biofuels is a biodiesel manufacturing facility located in Pittsboro, North Carolina, approximately 15 miles from the Mason Farm WWTP. Piedmont Biofuels is a BQ9000 certified producer; its biodiesel meets stringent standards and therefore the glycerin produced is likely to have minimal variations from batch to batch. Glycerin produced at Piedmont Biofuels has not been piloted in a wastewater treatment application.

Virginia Biodiesel, located in Virginia, sells a glycerin product made from soy beans. Glycerin produced from vegetable oil typically has a higher pH since, unlike animal fat derived glycerin, the addition of acid is not required. The Virginia Biodiesel product is used to compare the chemical composition but due to the limited supply it is not used in the price comparison.

The Washington Suburban Sanitary Commission (WSSC) investigated several crude glycerin carbon products at the full-scale level. Hazen and Sawyer performed bench-scale testing and full-scale basin profiles during the Brenntag and Virginia Biodiesel pilots. The Brenntag and Virginia Biodiesel had COD values measured at 1.2 kg/L and 1.3 kg/L, respectively. The bench-scale

testing showed a dose of 4.8 mg of COD per mg NO_3-N removed and a full scale dose of 6.5 mg of COD per mg NO_3-N removed. The full scale testing showed denitrification results in less than one day. The cold weather testing showed an increase in glycerin viscosity beyond the peristaltic pumps ability and the carbon feed was shut down. The testing also found inconsistencies between shipments from the same manufacturer and some deliveries experienced phase separation.

Hazen and Sawyer also assisted with trials at the City of Raleigh Neuse River Wastewater Treatment Plant using crude glycerin sold by Brenntag Southeast. The bench-scale testing showed a dose of 4.7 mg of COD per mg NO_3-N removed. The full scale pilot tested one train of methanol and one train of glycerin at the same time. This showed one gallon of methanol removed the same amount of nitrate as one gallon of glycerin. Similar to WSSC the shipments of glycerin had inconsistent color and viscosity.

BioCarbDN is a glycerin product being sold by Suffolk Sales and Service Corporation located in Virginia. The Henrico WRF (Henrico County, VA) has been using BioCarbDN since November 12, 2008 with good results. BioCarbDN is being stored in existing lined carbon steel tanks and the transfer pumps are being used to mix the material to prevent separation during storage. BioCarbDN is a proprietary product that contains 70 percent glycerin, less than 3 percent methanol and has a reported COD value of 0.46 kg/L to 0.60 kg/L. BioCarbDN has a slight health and flammability risk according to the NFPA classifications.

MicroC glycerin™ is proprietary glycerin product being sold by Environmental Operating Solutions (EOS) located in Baltimore, Maryland. The product contains 65 percent glycerin, has less than 0.06 percent methanol and has a reported COD value of 1.0 kg/L. MicroC glycerin™ has a slight health risk according to the NFPA classifications. Hazen and Sawyer will soon be assisting the Henrico WRF with bench scale denitrification testing of the MicroC glycerin™ product. EOS sells other proprietary chemicals for denitrification in addition to MicroC glycerin™.

The full and bench scale testing at WSSC, Neuse River and Henrico WRF with glycerin products has shown reliable and repeatable denitrification results. They have also shown that the viscosity of glycerin can drastically increase at cold temperatures. Increased viscosity at lower temperatures can cause difficulty in pumping and considerations should be made for viscosity in facility metering pump and pipe design. Prices of glycerin vary by the amount of

methanol recovery and processing of the glycerin, which impacts the glycerin and methanol content of the glycerin product.

Table 5A-1

Summary of Chemical Properties of Carbon Sources

Parameter	Methanol	20% Acetic Acid	56% Acetic Acid	BioCarbDN (Suffolk Sales)	Crude Glycerin	MicroC Glycerin (EOS)
Molecular Formula	CH_3OH	CH_3OOH	CH_3COOH	Proprietary	$C_3H_5(OH)_3$	Proprietary
Glycerin %	0	0	0	70%	75%	65%
Molecular Weight (g/mol)	32	60	60	NA	92	NA
Specific Gravity	0.79	1.00	1.06	1.18	1.24	1.21
Density (lb/gal)	6.6	8.34	8.8	9.80	10.30	10.09
Solubility in H2O	Miscible	100%	100%	Miscible	Miscible	Miscible
Freezing Point (°F)	-144	11.2	11.2	<20	32 [2]	Below 0
Viscosity (cps)	0.55	Unknown	Unknown	--	450 @ 72 F, 3500 @ 32 F	70 @ 72F, 252 @ 3F
Flashpoint (°F)	54	--	226	--	>160	No Flash
pH	NA	--	1.9	2.5	5-7	5.75- 6.75
Methanol %	99.80%	0	0	<3%	<3%	0.06%
NFPA hazard ratings: Health	1	3	3	1	1	1
NFPA hazard ratings: Fire	3	2	2	1	0	0
NFPA hazard ratings: Reactivity	0	0	0	0	0	0

Table 5A-2

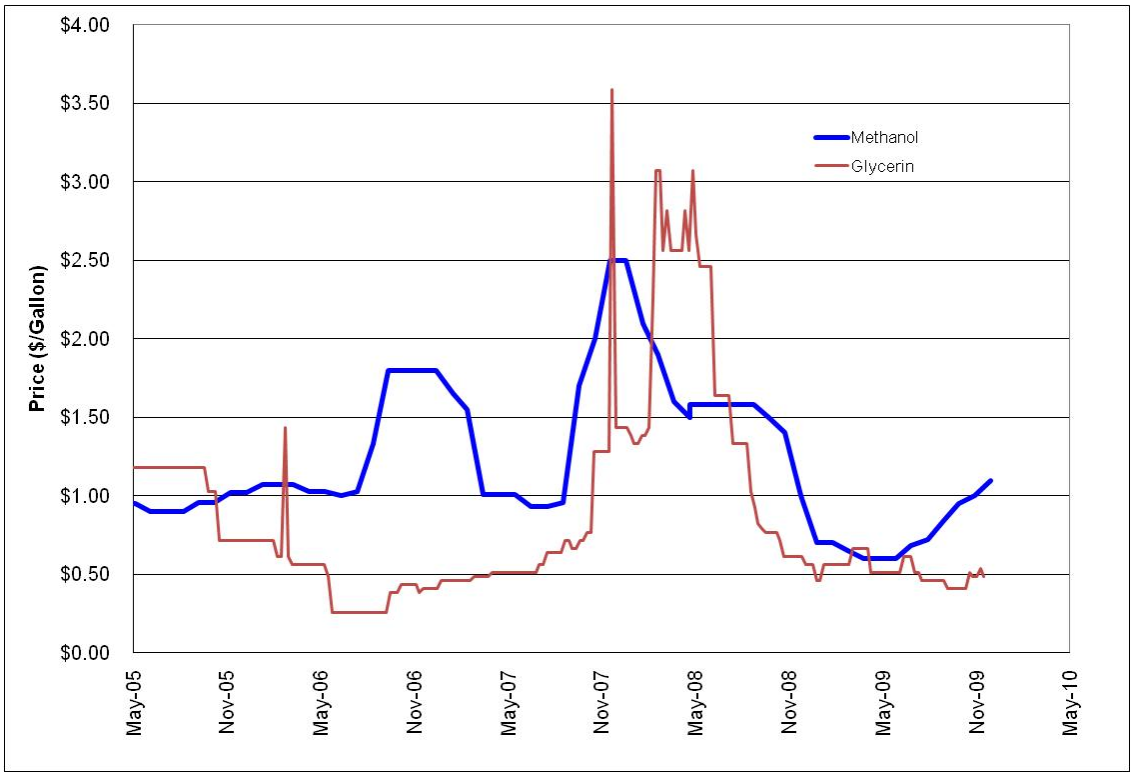
Construction Materials of Carbon Feed Facilities

Parameter	Acetic Acid	Acetic Acid	Methanol	Glycerin Compounds
% Solution	20%	56%	100%	50% to 75%
Bulk Delivery Volume (gal)	5,400	5,100	5,000 – 7,500	4,500
Storage Tank Materials	FRP, 304 SS	316L SS	Carbon Steel or 304 SS	304 SS
Tank Heat Tracing or Insulation	No	Both	No	Yes but not to exceed 120F
Piping Materials	304 SS	316L SS	Carbon Steel or 304 SST	304 SS
Piping heat tracing or insulation Required?	No	Yes	No	Yes but not to exceed 120F
Feed Pump Type	Diaphragm, Centrifugal	Diaphragm, Centrifugal	Diaphragm	Gear, Progressing Cavity
Feed Pump Materials	316 SS	316 SS	316 SS	316 SS
Safety Equipment Required	Alcohol Type Film Forming Foam, Secondary Containment	Alcohol Type Film Forming Foam, Secondary Containment	Alcohol Resistant Foam, Standard Sprinklers; Secondary Containment	Carbon dioxide, Dry Chemical, Foam or Water spray (FOG or fine spray), Secondary Containment

Extreme variations in methanol price have been observed in the last several years. The current price for methanol is $1.20 per gallon (December 2009). Figure 5A-1 shows the historical price fluctuations of methanol and glycerin since May 2005.

Figure 5A-1

Price Fluctuations– April 2001 through December 2009



3. EXISTING CARBON FEED FACILITY

Mason Farm WWTP currently feeds a 20 percent by weight acetic acid solution. The solution is stored in 4 tanks constructed of fiberglass reinforced plastic (FRP). Details of the existing tanks are presented in Table 5A-3. Each tank holds 20,000 gallons for a total capacity of 80,000 gallons. The pipes that fill the storage tanks are heat traced; however, the tanks and the piping from the tank to the feed pumps are not heat traced. These tanks do not have appurtenances such as flame arrestors that facilitate storage of chemicals with flammability hazards. Methanol cannot be stored in the existing tanks. Storage of glycerin in the existing tanks may be feasible. However, glycerin products have a significantly higher density that acetic acid and a structural evaluation would be required before storage of glycerin in these tanks could be considered.

Table 5A-3

Existing Acetic Acid Storage Facilities

Number	4
Volume per Tank (gallons)	20,000
Total Volume (gallons)	80,000
Tank Material	FRP
Heat Tracing	Yes - Tank Fill Line No – Pump Suction Line
Safety Equipment (flame arrestors, vapor recovery, etc.)	No

The present plant configuration allows for addition of acetic acid to both the NSL Cells and the Filters. Two Pulsafeeder Pulsatron solenoid pumps feed the acetic acid to the NSL cells. Additionally, two Pulsafeeder Pulsar diaphragm pumps can feed acetic acid solution to the filter influent channel to provide a carbon for denitrification in the filters. Table 5A-4 summarizes the existing acetic acid feed pumps.

Table 5A-4

Existing Acetic Acid Feed Pumps

NSL Feed Pumps	
Number	2
Type	Solenoid
Capacity (gpd)	500
Filter Feed Pumps	
Number	2
Type	Diaphragm
Capacity (gpm)	240

The existing solenoid pumps are not recommended for feed of glycerin or methanol. Glycerin is a highly viscous chemical that is more easily handled by gear pumps or diaphragm metering pumps (up to viscosities around 200 centipoise). Methanol, due to its flammable nature, must be pumped with explosion-proof equipment. New pumps would be required at the Mason Farm WWTP in order to feed a carbon source other than acetic acid.

4. SAFETY CONSIDERATIONS

4.1 METHANOL

Methanol has several safety concerns since it is a Class 1B flammable liquid fire hazard with a flash point of 54^{o}F. Methanol is rated by the National Fire Protection Association (NFPA) as a

slight health hazard and high flammability hazard and is classified as an irritant. Methanol poisoning is usually caused by ingestion; however, inhalation of methanol vapors and prolonged or repeated skin contact are also harmful. Methanol is a hazardous material according to Resource Conservation and Recovery Act (RCRA) regulations and must be disposed of in accordance with applicable local, state, and federal regulations.

It is recommended that methanol storage and feed facilities should be located outside to minimize the possibility of explosions due to methanol fumes. For the same reason, these facilities are also located at a minimum distance of 30 feet from other structures. All equipment in methanol storage and feed areas should be explosion proof. The chemical unloading facilities should not be located in a heavy traffic area and should have the ability to ground the offloading truck. Methanol tanks must be grounded, and secondary containment should be provided to hold the contents of the largest tank in the containment area.

Methanol, being a polar solvent, can be successfully extinguished with special alcohol resistant foam concentrates and standard sprinklers. When pumps or feed equipment is located indoors, a foam sprinkler system must be provided. The foam sprinkler system needs to be approved for methanol application per NFPA regulations and be acceptable to the authority having jurisdiction. A reliable water and foam concentrate supply must be available in pressure and quantity to meet the fire demands indicated by the specific hazards of methanol-processing operations, storage, or exposure. Water supplied to foam-water systems must be compatible with the foam concentrate to be used. Water that contains solids likely to clog orifices in discharge devices but that is otherwise acceptable for making foam may be permitted to be used after passing through line strainers.

Foam chambers and hose lines should be provided based on the diameter of the methanol storage tank as required by standards (NFPA 11 Index B, Table B1). For liquid depths greater than 1 inch, monitor and foam hose streams should be limited for use with alcohol resistant foams approved for methanol. Where monitors and handline nozzles are used to protect tanks the operation time shall be 65 minutes at listed application rates, unless the foam manufacturer has established, by fire test, that a shorter time can be permitted.

The design and materials of construction for foam concentrate pumps shall be approved for use with the type of foam concentrate used in the system. Special attention should be paid to the

type of seal or packing used. Foam concentrate pumps shall have rated capacities at or in excess of the maximum system demand.

4.2 ACETIC ACID

Acetic acid (56 percent solution) is classified as a high health and moderate fire hazard and is very corrosive to many materials, as mentioned previously. Exposure to the skin or eyes must be avoided. Storage facilities for acetic acid must be carefully designed to avoid the introduction of water into the storage tank, which could cause an explosion and rupture the storage tank.

4.3 GLYCERIN

The glycerin products are a slight health and fire hazard, mainly due to the small methanol content. There are minimal provisions that must be made in order to design and build a glycerin facility with proper safety equipment. Since glycerin products for wastewater applications are byproducts of biodiesel production, product consistency can significantly vary. It is important that, since there is limited experience with feeding glycerin to deep bed filters, OWASA pilot a chosen glycerin product before further consideration is given to capital investments for glycerin storage and feed.

5. COMPARISON OF CARBON ALTERNATIVES AND COST EVALUATION

The cost per pound of nitrate removed for each of the supplemental carbon donors was calculated as shown in Table 5A-5 below. This provides a common set of units to compare the alternatives. The COD required per pound of nitrate removed varies for each product since a portion of the COD is used for biomass synthesis, and each of the donors evaluated has a different anoxic biomass yield.

Table 5A-5
Chemical Comparison

Parameter	Methanol	Acetic Acid (20%)	Acetic Acid (56%)	BioCarbDN (Suffolk Sales)	Crude Glycerin	MicroC Glycerin (EOS)
COD:NO_3-N	4.8	3.5	3.5	5.8 [1]	4.8 [3]	4.8 [2]
COD:Substrate	1.50	1.07	1.07	0.60	0.99	0.83
Substrate:NO_3-N	3.2	3.3	3.3	9.7	4.8	5.8
Strength (%/w)	100%	20%	56%	100%	100%	100%
Density (lb/gal)	6.60	8.34	8.80	9.80	10.30	10.09
Cost ($/gallon)[6]	$1.20	$0.83	$3.17	$1.64	$3.59	$1.60
Cost ($/lb NO_3-N)	$0.58	$1.61	$2.16	$1.14	$1.59	$0.92

1 Based on Henrico WRF bench tests

2 Assumed to be the same as crude glycerin

3 Based on Parkway WWTP bench tests

Methanol has the lowest cost per pound nitrate removed of the supplemental carbon donors evaluated. Methanol costs recently increased but are historically around $1 per gallon. Twenty-percent acetic acid costs are nearly three times that of methanol on a nitrate removal basis. The glycerin products have a large price distribution from 1.5 to 2.75 times the price of methanol based on the price of nitrate removed.

Acetic acid feed requirements were developed using BioWin 3.0 process simulation software. The acetic acid feed rate presented in Table 5A-6 is based on feeding carbon to the recommended step feed configuration at design annual average conditions and was used to estimate feed rates for the alternative carbon donors. Table 5A-6 smmarizes the annual average feed requirements and annual costs for each of the supplemental carbon donors evaluated.

Table 5A-6

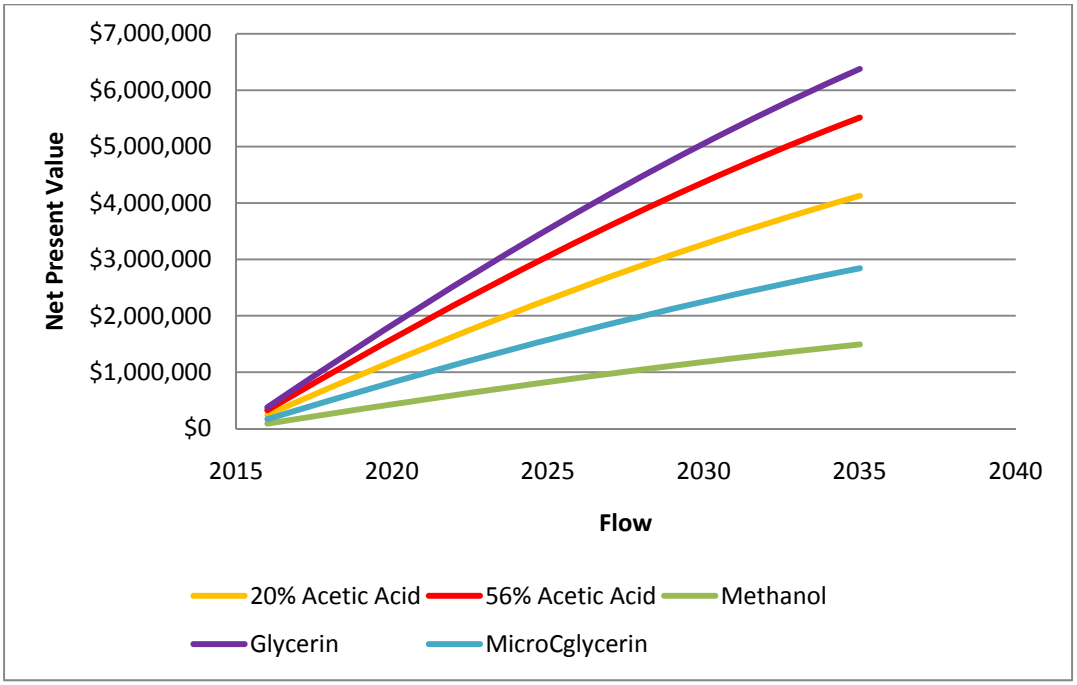
Estimated Chemical Feed Rates and Annual Costs

Carbon Source	Annual Average Dose to Filters (gpd)	Cost/Gallon	Relative Annual Cost
Acetic Acid (20%)	1,600	$0.83	$485,000
Acetic Acid (56%)	560	$3.17	$648,000
Methanol	400	$1.20	$175,000
Glycerin	570	$3.59	$747,000
MicroC glycerin (EOS)	570	$1.60	$333,000

Figure 5A-2 presents the cumulative net present value for each of the evaluated carbon donors over the 20-year period from 2016 to 2035 when annual average flows are projected to increase from 8.16 mgd to 11.03 mgd. As shown in the figure, conversion to feeding methanol will result in immediate chemical cost savings.

Figure 5A-2

Cumulative Net Present Value for Supplemental Carbon Sources



6. CONCLUSIONS

Based on this evaluation and current market pricing, methanol is currently the low cost option for the Mason Farm WWTP. A present worth analysis was conducted in order to compare maintaining acetic acid feed for denitrification to construction of a new methanol feed facility. Table 5A-7 presents the new present worth of the chemicals costs over a 20-year period assuming a discount rate of 5 percent.

Table 5A-7

Net Present Worth of Chemical Costs

Carbon Source	Annual Chemical Cost ($)	PW Chemical Cost ($ million)	Capital Improvement Costs ($ million)	Total NPW ($ million)
Acetic Acid (20%)	$485,000	6.0	0	6.0
Acetic Acid (56%)	$648,000	8.1	1.0	9.1
Methanol	$175,000	2.2	1.0	3.2
Glycerin	$747,000	9.3	0.5	9.8
MicroC glycerin (EOS)	$333,000	4.1	0.5	4.6

There is the potential for the plant to experience significant operational savings by using methanol as a supplemental carbon source over acetic acid. The difference in the net present chemical cost of using methanol over 20% acetic acid is $3.8 million. A methanol facility could be constructed on site for less than $1.5 million, resulting in savings of over $2.3 million. While there are safety concerns regarding the storage and handling of methanol, proper design considerations and training would ensure that responsible use of the chemical is achieved.

Flexibility could also be incorporated into the design of the methanol facility in order to allow the use of a glycerin product if desired. The acetic acid facility should be retained in order to continue to exclusively feed chemical for enhanced biological phosphorus removal.

NCDENR
North Carolina Department of Environment and Natural Resources
Division of Water Quality
Coleen H. Sullins
Director

Beverly Eaves Perdue
Governor

Dee Freeman
Secretary

RECEIVED
NOV - 9 2009

November 4, 2009

Mr. Ed Kerwin, Executive Director
Orange Water And Sewer Authority
PO Box 366
Carrboro, North Carolina 27510-0366

Subject: Draft NPDES Permit Modification
Permit NC0025241
Mason Farm WWTP
Orange County

Dear Mr. Kerwin:

By her letter of October 1, the Director notified affected dischargers of the Division's intent to modify their NPDES permits to incorporate new requirements under the Jordan Lake Nutrient Management Strategy and its Wastewater Discharge rule, T15A NCAC 02B .0270. Accordingly, the Division proposes to modify the subject permit and is enclosing the draft modifications for your review and comment.

At this time, the Division is submitting a notice to the newspaper of general circulation in Orange County, inviting public comment on the draft permits for the affected facilities. The notice is scheduled for publication on or about November 6 and initiates a 30-day comment period.

Please provide any comments you have regarding the draft permit to this office, in writing, no later than 30 days of receipt of this letter.

The draft permit includes the following changes:

- Total Phosphorus (TP) Limit. The total phosphorus limit has been revised to reflect the allocation allowed under the point source rule. This annual mass limit becomes effective January 1, 2010. The current concentration limits remain in effect through December 31, 2009.
- Total Nitrogen (TN) Limit. The permit does not affect the facility's total nitrogen limit at this time. Under the Jordan Lake Wastewater rule, a new nitrogen limit will become effective in 2016, and the Division proposes to include that limit during the 2011 permit renewal process.
- Nitrogen Optimization. The permit includes a new special condition that re-iterates nitrogen optimization requirements from the point source rule. The rule requires development of an optimization plan that, upon acceptance by the Division, is to be implemented until the nitrogen limits become effective in 2016. It also requires annual status reports until 2016 (the final report will cover calendar year 2015).

1617 Mail Service Center, Raleigh, North Carolina 27699-1617
Location: 512 N. Salisbury St. Raleigh, North Carolina 27604
Phone: 919-807-6300 \ FAX: 919-807-6492 \ Customer Service: 1-877-623-6748
Internet: www.ncwaterquality.org
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Some members of the Jordan Partnership undertook a similar effort in 1999. Regardless of its success at individual facilities, most facilities have seen significant changes in their processes or operating conditions since then. The Division does not expect any discharger to conduct an extensive evaluation if it merely repeats the 1999 effort. However, we expect all dischargers to make a reasonable effort to reduce nitrogen discharges until process improvements are completed, as is the intent of the rule.

To satisfy this requirement, each discharger should evaluate its treatment facilities and operations or update its previous evaluation(s) and report its findings and recommendations, including, if necessary, an update of its current optimization measures. All facilities will be expected to provide annual reports on the status and effectiveness of the recommended measures.

- Additional Special Conditions for Nutrients. The permit includes new conditions that (1) document the nitrogen and phosphorus allocations assigned to the facility and (2) describe how the mass limits will be managed in individual permits and in the event that a group compliance association is formed. These conditions replace and expand upon Special Conditions A.(4.) through A.(7.) in the current permit.
- Additional Changes. Now that the expansion to 14.5 MGD has been completed, requirements for the 12.0 MGD discharge no longer apply. The effluent limits page and chronic toxicity requirements for 12.0 MGD have been deleted. Conductivity monitoring in condition A.(1.) was corrected to a Grab sample. The special conditions have been re-ordered to put all of the nutrient-related conditions together for easier reference.

If you have any questions concerning the draft permit, please contact me at (919) 807-6402 or at *mike.templeton@ncdenr.gov*.

Sincerely,

Michael E. Templeton, P.E.
Point Source Branch

Enclosure: Draft permit modification, NC0025241

Copies: Chief, NPDES and Biosolids Permits Section, Region IV, EPA
Raleigh Regional Office, Surface Water Protection
Michael Douglas, Raleigh Regional Office, DEH
Steve Reid, Technical Assistance and Certification Unit
PERCS Unit
NPDES Files

STATE OF NORTH CAROLINA
DEPARTMENT OF ENVIRONMENT AND NATURAL RESOURCES
DIVISION OF WATER QUALITY

PERMIT

TO DISCHARGE WASTEWATER UNDER THE

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM

In compliance with the provisions of North Carolina General Statute 143-215.1, other lawful standards and regulations promulgated and adopted by the North Carolina Environmental Management Commission, and the Federal Water Pollution Control Act, as amended, the

Orange Water and Sewer Authority

is hereby authorized to discharge wastewater from a facility located at the

Mason Farm WWTP
100 Old Mason Farm Road
Chapel Hill
Orange County

to receiving waters designated as Morgan Creek in the Cape Fear River Basin

in accordance with effluent limitations, monitoring requirements, and other conditions set forth in Parts I, II, III, and IV hereof.

The permit shall become effective ..... _____.

This permit and the authorization to discharge shall expire at midnight on ..... July 31, 2011.

Signed this day ..... _____.

DRAFT

Coleen H. Sullins, Director
Division of Water Quality
By Authority of the Environmental Management Commission

SUPPLEMENT TO PERMIT COVER SHEET

All previous NPDES Permits issued to this facility, whether for operation or discharge are hereby revoked. As of this permit issuance, any previously issued permit bearing this number is no longer effective. Therefore, the exclusive authority to operate and discharge from this facility arises under the permit conditions, requirements, terms, and provisions included herein.

Orange Water and Sewer Authority

is hereby authorized to:

1. Continue to operate an existing 14.5 MGD wastewater treatment facility located at the Mason Farm Wastewater Treatment Plant [100 Old Mason Farm Road, Chapel Hill] in Orange County. This facility discharges via outfall 001 and includes the following wastewater treatment components:
   - Influent screen
   - Grit removal
   - Primary clarifiers
   - Trickling filters
   - Aeration basins
   - Secondary clarifiers
   - Primary sludge fermenter
   - UV disinfection system
   - Anaerobic digester
   - Gravity belt thickeners
   - Sludge holding tank; and

2. Upon receipt of a wastewater reuse permit from the Non-Discharge Permitting Unit of the Division of Water Quality construct and operate a water reclamation and distribution system to provide beneficial reuse of treated effluent from the treatment plant, as approved and conditioned pursuant to the reuse permit, and

3. Discharge from said treatment works into Morgan Creek, a class WS-IV NSW water in the Cape Fear River Basin, at the location specified on the attached map.

A.(1.) EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS (14.5 MGD)

Beginning upon the effective date of this permit and lasting until expiration, the Permittee is authorized to discharge treated wastewater from Outfall 001. Such discharges shall be limited and monitored by the Permittee as specified below:

EFFLUENT CHARACTERISTICS	EFFLUENT LIMITATIONS			MONITORING REQUIREMENTS		
	Monthly Average	Weekly Average	Daily Maximum	Measurement Frequency	Sample Type	Sample Location
Flow	14.5 MGD			Continuous	Recording	Influent or Effluent
Total Monthly Flow (MG)	Monitor & Report			Monthly	Rec. or Calc.	Influent or Effluent
CBOD, 5-day, (20°C) [1] (April 1 – October 31)	4.0 mg/L	6.0 mg/L		Daily	Composite	Influent & Effluent
CBOD, 5-day, (20°C) [1] (November 1 – March 31)	8.0 mg/L	12.0 mg/L		Daily	Composite	Influent & Effluent
Total Suspended Solids [1]	30.0 mg/L	45.0 mg/L		Daily	Composite	Influent & Effluent
NH_3 as N (April 1-October 31)	1.0 mg/L	3.0 mg/L		Daily	Composite	Effluent
NH_3 as N (November 1-March 31)	2.0 mg/L	6.0 mg/L		Daily	Composite	Effluent
Temperature				Daily	Grab	Effluent
Conductivity				Daily	Grab	Effluent
pH [2]				Daily	Grab	Effluent
Total Residual Chlorine			18 µg/L	Daily	Grab	Effluent
Fecal Coliform (geometric mean)	200/100mL	400/100mL		Daily	Grab	Effluent
Dissolved Oxygen [3]				Daily	Grab	Effluent
TKN	Monitor & Report (mg/L)			Weekly	Composite	Effluent
NO_3-N + NO_2-N	Monitor & Report (mg/L)			Weekly	Composite	Effluent
Total Nitrogen, TN	Monitor & Report (mg/L)			Weekly	Composite	Effluent
TN Load [4,5]	Monitor & Report (lb/mo) 409,448 lb/yr			Monthly Annually	Calculated	Effluent
Total Phosphorus, TP	Monitor & Report			Weekly	Composite	Effluent
TP Load [4,5]	Monitor & Report (lb/mo) 11,030 lb/yr (effective thru 12/31/2009) 10,188 lb/yr (effective 1/1/2010)			Monthly Annually Annually	Calculated	Effluent
Total Copper				2/Month	Composite	Effluent
Total Zinc				2/Month	Composite	Effluent
Total Cyanide [6]				Quarterly	Composite	Effluent
Total Mercury [6]				Quarterly	Composite	Effluent
Chronic Toxicity [7]				Quarterly	Composite	Effluent

All footnotes are listed on the following page.

Footnotes:

1. The monthly average effluent CBOD5 and TSS concentrations shall not exceed 15% of the respective influent value (85% removal).
2. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.
3. The daily average effluent dissolved oxygen concentration shall not be less than 6.0 mg/L.
4. See Special Condition A. (8.), Calculation of Total Nitrogen or Total Phosphorus Loads.
5. Compliance with mass limits shall be determined in accordance with Special Condition A.(7.), Annual Limits for Total Nitrogen or Total Phosphorus.
6. Monitoring shall be conducted for two years. If reasonable potential to exceed applicable water quality standards exists, the limit will be added to the permit.
7. Chronic Toxicity (Ceriodaphnia) at 90 %; February, May, August and November; refer to condition A. (4.).

There shall be no discharge of floating solids or visible foam in other than trace amounts.

Instream monitoring shall be conducted as specified in Special Condition A. (2.).

A.(2.) INSTREAM MONITORING REQUIREMENTS

Sample Location[1]	Measurement Frequency		Parameters[2]
UP1 DN1 DN2	October - May	Weekly	Temperature Dissolved Oxygen Conductivity Fecal Coliform
	June - September	3/week	
UP1 DN1 DN2	June-September	Once every 2 weeks[3]	Total Phosphorous PO_4 NH_3 Total Kjendahl Nitrogen NOx
DN3 DN4	April-October	Monthly[3]	Temperature Dissolved Oxygen Conductivity Fecal Coliform Secchi depth Chlorophyl-a

Notes:

1. Sample locations:
   UP1 – Upstream location at Morgan Creek at the bridge at the plant entrance
   DN1 – Downstream location #1 at Morgan Creek at the cement ford downstream of outfall
   DN2 – Downstream location #2 at Morgan Creek at the Farrington Road bridge
   DN3 – Downstream location # 3 at Morgan Creek at the mouth of Morgan Creek
   DN4 - Downstream Location # 4 at Jordan Lake at the old railroad grade
   - Sampling of temperature and DO at the surface and one-meter intervals to a depth equal to the Secchi depth. If the Secchi depth is less than 1.0 meters, sample at 0.5-meter intervals.
   - As a participant in the Upper Cape Fear River Basin Association, the instream monitoring requirements as stated above are waived. Should your membership in the association be terminated, you shall notify the Division immediately and the instream monitoring requirements specified in your permit shall be reinstated.
2. Upstream and downstream samples shall be grab samples.
3. Monthly sampling at DN3 and DN4 should be separated at least two weeks within the appropriate month.

A.(3.) CN and Hg MONITORING REQUIREMENTS

The Permittee shall compile monitoring data for CN and Hg into report after 2 years of monitoring have been completed and submit the report with all available data to the Division for review.

A.(4.) CHRONIC TOXICITY PERMIT LIMIT (QUARTERLY) @ 14.5 MGD

The effluent discharge shall at no time exhibit observable inhibition of reproduction or significant mortality to *Ceriodaphnia dubia* at an effluent concentration of 90%.

The permit holder shall perform at a minimum, *quarterly* monitoring using test procedures outlined in the "North Carolina *Ceriodaphnia* Chronic Effluent Bioassay Procedure," Revised February 1998, or subsequent versions or "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions. The tests will be performed during the months of **February, May, August and November**. Effluent sampling for this testing shall be performed at the NPDES permitted final effluent discharge below all treatment processes.

If the test procedure, performed as the first test of any single quarter, results in a failure or ChV below the permit limit, then multiple-concentration testing shall be performed at a minimum, in each of the two following months, as described in "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions.

The chronic value for multiple concentration tests will be determined using the geometric mean of the highest concentration having no detectable impairment of reproduction or survival and the lowest concentration that does have a detectable impairment of reproduction or survival. The definition of "detectable impairment," collection methods, exposure regimes, and further statistical methods are specified in the "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions.

All toxicity testing results required as part of this permit condition will be entered on the Effluent Discharge Monitoring Form (MR-1) for the months in which tests were performed, using the **parameter code TGP3B** for the pass/fail results and **THP3B** for the Chronic Value. Additionally, DWQ Form AT-3 (original) is to be sent to the following address:

NCDENR/ DWQ/ Environmental Sciences Section
1621 Mail Service Center
Raleigh, North Carolina 27699-1621

Completed Aquatic Toxicity Test Forms shall be filed with the Environmental Sciences Section no later than 30 days after the end of the reporting period for which the report is made. Test data shall be complete, accurate, include all supporting chemical/physical measurements and all concentration/response data, and be certified by laboratory supervisor and ORC or approved designate signature. Total residual chlorine of the effluent toxicity sample must be measured and reported if chlorine is employed for disinfection of the waste stream.

Should there be no discharge of flow from the facility during a month in which toxicity monitoring is required, the Permittee will complete the information located at the top of the aquatic toxicity (AT) test form indicating the facility name, permit number, pipe number, county, and the month/year of the report with the notation of "No Flow" in the comment area of the form. The report shall be submitted to the Environmental Sciences Section at the address cited above.

Should the Permittee fail to monitor during a month in which toxicity monitoring is required, monitoring will be required during the following month.

Should any test data from this monitoring requirement or tests performed by the North Carolina Division of Water Quality indicate potential impacts to the receiving stream, this permit may be re-opened and modified to include alternate monitoring requirements or limits.

NOTE: Failure to achieve test conditions as specified in the cited document (such as minimum control organism survival, minimum control organism reproduction, and/or appropriate environmental controls) shall constitute an invalid test and will require immediate follow-up testing, to be completed no later than the last day of the month following the month of initial monitoring.

A.(5.) EFFLUENT POLLUTANT SCAN

The Permittee shall perform an annual Effluent Pollutant Scan for all parameters listed in the attached table. The Permittee shall perform an annual Effluent Pollutant Scan for all parameters listed in the attached table. The analytical methods shall be in accordance with 40 CFR Part 136 and shall be sufficiently sensitive to determine whether parameters are present in concentrations greater than applicable standards and criteria. Samples shall **represent seasonal variations**. Unless otherwise indicated, metals shall be analyzed as "total recoverable."

Ammonia (as N)
Chlorine (total residual, TRC)
Dissolved oxygen
Nitrate/Nitrite
Total Kjeldahl nitrogen
Oil and grease
Total Phosphorus
Total dissolved solids
Hardness
Antimony
Arsenic
Beryllium
Cadmium
Chromium
Copper
Lead
Mercury
Nickel
Selenium
Silver
Thallium
Zinc
Cyanide
Total phenolic compounds

Volatile organic compounds:

Acrolein
Acrylonitrile
Benzene
Bromoform
Carbon tetrachloride
Chlorobenzene
Chlorodibromomethane
Chloroethane
2-chloroethylvinyl ether
Chloroform
Dichlorobromomethane
1,1-dichloroethane
1,2-dichloroethane
Trans-1,2-dichloroethylene
1,1-dichloroethylene
1,2-dichloropropane
1,3-dichloropropylene
Ethylbenzene
Methyl bromide
Methyl chloride
Methylene chloride
1,1,2,2-tetrachloroethane
Tetrachloroethylene
Toluene
1,1,1-trichloroethane
1,1,2-trichloroethane
Trichloroethylene
Vinyl chloride

Acid-extractable compounds:

P-chloro-m-cresol
2-chlorophenol
2,4-dichlorophenol
2,4-dimethylphenol
4,6-dinitro-o-cresol
2,4-dinitrophenol
2-nitrophenol
4-nitrophenol
Pentachlorophenol
Phenol
2,4,6-trichlorophenol

Base-neutral compounds:

Acenaphthene
Acenaphthylene
Anthracene
Benzidine
Benzo(a)anthracene
Benzo(a)pyrene
3,4 benzofluoranthene
Benzo(ghi)perylene
Benzo(k)fluoranthene
Bis (2-chloroethoxy) methane
Bis (2-chloroethyl) ether
Bis (2-chloroisopropyl) ether
Bis (2-ethylhexyl) phthalate
4-bromophenyl phenyl ether
Butyl benzyl phthalate
2-chloronaphthalene
4-chlorophenyl phenyl ether
Chrysene
Di-n-butyl phthalate
Di-n-octyl phthalate
Dibenzo(a,h)anthracene
1,2-dichlorobenzene
1,3-dichlorobenzene
1,4-dichlorobenzene
3,3-dichlorobenzidine
Diethyl phthalate
Dimethyl phthalate
2,4-dinitrotoluene
2,6-dinitrotoluene
1,2-diphenylhydrazine
Fluoranthene
Fluorene
Hexachlorobenzene
Hexachlorobutadiene
Hexachlorocyclo-pentadiene
Hexachloroethane
Indeno(1,2,3-cd)pyrene
Isophorone
Naphthalene
Nitrobenzene
N-nitrosodi-n-propylamine
N-nitrosodimethylamine
N-nitrosodiphenylamine
Phenanthrene
Pyrene
1,2,4-trichlorobenzene

Test results shall be reported to the Division in DWQ Form- DMR-PPA1 or in a form approved by the Director within 90 days of sampling. The report shall be submitted to the following address:

NC DENR / DWQ / Central Files
1617 Mail Service Center
Raleigh, North Carolina 27699-1617.

A.(6.) NUTRIENT ALLOCATIONS

(a.) The following table lists the Total Nitrogen (TN) and Total Phosphorus (TP) allocations assigned to, acquired by, or transferred to the Permittee in accordance with the Jordan Lake nutrient management rule (T15A NCAC 02B .0270) and the status of each as of permit issuance. For compliance purposes, this table does not supersede any TN or TP limit established elsewhere in this permit or in the NPDES permit of a compliance association of which the Permittee is a Co-Permittee Member.

Total Nitrogen Allocation

ALLOCATION TYPE	SOURCE	DATE	ALLOCATION AMOUNT (1)		STATUS
			Delivered (lb/yr)	Discharge (lb/yr)	
Base	Assigned by Rule (T15A NCAC 02B .0270)	8/11/09	84,656	134,375	Active
		TOTAL	84,656	134,375	Active

Footnote:

(1) Nitrogen Transport Factor = 63%

Total Phosphorus Allocation

ALLOCATION TYPE	SOURCE	DATE	ALLOCATION AMOUNT (1)		STATUS
			Delivered (lb/yr)	Discharge (lb/yr)	
Base	Assigned by Rule (T15A NCAC 02B .0270)	8/11/09	4,789	10,188	Active
		TOTAL	4,789	10,188	Active

Footnote:

(1) Phosphorus Transport Factor = 47%

(b.) Any addition, deletion, or modification of the listed allocation(s) (other than to correct typographical errors) or any change in status of any of the listed allocations shall be considered a major modification of this permit and shall be subject to the public review process afforded such modifications under state and federal rules.

A.(7.) ANNUAL LIMITS FOR TOTAL NITROGEN OR TOTAL PHOSPHORUS

(a.) Total Nitrogen (TN) and Total Phosphorus (TP) allocations and load limits for NPDES dischargers in the Jordan Lake watershed are annual limits and apply on a calendar year basis.

(b.) For any given calendar year, the Permittee shall be in compliance with the annual TN (or TP) Load limit in this Permit if:

(i.) the Permittee's annual TN (or TP) Load is less than or equal to the effective limit, or

(ii.) the Permittee is a Co-Permittee Member of a compliance association.

(c.) The TN (or TP) Load limit in this Permit may be modified as the result of allowable changes in the Permittee's allocations.

(i.) Allowable changes include those resulting from purchase of TN (or TP) allocation from an authorized mitigation banker, the Ecosystem Enhancement Program, or other source allowed under applicable regulations; purchase, sale, trade, or lease of allocation between the Permittee and other dischargers; regionalization; and other transactions approved by the Division.

(ii.) The Permittee may request a modification of the TN (or TP) Load limit in this Permit to reflect allowable changes in its allocation(s).

(A) Upon receipt of timely and proper application, the Division will modify the permit as appropriate and in accordance with state and federal program requirements.

(B) Changes in TN (or TP) limits become effective on January 1 of the year following permit modification. The Division must receive application no later than August 31 for changes proposed for the following calendar year.

(iii.) Any requests for modification should be sent to:

NCDENR/ DWQ/ NPDES Programs
Attn: Jordan Lake Watershed Coordinator
1617 Mail Service Center
Raleigh, NC 27699-1617

(d.) If the Permittee is a member and co-permittee of an approved compliance association on January 1 of a given year, its TN and TP discharges during that year are governed by that association's group NPDES permit and the limits therein.

(i.) The Permittee shall be considered a Co-Permittee Member for any given calendar year in which it is identified as such in Appendix A of the association's group NPDES permit.

(ii.) Association roster(s) and members' TN and TP allocations will be updated annually and in accordance with state and federal program requirements.

(iii.) If the Permittee intends to join or leave a compliance association, the Division must be notified of the proposed action in accordance with the procedures defined in the association's NPDES permit.

(A) Upon receipt of timely and proper notification, the Division will modify the permit as appropriate and in accordance with state and federal program requirements.

(B) Membership changes in a compliance association become effective on January 1 of the year following modification of the association's permit.

(e.) The TN and TP monitoring and reporting requirements in this Permit remain in effect throughout the term of the Permit and are not affected by the Permittee's membership in a compliance association.

A.(8.) CALCULATION OF TOTAL NITROGEN OR TOTAL PHOSPHORUS LOADS

(a.) The Permittee shall calculate monthly and annual TN Loads as follows:

(i.) Monthly TN (or TP) Load (lb/mo) = TN (or TP) x TMF x 8.34

where:

TN or TP	=	the average Total Nitrogen or Total Phosphorus concentration (mg/L) of the composite samples collected during the month
TMF	=	the Total Monthly Flow of wastewater discharged during the month (MG/mo)
8.34	=	conversion factor, from (mg/L x MG) to pounds

(ii.) Annual TN (or TP) Load (lb/yr) = Sum of the 12 Monthly TN (or TP) Loads for the calendar year

(b.) The Permittee shall report monthly Total Nitrogen and Total Phosphorus results (mg/L and lb/mo) in the appropriate discharge monitoring report for each month and shall report each calendar year's results (lb/yr) with the December report for that year.

A.(9.) NITROGEN OPTIMIZATION

(a.) Not later than February 11, 2010, the Permittee shall:

(i.) evaluate its treatment facilities and operations,

(ii.) identify further opportunities to improve and optimize nitrogen reduction in the existing facilities beyond those previously implemented pursuant to G.S. 143-215.1B(d), and

(iii.) submit a report to the Division documenting its findings, proposing optimization measures, and describing expected results.

(b.) No later than six months following the Division's written acceptance of the report, or as provided in the acceptance, the Permittee shall implement the proposed measures.

(c.) Beginning one year following the Division's acceptance of the report and continuing through 2016, the Permittee shall submit a progress report to the Division documenting the status of the proposed measures and the nitrogen reductions achieved at the facility during the previous calendar year.

(d.) Reports shall be submitted to:

NCDENR/ DWQ/ NPDES Programs
Attn: Jordan Lake Watershed Coordinator
1617 Mail Service Center
Raleigh, NC 27699-1617

A.(10.) NUTRIENT MONITORING RE-OPENER

Pursuant to N.C. General Statutes Section 143-215.1 and the implementing rules found in Title 15A of the North Carolina Administrative Code, Subchapter 02H, specifically, 15A NCAC 02H.0112(b)(1) and 02H.0114(a), and Part II, Sections B.12. and B.13. of this Permit, the Director of DWQ may reopen this permit to require supplemental nutrient monitoring of the discharge. The additional monitoring will be to support water quality modeling efforts within the Cape Fear River Basin, and shall be consistent with a monitoring plan developed jointly by the Division and affected stakeholders.

PART II
STANDARD CONDITIONS FOR NPDES PERMITS

Section A. Definitions

2/Month
Samples are collected twice per month with at least ten calendar days between sampling events.

3/Week
Samples are collected three times per week on three separate calendar days.

Act or "the Act"
The Federal Water Pollution Control Act, also known as the Clean Water Act, as amended, 33 USC 1251, et. seq.

Annual Average
The arithmetic mean of all "daily discharges" of a pollutant measured during the calendar year. In the case of fecal coliform, the geometric mean of such discharges.

Arithmetic Mean
The summation of the individual values divided by the number of individual values.

Bypass
The known diversion of waste streams from any portion of a treatment facility including the collection system, which is not a designed or established or operating mode for the facility.

Calendar Day
The period from midnight of one day until midnight of the next day. However, for purposes of this permit, any consecutive 24-hour period that reasonably represents the calendar day may be used for sampling.

Calendar Quarter
One of the following distinct periods: January through March, April through June, July through September, and October through December.

Composite Sample
A sample collected over a 24-hour period by continuous sampling or combining grab samples of at least 100 ml in such a manner as to result in a total sample representative of the wastewater discharge during the sample period. The Director may designate the most appropriate method (specific number and size of aliquots necessary, the time interval between grab samples, etc.) on a case-by-case basis. Samples may be collected manually or automatically. Composite samples may be obtained by the following methods:

(1) Continuous: a single, continuous sample collected over a 24-hour period proportional to the rate of flow.
(2) Constant time/variable volume: a series of grab samples collected at equal time intervals over a 24 hour period of discharge and combined proportional to the rate of flow measured at the time of individual sample collection, or
(3) Variable time/constant volume: a series of grab samples of equal volume collected over a 24 hour period with the time intervals between samples determined by a preset number of gallons passing the sampling point. Flow measurement between sample intervals shall be determined by use of a flow recorder and totalizer, and the preset gallon interval between sample collection fixed at no greater than 1/24 of the expected total daily flow at the treatment system, or
(4) Constant time/constant volume: a series of grab samples of equal volume collected over a 24-hour period at a constant time interval. **Use of this method requires prior approval by the Director. This method may only be used in situations where effluent flow rates vary less than 15 percent.** The following restrictions also apply:
   - Influent and effluent grab samples shall be of equal size and of no less than 100 milliliters

- Influent samples shall not be collected more than once per hour.
- Permittees with wastewater treatment systems whose detention time ≤ 24 hours shall collect effluent grab samples at intervals of no greater than 20 minutes apart during any 24-hour period.
- Permittees with wastewater treatment systems whose detention time exceeds 24 hours shall collect effluent grab samples at least every X hours [X = days detention time] over a 24-hour period. Effluent samples shall be collected at least every six hours; there must be a minimum of four samples during a 24-hour sampling period.

Continuous flow measurement
Flow monitoring that occurs without interruption throughout the operating hours of the facility. Flow shall be monitored continually except for the infrequent times when there may be no flow or for infrequent maintenance activities on the flow device.

Daily Discharge
The discharge of a pollutant measured during a calendar day or any 24-hour period that reasonably represents the calendar day for purposes of sampling. For pollutants measured in units of mass, the "daily discharge" is calculated as the total mass of the pollutant discharged over the day. The "daily discharge" concentration comprises the mean concentration for a 24-hour sampling period as either a composite sample concentration or the arithmetic mean of all grab samples collected during that period. (40 CFR 122.2)

Daily Maximum
The highest "daily discharge" for conventional and other non-toxicant parameters. NOTE: Permittees may not submit a "daily average" calculation [for determining compliance with permit limits] for toxicants. See the relevant Federal effluent guideline[s] for the appropriate calculation interval.

Daily Sampling
Parameters requiring daily sampling shall be sampled 5 out of every 7 days per week unless otherwise specified in the permit. The Division expects that sampling shall be conducted on weekdays except where holidays or other disruptions of normal operations prevent weekday sampling. If sampling is required for all seven days of the week for any permit parameter(s), that requirement will be so noted on the Effluent Limitations and Monitoring Page(s).

DWQ or "the Division"
The Division of Water Quality, Department of Environment and Natural Resources.

EMC
The North Carolina Environmental Management Commission.

Facility Closure
The cessation of wastewater treatment at a permitted facility, or the cessation of all activities that require coverage under the NPDES. Completion of facility closure will allow this permit to be rescinded.

Geometric Mean
The Nth root of the product of the individual values where N = the number of individual values. For purposes of calculating the geometric mean, values of "0" (or "< [detection level]") shall be considered = 1.

Grab Sample
Individual samples of at least 100 ml collected over a period of time not exceeding 15 minutes. Grab samples can be collected manually. Grab samples must be representative of the discharge (or the receiving stream, for instream samples).

Hazardous Substance
Any substance designated under 40 CFR Part 116 pursuant to Section 311 of the Clean Water Act.

Instantaneous flow measurement
A measure of flow taken at the time of sampling, when both the sample and flow will be representative of the total discharge.

Monthly Average (concentration limit)
The arithmetic mean of all "daily discharges" of a pollutant measured during the calendar month. In the case of fecal coliform, the geometric mean of such discharges.

Permit Issuing Authority
The Director of the Division of Water Quality.

Quarterly Average (concentration limit)
The average of all samples taken over a calendar quarter.

Severe property damage
Substantial physical damage to property, damage to the treatment facilities which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage excludes economic loss caused by delays in production.

Toxic Pollutant:
Any pollutant listed as toxic under Section 307(a)(l) of the Clean Water Act.

Upset
An incident beyond the reasonable control of the Permittee causing unintentional and temporary noncompliance with permit effluent limitations and/or monitoring requirements. An upset does not include noncompliance caused by operational error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventive maintenance, or careless or improper operation.

Weekly Average (concentration limit)
The arithmetic mean of all "daily discharges" of a pollutant measured during the calendar week. In the case of fecal coliform, the geometric mean of such discharges.

Section B. General Conditions

1. Duty to Comply
   The Permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of the Clean Water Act and is grounds for enforcement action; for permit termination, revocation and reissuance, or modification; or denial of a permit renewal application [40 CFR 122.41].

   a. The Permittee shall comply with effluent standards or prohibitions established under section 307(a) of the Clean Water Act for toxic pollutants and with standards for sewage sludge use or disposal established under section 405(d) of the Clean Water Act within the time provided in the regulations that establish these standards or prohibitions or standards for sewage sludge use or disposal, even if the permit has not yet been modified to incorporate the requirement.

   b. The Clean Water Act provides that any person who violates section 301, 302, 306, 307, 308, 318 or 405 of the Act, or any permit condition or limitation implementing any such sections in a permit issued under section 402, or any requirement imposed in a pretreatment program approved under sections 402(a)(3) or 402(b)(8) of the Act, is subject to a civil penalty not to exceed $25,000 per day for each violation. [40 CFR 122.41 (a)(2)]

   c. The Clean Water Act provides that any person who *negligently* violates sections 301, 302, 306, 307, 308, 318, or 405 of the Act, or any condition or limitation implementing any of such sections in a permit issued under

section 402 of the Act, or any requirement imposed in a pretreatment program approved under section 402(a)(3) or 402(b)(8) of the Act, is subject to criminal penalties of $2,500 to $25,000 per day of violation, or imprisonment of not more than 1 year, or both. In the case of a second or subsequent conviction for a negligent violation, a person shall be subject to criminal penalties of not more than $50,000 per day of violation, or by imprisonment of not more than 2 years, or both. [40 CFR 122.41 (a) (2)]

d. Any person who *knowingly* violates such sections, or such conditions or limitations is subject to criminal penalties of $5,000 to $50,000 per day of violation, or imprisonment for not more than 3 years, or both. In the case of a second or subsequent conviction for a knowing violation, a person shall be subject to criminal penalties of not more than $100,000 per day of violation, or imprisonment of not more than 6 years, or both. [40 CFR 122.41 (a) (2)]

e. Any person who knowingly violates section 301, 302, 303, 306, 307, 308, 318 or 405 of the Act, or any permit condition or limitation implementing any of such sections in a permit issued under section 402 of the Act, and who knows at that time that he thereby places another person in imminent danger of death or serious bodily injury, shall, upon conviction, be subject to a fine of not more than $250,000 or imprisonment of not more than 15 years, or both. In the case of a second or subsequent conviction for a knowing endangerment violation, a person shall be subject to a fine of not more than $500,000 or by imprisonment of not more than 30 years, or both. An organization, as defined in section 309(c)(3)(B)(iii) of the CWA, shall, upon conviction of violating the imminent danger provision, be subject to a fine of not more than $1,000,000 and can be fined up to $2,000,000 for second or subsequent convictions. [40 CFR 122.41 (a) (2)]

f. Under state law, a civil penalty of not more than $25,000 per violation may be assessed against any person who violates or fails to act in accordance with the terms, conditions, or requirements of a permit. [North Carolina General Statutes § 143-215.6A]

g. Any person may be assessed an administrative penalty by the Administrator for violating section 301, 302, 306, 307, 308, 318 or 405 of this Act, or any permit condition or limitation implementing any of such sections in a permit issued under section 402 of this Act. Administrative penalties for Class I violations are not to exceed $10,000 per violation, with the maximum amount of any Class I penalty assessed not to exceed $25,000. Penalties for Class II violations are not to exceed $10,000 per day for each day during which the violation continues, with the maximum amount of any Class II penalty not to exceed $125,000. [40 CFR 122.41 (a) (3)]

2. Duty to Mitigate
The Permittee shall take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit with a reasonable likelihood of adversely affecting human health or the environment [40 CFR 122.41 (d)].

3. Civil and Criminal Liability
Except as provided in permit conditions on "Bypassing" (Part II. C. 4), "Upsets" (Part II. C. 5) and "Power Failures" (Part II. C. 7), nothing in this permit shall be construed to relieve the Permittee from any responsibilities, liabilities, or penalties for noncompliance pursuant to NCGS 143-215.3, 143-215.6 or Section 309 of the Federal Act, 33 USC 1319. Furthermore, the Permittee is responsible for consequential damages, such as fish kills, even though the responsibility for effective compliance may be temporarily suspended.

4. Oil and Hazardous Substance Liability
Nothing in this permit shall be construed to preclude the institution of any legal action or relieve the Permittee from any responsibilities, liabilities, or penalties to which the Permittee is or may be subject to under NCGS 143-215.75 et seq. or Section 311 of the Federal Act, 33 USG 1321. Furthermore, the Permittee is responsible for consequential damages, such as fish kills, even though the responsibility for effective compliance may be temporarily suspended.

5. Property Rights
The issuance of this permit does not convey any property rights in either real or personal property, or any exclusive privileges, nor does it authorize any injury to private property or any invasion of personal rights, nor any infringement of Federal, State or local laws or regulations [40 CFR 122.41 (g)].

6. Onshore or Offshore Construction
This permit does not authorize or approve the construction of any onshore or offshore physical structures or facilities or the undertaking of any work in any navigable waters.

7. Severability
The provisions of this permit are severable. If any provision of this permit, or the application of any provision of this permit to any circumstances, is held invalid, the application of such provision to other circumstances, and the remainder of this permit, shall not be affected thereby [NCGS 150B-23].

8. Duty to Provide Information
The Permittee shall furnish to the Permit Issuing Authority, within a reasonable time, any information which the Permit Issuing Authority may request to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit or to determine compliance with this permit. The Permittee shall also furnish to the Permit Issuing Authority upon request, copies of records required by this permit [40 CFR 122.41 (h)].

9. Duty to Reapply
If the Permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the Permittee must apply for and obtain a new permit [40 CFR 122.41 (b)].

10. Expiration of Permit
The Permittee is not authorized to discharge after the expiration date. In order to receive automatic authorization to discharge beyond the expiration date, the Permittee shall submit such information, forms, and fees as are required by the agency authorized to issue permits no later than 180 days prior to the expiration date. Any Permittee that has not requested renewal at least 180 days prior to expiration, or any Permittee that does not have a permit after the expiration and has not requested renewal at least 180 days prior to expiration, will subject the Permittee to enforcement procedures as provided in NCGS 143-215.6 and 33 USC 1251 et. seq.

11. Signatory Requirements
All applications, reports, or information submitted to the Permit Issuing Authority shall be signed and certified [40 CFR 122.41 (k)].

a. **All permit applications shall be signed as follows:**
   (1) For a corporation: by a responsible corporate officer. For the purpose of this Section, a responsible corporate officer means: (a) a president, secretary, treasurer or vice president of the corporation in charge of a principal business function, or any other person who performs similar policy or decision making functions for the corporation, or (b) the manager of one or more manufacturing, production, or operating facilities, provided, the manager is authorized to make management decisions which govern the operation of the regulated facility including having the explicit or implicit duty of making major capital investment recommendations, and initiating and directing other comprehensive measures to assure long term environmental compliance with environmental laws and regulations; the manager can ensure that the necessary systems are established or actions taken to gather complete and accurate information for permit application requirements; and where authority to sign documents has been assigned or delegated to the manager in accordance with corporate procedures .
   (2) For a partnership or sole proprietorship: by a general partner or the proprietor, respectively; or
   (3) For a municipality, State, Federal, or other public agency: by either a principal executive officer or ranking elected official [40 CFR 122.22].

b. All reports required by the permit and other information requested by the Permit Issuing Authority shall be signed by a person described in paragraph a. above or by a duly authorized representative of that person. A person is a duly authorized representative only if:
   1. The authorization is made in writing by a person described above;
   2. The authorization specified either an individual or a position having responsibility for the overall operation of the regulated facility or activity, such as the position of plant manager, operator of a well or well field, superintendent, a position of equivalent responsibility, or an individual or position having overall responsibility for environmental matters for the company. (A duly authorized representative may thus be either a named individual or any individual occupying a named position.); and
   3. The written authorization is submitted to the Permit Issuing Authority [40 CFR 122.22]

c. Changes to authorization: If an authorization under paragraph (b) of this section is no longer accurate because a different individual or position has responsibility for the overall operation of the facility, a new authorization satisfying the requirements of paragraph (b) of this section must be submitted to the Director prior to or together with any reports, information, or applications to be signed by an authorized representative [40 CFR 122.22]

d. Certification. Any person signing a document under paragraphs a. or b. of this section shall make the following certification [40 CFR 122.22]:
"I certify, under penalty of law, that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fines and imprisonment for knowing violations."

12. Permit Actions
This permit may be modified, revoked and reissued, or terminated for cause. The filing of a request by the Permittee for a permit modification, revocation and reissuance, or termination, or a notification of planned changes or anticipated noncompliance does not stay any permit condition [40 CFR 122.41 (f)].

13. Permit Modification, Revocation and Reissuance, or Termination
The issuance of this permit does not prohibit the permit issuing authority from reopening and modifying the permit, revoking and reissuing the permit, or terminating the permit as allowed by the laws, rules, and regulations contained in Title 40, Code of Federal Regulations, Parts l22 and l23; Title l5A of the North Carolina Administrative Code, Subchapter 2H .0l00; and North Carolina General Statute l43-215.l et. al.

14. Annual Administering and Compliance Monitoring Fee Requirements
The Permittee must pay the annual administering and compliance monitoring fee within thirty days after being billed by the Division. Failure to pay the fee in a timely manner in accordance with 15A NCAC 2H.0105 (b) (2) may cause this Division to initiate action to revoke the permit.

Section C. Operation and Maintenance of Pollution Controls

1. Certified Operator
Upon classification of the permitted facility by the Certification Commission, the Permittee shall employ a certified water pollution control treatment system operator in responsible charge (ORC) of the water pollution control treatment system. Such operator must hold a certification of the grade equivalent to or greater than the classification assigned to the water pollution control treatment system by the Certification Commission. The Permittee must also employ one or more certified Back-up ORCs who possess a currently valid certificate of the type of the system. Back-up ORCs must possess a grade equal to (or no more than one grade less than) the grade of the system [15A NCAC 8G.0201].

The ORC of each Class I facility must:

- Visit the facility at least weekly
- Comply with all other conditions of 15A NCAC 8G.0204.

The ORC of each Class II, III and IV facility must:

- Visit the facility at least five days per week, excluding holidays
- Properly manage and document daily operation and maintenance of the facility
- Comply with all other conditions of 15A NCAC 8G.0204.

Once the facility is classified, the Permittee shall submit a letter to the Certification Commission designating the operator in responsible charge:

a. Within 60 calendar days prior to wastewater being introduced into a new system
b. Within 120 calendar days of:
    - Receiving notification of a change in the classification of the system requiring the designation of a new ORC and back-up ORC
    - A vacancy in the position of ORC or back-up ORC.

2. Proper Operation and Maintenance

The Permittee shall at all times provide the operation and maintenance resources necessary to operate the existing facilities at optimum efficiency. The Permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the Permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls and appropriate quality assurance procedures. This provision requires the Permittee to install and operate backup or auxiliary facilities only when necessary to achieve compliance with the conditions of the permit [40 CFR 122.41 (e)].

3. Need to Halt or Reduce not a Defense

It shall not be a defense for a Permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the condition of this permit [40 CFR 122.41 (c)].

4. Bypassing of Treatment Facilities

a. Bypass not exceeding limitations [40 CFR 122.41 (m) (2)]

The Permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, but only if it also is for essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of Paragraphs b. and c. of this section.

b. Notice [40 CFR 122.41 (m) (3)]

(1) Anticipated bypass. If the Permittee knows in advance of the need for a bypass, it shall submit prior notice, if possible at least ten days before the date of the bypass; including an evaluation of the anticipated quality and effect of the bypass.

(2) Unanticipated bypass. The Permittee shall submit notice of an unanticipated bypass as required in Part II. E. 6. (24-hour notice).

c. Prohibition of Bypass

(1) Bypass from the treatment facility is prohibited and the Permit Issuing Authority may take enforcement action against a Permittee for bypass, unless:

(A) Bypass was unavoidable to prevent loss of life, personal injury or severe property damage;

(B) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass which occurred during normal periods of equipment downtime or preventive maintenance; and

(C) The Permittee submitted notices as required under Paragraph b. of this section.

(2) Bypass from the collection system is prohibited and the Permit Issuing Authority may take enforcement action against a Permittee for a bypass as provided in any current or future system-wide collection system permit associated with the treatment facility.

(3) The Permit Issuing Authority may approve an anticipated bypass, after considering its adverse effects, if the Permit Issuing Authority determines that it will meet the three conditions listed above in Paragraph c. (l) of this section.

5. Upsets

a. Effect of an upset [40 CFR 122.41 (n) (2)]: An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of paragraph b. of this condition are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

b. Conditions necessary for a demonstration of upset: A Permittee who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:
(1) An upset occurred and that the Permittee can identify the cause(s) of the upset;
(2) The Permittee facility was at the time being properly operated; and
(3) The Permittee submitted notice of the upset as required in Part II. E. 6. (b) of this permit.
(4) The Permittee complied with any remedial measures required under Part II. B. 2. of this permit.

c. Burden of proof [40 CFR 122.41 (n) (4)]: The Permittee seeking to establish the occurrence of an upset has the burden of proof in any enforcement proceeding.

6. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters shall be utilized/disposed of in accordance with NCGS 143-215.1 and in a manner such as to prevent any pollutant from such materials from entering waters of the State or navigable waters of the United States. The Permittee shall comply with all existing Federal regulations governing the disposal of sewage sludge. Upon promulgation of 40 CFR Part 503, any permit issued by the Permit Issuing Authority for the utilization/disposal of sludge may be reopened and modified, or revoked and reissued, to incorporate applicable requirements at 40 CFR 503. The Permittee shall comply with applicable 40 CFR 503 Standards for the Use and Disposal of Sewage Sludge (when promulgated) within the time provided in the regulation, even if the permit is not modified to incorporate the requirement. The Permittee shall notify the Permit Issuing Authority of any significant change in its sludge use or disposal practices.

7. Power Failures

The Permittee is responsible for maintaining adequate safeguards (as required by 15A NCAC 2H.0124 – Reliability) to prevent the discharge of untreated or inadequately treated wastes during electrical power failures either by means of alternate power sources, standby generators or retention of inadequately treated effluent.

Section D. Monitoring and Records

1. Representative Sampling

Samples collected and measurements taken, as required herein, shall be characteristic of the volume and nature of the permitted discharge. Samples collected at a frequency less than daily shall be taken on a day and time that is characteristic of the discharge over the entire period the sample represents. All samples shall be taken at the monitoring points specified in this permit and, unless otherwise specified, before the effluent joins or is diluted by any other wastestream, body of water, or substance. Monitoring points shall not be changed without notification to and the approval of the Permit Issuing Authority [40 CFR 122.41 (j)].

2. Reporting

Monitoring results obtained during the previous month(s) shall be summarized for each month and reported on a monthly Discharge Monitoring Report (DMR) Form (MR 1, 1.1, 2, 3) or alternative forms approved by the Director, postmarked no later than the last calendar day of the month following the completed reporting period.

The first DMR is due on the last day of the month following the issuance of the permit or in the case of a new facility, on the last day of the month following the commencement of discharge. Duplicate signed copies of these, and all other reports required herein, shall be submitted to the following address:

NC DENR / Division of Water Quality / Water Quality Section
ATTENTION: Central Files
1617 Mail Service Center
Raleigh, North Carolina 27699-1617

3. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices shall be installed, calibrated and maintained to ensure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected shall be capable of measuring flows with a maximum deviation of less than 10% from the true discharge rates throughout the range of expected discharge volumes. Flow measurement devices shall be accurately calibrated at a minimum of once per year and maintained to ensure that the accuracy of the measurements is consistent with the accepted capability of that type of device. The Director shall approve the flow measurement device and monitoring location prior to installation.

Once-through condenser cooling water flow monitored by pump logs, or pump hour meters as specified in Part I of this permit and based on the manufacturer's pump curves shall not be subject to this requirement.

4. Test Procedures

Test procedures for the analysis of pollutants shall conform to the EMC regulations (published pursuant to NCGS 143-215.63 et. seq.), the Water and Air Quality Reporting Acts, and to regulations published pursuant to Section 304(g), 33 USC 1314, of the Federal Water Pollution Control Act (as Amended), and 40 CFR 136; or in the case of sludge use or disposal, approved under 40 CFR 136, unless otherwise specified in 40 CFR 503, unless other test procedures have been specified in this permit [40 CFR 122.41].

To meet the intent of the monitoring required by this permit, all test procedures must produce minimum detection and reporting levels that are below the permit discharge requirements and all data generated must be reported down to the minimum detection or lower reporting level of the procedure. If no approved methods are determined capable of achieving minimum detection and reporting levels below permit discharge requirements, then the most sensitive (method with the lowest possible detection and reporting level) approved method must be used.

5. Penalties for Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate, any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than two years per violation, or by both. If a conviction of a person is for a violation committed after a first conviction of such person under this paragraph, punishment is a fine of not more than $20,000 per day of violation, or by imprisonment of not more than 4 years, or both [40 CFR 122.41].

6. Records Retention

Except for records of monitoring information required by this permit related to the Permittee's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR 503), the Permittee shall retain records of all monitoring information, including:

- all calibration and maintenance records
- all original strip chart recordings for continuous monitoring instrumentation

- copies of all reports required by this permit
- copies of all data used to complete the application for this permit

These records or copies shall be maintained for a period of at least 3 years from the date of the sample, measurement, report or application. This period may be extended by request of the Director at any time [40 CFR 122.41].

7. Recording Results

For each measurement or sample taken pursuant to the requirements of this permit, the Permittee shall record the following information [40 CFR 122.41]:

a. The date, exact place, and time of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

8. Inspection and Entry

The Permittee shall allow the Director, or an authorized representative (including an authorized contractor acting as a representative of the Director), upon the presentation of credentials and other documents as may be required by law, to;

a. Enter upon the Permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit; and
d. Sample or monitor at reasonable times, for the purposes of assuring permit compliance or as otherwise authorized by the Clean Water Act, any substances or parameters at any location [40 CFR 122.41 (i)].

Section E Reporting Requirements

1. Change in Discharge

All discharges authorized herein shall be consistent with the terms and conditions of this permit. The discharge of any pollutant identified in this permit more frequently than or at a level in excess of that authorized shall constitute a violation of the permit.

2. Planned Changes

The Permittee shall give notice to the Director as soon as possible of any planned physical alterations or additions to the permitted facility [40 CFR 122.41 (l)]. Notice is required only when:

a. The alteration or addition to a permitted facility may meet one of the criteria for new sources at 40 CFR 122.29 (b); or
b. The alteration or addition could significantly change the nature or increase the quantity of pollutants discharged. This notification applies to pollutants subject neither to effluent limitations in the permit, nor to notification requirements under 40 CFR 122.42 (a) (l).
c. The alteration or addition results in a significant change in the Permittee's sludge use or disposal practices, and such alteration, addition or change may justify the application of permit conditions that are different from or absent in the existing permit, including notification of additional use or disposal sites not reported during the permit application process or not reported pursuant to an approved land application plan.

3. Anticipated Noncompliance

The Permittee shall give advance notice to the Director of any planned changes to the permitted facility or other activities that might result in noncompliance with the permit [40 CFR 122.41 (l) (2)].

4. Transfers
This permit is not transferable to any person except after notice to the Director. The Director may require modification or revocation and reissuance of the permit to document the change of ownership. Any such action may incorporate other requirements as may be necessary under the Clean Water Act [40 CFR 122.41 (l) (3)].

5. Monitoring Reports
Monitoring results shall be reported at the intervals specified elsewhere in this permit [40 CFR 122.41 (l) (4)].

   a. Monitoring results must be reported on a Discharge Monitoring Report (DMR) (See Part II. D. 2) or forms provided by the Director for reporting results of monitoring of sludge use or disposal practices.
   b. If the Permittee monitors any pollutant more frequently than required by this permit, the results of such monitoring shall be included in the calculation and reporting of the data submitted on the DMR.

6. Twenty-four Hour Reporting
   a. The Permittee shall report to the Director or the appropriate Regional Office any noncompliance that potentially threatens public health or the environment. Any information shall be provided orally within 24 hours from the time the Permittee became aware of the circumstances. A written submission shall also be provided within 5 days of the time the Permittee becomes aware of the circumstances. The written submission shall contain a description of the noncompliance, and its cause; the period of noncompliance, including exact dates and times, and if the noncompliance has not been corrected, the anticipated time it is expected to continue; and steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance [40 CFR 122.41 (l) (6)].

   b. The Director may waive the written report on a case-by-case basis for reports under this section if the oral report has been received within 24 hours.

   c. Occurrences outside normal business hours may also be reported to the Division's Emergency Response personnel at (800) 662-7956, (800) 858-0368 or (919) 733-3300.

7. Other Noncompliance
The Permittee shall report all instances of noncompliance not reported under Part II. E. 5 and 6. of this permit at the time monitoring reports are submitted. The reports shall contain the information listed in Part II. E. 6. of this permit [40 CFR 122.41 (l) (7)].

8. Other Information
Where the Permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or in any report to the Director, it shall promptly submit such facts or information [40 CFR 122.41 (l) (8)].

9. Noncompliance Notification
The Permittee shall report by telephone to either the central office or the appropriate regional office of the Division as soon as possible, but in no case more than 24 hours or on the next working day following the occurrence or first knowledge of the occurrence of any of the following:

   a. Any occurrence at the water pollution control facility which results in the discharge of significant amounts of wastes which are abnormal in quantity or characteristic, such as the dumping of the contents of a sludge digester; the known passage of a slug of hazardous substance through the facility; or any other unusual circumstances.
   b. Any process unit failure, due to known or unknown reasons, that render the facility incapable of adequate wastewater treatment such as mechanical or electrical failures of pumps, aerators, compressors, etc.
   c. Any failure of a pumping station, sewer line, or treatment facility resulting in a by-pass directly to receiving waters without treatment of all or any portion of the influent to such station or facility.

Persons reporting such occurrences by telephone shall also file a written report within 5 days following first knowledge of the occurrence.

10. Availability of Reports
Except for data determined to be confidential under NCGS 143-215.3 (a)(2) or Section 308 of the Federal Act, 33 USC 1318, all reports prepared in accordance with the terms shall be available for public inspection at the offices of the Division. As required by the Act, effluent data shall not be considered confidential. Knowingly making any false statement on any such report may result in the imposition of criminal penalties as provided for in NCGS 143-215.1 (b)(2) or in Section 309 of the Federal Act.

11. Penalties for Falsification of Reports
The Clean Water Act provides that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance shall, upon conviction, be punished by a fine of not more than $25,000 per violation, or by imprisonment for not more than two years per violation, or by both [40 CFR 122.41].

12. Annual Performance Reports
Permittees who own or operate facilities that collect or treat municipal or domestic waste shall provide an annual report to the Permit Issuing Authority and to the users/customers served by the Permittee (NCGS 143-215.1C). The report shall summarize the performance of the collection or treatment system, as well as the extent to which the facility was compliant with applicable Federal or State laws, regulations and rules pertaining to water quality. The report shall be provided no later than sixty days after the end of the calendar or fiscal year, depending upon which annual period is used for evaluation.

PART III
OTHER REQUIREMENTS

Section A. Construction
The Permittee shall not commence construction of wastewater treatment facilities, nor add to the plant's treatment capacity, nor change the treatment process(es) utilized at the treatment plant unless the Division has issued an Authorization to Construct (AtC) permit. Issuance of an AtC will not occur until Final Plans and Specifications for the proposed construction have been submitted by the Permittee and approved by the Division.

Section B. Groundwater Monitoring
The Permittee shall, upon written notice from the Director, conduct groundwater monitoring as may be required to determine the compliance of this NPDES permitted facility with the current groundwater standards.

Section C. Changes in Discharges of Toxic Substances
The Permittee shall notify the Permit Issuing Authority as soon as it knows or has reason to believe (40 CFR 122.42):

a. That any activity has occurred or will occur which would result in the discharge, on a routine or frequent basis, of any toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels";
   (1) One hundred micrograms per liter (100 µg/L);
   (2) Two hundred micrograms per liter (200 µg/L) for acrolein and acrylonitrile; five hundred micrograms per liter (500 µg/L) for 2.4-dinitrophenol and for 2-methyl-4.6-dinitrophenol; and one milligram per liter (1 mg/L) for antimony;
   (3) Five times the maximum concentration value reported for that pollutant in the permit application.

b. That any activity has occurred or will occur which would result in any discharge, on a non-routine or infrequent basis, of a toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels";

(1) Five hundred micrograms per liter (500 µg/L);
(2) One milligram per liter (1 mg/L) for antimony;
(3) Ten times the maximum concentration value reported for that pollutant in the permit application.

Section D. Evaluation of Wastewater Discharge Alternatives

The Permittee shall evaluate all wastewater disposal alternatives and pursue the most environmentally sound alternative of the reasonably cost effective alternatives. If the facility is in substantial non-compliance with the terms and conditions of the NPDES permit or governing rules, regulations or laws, the Permittee shall submit a report in such form and detail as required by the Division evaluating these alternatives and a plan of action within 60 days of notification by the Division.

Section E. Facility Closure Requirements

The Permittee must notify the Division at least 90 days prior to the closure of any wastewater treatment system covered by this permit. The Division may require specific measures during deactivation of the system to prevent adverse impacts to waters of the State. This permit cannot be rescinded while any activities requiring this permit continue at the permitted facility.

PART IV
SPECIAL CONDITIONS FOR MUNICIPAL FACILITIES

Section A. Publicly Owned Treatment Works (POTWs)

All POTWs must provide adequate notice to the Director of the following:

1. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of CWA if it were directly discharging those pollutants; and
2. Any substantial change in the volume or character of pollutants being introduced by an indirect discharger as influent to that POTW at the time of issuance of the permit.
3. For purposes of this paragraph, adequate notice shall include information on (1) the quality and quantity of effluent introduced into the POTW, and (2) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

Section B. Municipal Control of Pollutants from Industrial Users.

1. Effluent limitations are listed in Part I of this permit. Other pollutants attributable to inputs from industries using the municipal system may be present in the Permittee's discharge. At such time as sufficient information becomes available to establish limitations for such pollutants, this permit may be revised to specify effluent limitations for any or all of such other pollutants in accordance with best practicable technology or water quality standards.

2. Under no circumstances shall the Permittee allow introduction of the following wastes in the waste treatment system:
   a. Pollutants which create a fire or explosion hazard in the POTW, including, but not limited to, wastestreams with a closed cup flashpoint of less than 140 degrees Fahrenheit or 60 degrees Centigrade using the test methods specified in 40 CFR 261.21;
   b. Pollutants which will cause corrosive structural damage to the POTW, but in no case Discharges with pH lower than 5.0, unless the works is specifically designed to accommodate such Discharges;
   c. Solid or viscous pollutants in amounts which will cause obstruction to the flow in the POTW resulting in Interference;
   d. Any pollutant, including oxygen demanding pollutants (BOD, etc.) released in a Discharge at a flow rate and/or pollutant concentration which will cause Interference with the POTW;
   e. Heat in amounts which will inhibit biological activity in the POTW resulting in Interference, but in no case heat in such quantities that the temperature at the POTW Treatment Plant exceeds 40°C (104°F) unless the Division, upon request of the POTW, approves alternate temperature limits;

f. Petroleum oil, nonbiodegradable cutting oil, or products of mineral oil origin in amounts that will cause interference or pass through;
g. Pollutants which result in the presence of toxic gases, vapors, or fumes within the POTW in a quantity that may cause acute worker health and safety problems;
h. Any trucked or hauled pollutants, except at discharge points designated by the POTW.

3. With regard to the effluent requirements listed in Part I of this permit, it may be necessary for the Permittee to supplement the requirements of the Federal Pretreatment Standards (40 CFR, Part 403) to ensure compliance by the Permittee with all applicable effluent limitations. Such actions by the Permittee may be necessary regarding some or all of the industries discharging to the municipal system.

4. The Permittee shall require any industrial discharges sending influent to the permitted system to meet Federal Pretreatment Standards promulgated in response to Section 307(b) of the Act. Prior to accepting wastewater from any significant industrial user, the Permittee shall either develop and submit to the Division a Pretreatment Program for approval per 15A NCAC 2H .0907(a) or modify an existing Pretreatment Program per 15A NCAC 2H .0907(b).

5. This permit shall be modified, or alternatively, revoked and reissued, to incorporate or modify an approved POTW Pretreatment Program or to include a compliance schedule for the development of a POTW Pretreatment Program as required under Section 402(b)(8) of the Clean Water Act and implementing regulations or by the requirements of the approved State pretreatment program, as appropriate.

Section C. Pretreatment Programs

Under authority of sections 307(b) and (c) and 402(b)(8) of the Clean Water Act and implementing regulations 40 CFR Part 403, North Carolina General Statute 143-215.3 (14) and implementing regulations 15A NCAC 2H .0900, and in accordance with the approved pretreatment program, all provisions and regulations contained and referenced in the Pretreatment Program Submittal are an enforceable part of this permit.

The Permittee shall operate its approved pretreatment program in accordance with Section 402(b)(8) of the Clean Water Act, the Federal Pretreatment Regulations 40 CFR Part 403, the State Pretreatment Regulations 15A NCAC 2H .0900, and the legal authorities, policies, procedures, and financial provisions contained in its pretreatment program submission and Division approved modifications there of. Such operation shall include but is not limited to the implementation of the following conditions and requirements:

1. Sewer Use Ordinance (SUO)
The Permittee shall maintain adequate legal authority to implement its approved pretreatment program.

2. Industrial Waste Survey (IWS)
The Permittee shall update its Industrial Waste Survey (IWS) to include all users of the sewer collection system at least once every five years.

3. Monitoring Plan
The Permittee shall implement a Division-approved Monitoring Plan for the collection of facility specific data to be used in a wastewater treatment plant Headworks Analysis (HWA) for the development of specific pretreatment local limits. Effluent data from the Plan shall be reported on the DMRs (as required by Part II, Section D, and Section E.5.).

4. Headworks Analysis (HWA) and Local Limits
The Permittee shall obtain Division approval of a Headworks Analysis (HWA) at least once every five years, and as required by the Division. Within 180 days of the effective date of this permit (or any subsequent permit modification) the Permittee shall submit to the Division a written technical evaluation of the need to revise local limits (i.e., an updated HWA or documentation of why one is not needed) [40 CFR 122.44]. The Permittee shall develop, in accordance with 40 CFR 403.5(c) and 15A NCAC 2H .0909, specific Local Limits to implement the prohibitions listed in 40 CFR 403.5(a) and (b) and 15A NCAC 2H .0909.

5. Industrial User Pretreatment Permits (IUP) & Allocation Tables

In accordance with NCGS 143-215.1, the Permittee shall issue to all significant industrial users, permits for operation of pretreatment equipment and discharge to the Permittee's treatment works. These permits shall contain limitations, sampling protocols, reporting requirements, appropriate standard and special conditions, and compliance schedules as necessary for the installation of treatment and control technologies to assure that their wastewater discharge will meet all applicable pretreatment standards and requirements. The Permittee shall maintain a current Allocation Table (AT) which summarizes the results of the Headworks Analysis (HWA) and the limits from all Industrial User Pretreatment Permits (IUP). Permitted IUP loadings for each parameter cannot exceed the treatment capacity of the POTW as determined by the HWA.

6. Authorization to Construct (AtC)

The Permittee shall ensure that an Authorization to Construct permit (AtC) is issued to all applicable industrial users for the construction or modification of any pretreatment facility. Prior to the issuance of an AtC, the proposed pretreatment facility and treatment process must be evaluated for its capacity to comply with all Industrial User Pretreatment Permit (IUP) limitations.

7. POTW Inspection & Monitoring of their SIUs

The Permittee shall conduct inspection, surveillance, and monitoring activities as described in its Division approved pretreatment program in order to determine, independent of information supplied by industrial users, compliance with applicable pretreatment standards. The Permittee must:

a. Inspect all Significant Industrial Users (SIUs) at least once per calendar year; and
b. Sample all Significant Industrial Users (SIUs) at least twice per calendar year for all permit-limited pollutants, once during the period from January 1 through June 30 and once during the period from July 1 through December 31, except for organic compounds which shall be sampled once per calendar year;

8. SIU Self Monitoring and Reporting

The Permittee shall require all industrial users to comply with the applicable monitoring and reporting requirements outlined in the Division-approved pretreatment program, the industry's pretreatment permit, or in 15A NCAC 2H .0908.

9. Enforcement Response Plan (ERP)

The Permittee shall enforce and obtain appropriate remedies for violations of all pretreatment standards promulgated pursuant to section 307(b) and (c) of the Clean Water Act (40 CFR 405 et. seq.), prohibitive discharge standards as set forth in 40 CFR 403.5 and 15A NCAC 2H .0909, and specific local limitations. All enforcement actions shall be consistent with the Enforcement Response Plan (ERP) approved by the Division.

10. Pretreatment Annual Reports (PAR)

The Permittee shall report to the Division in accordance with 15A NCAC 2H .0908. In lieu of submitting annual reports, Modified Pretreatment Programs developed under 15A NCAC 2H .0904 (b) may be required to meet with Division personnel periodically to discuss enforcement of pretreatment requirements and other pretreatment implementation issues.

For all other active pretreatment programs, the Permittee shall submit two copies of a Pretreatment Annual Report (PAR) describing its pretreatment activities over the previous twelve months to the Division at the following address:

NC DENR / DWQ / Pretreatment Unit
1617 Mail Service Center
Raleigh, NC 27699-1617

These reports shall be submitted according to a schedule established by the Director and shall contain the following:

a.) Narrative
A brief discussion of reasons for, status of, and actions taken for all Significant Industrial Users (SIUs) in Significant Non-Compliance (SNC);

b.) Pretreatment Program Summary (PPS)
A pretreatment program summary (PPS) on specific forms approved by the Division;

c.) Significant Non-Compliance Report (SNCR)
The nature of the violations and the actions taken or proposed to correct the violations on specific forms approved by the Division;

d.) Industrial Data Summary Forms (IDSF)
Monitoring data from samples collected by both the POTW and the Significant Industrial User (SIU). These analytical results must be reported on Industrial Data Summary Forms (IDSF) or other specific format approved by the Division;

e.) Other Information
Copies of the POTW's allocation table, new or modified enforcement compliance schedules, public notice of SIUs in SNC, and any other information, upon request, which in the opinion of the Director is needed to determine compliance with the pretreatment implementation requirements of this permit;

11. Public Notice
The Permittee shall publish annually a list of Significant Industrial Users (SIUs) that were in Significant Non-Compliance (SNC) as defined in the Permittee's Division-approved Sewer Use Ordinance with applicable pretreatment requirements and standards during the previous twelve month period. This list shall be published within four months of the applicable twelve-month period.

12. Record Keeping
The Permittee shall retain for a minimum of three years records of monitoring activities and results, along with support information including general records, water quality records, and records of industrial impact on the POTW.

13. Funding and Financial Report
The Permittee shall maintain adequate funding and staffing levels to accomplish the objectives of its approved pretreatment program.

14. Modification to Pretreatment Programs
Modifications to the approved pretreatment program including but not limited to local limits modifications, POTW monitoring of their Significant Industrial Users (SIUs), and Monitoring Plan modifications, shall be considered a permit modification and shall be governed by 15 NCAC 2H .0114 and 15A NCAC 2H .0907.

DISCLAIMER

The full text of certain NPDES permits and the associated fact sheets has been made available to provide online access to this public information. EPA is making permits and fact sheets available electronically to provide convenient access for interested public parties and as a reference for permit writers. The ownership of these documents lies with the permitting authority, typically a State with an authorized NPDES program.

While EPA makes every effort to ensure that this web site remains current and contains the final version of the active permit, we cannot guarantee it is so. For example, there may be some delay in posting modifications made after a permit is issued. Also note that not all active permits are currently available electronically. Only permits and fact sheets for which the full text has been provided to Headquarters by the permitting authority may be made available. Headquarters has requested the full text only for permits as they are issued or reissued, beginning November 1, 2002.

Please contact the appropriate permitting authority (either a State or EPA Regional office) prior to acting on this information to ensure you have the most up-to-date permit and/or fact sheet. EPA recognizes the official version of a permit or fact sheet to be the version designated as such and appropriately stored by the respective permitting authority.

The documents are gathered from all permitting authorities, and all documents thus obtained are made available electronically, with no screening for completeness or quality. Thus, availability on the website does not constitute endorsement by EPA.

Michael F. Easley
Governor

William G. Ross, Jr., Secretary
North Carolina Department of Environment and Natural Resources

Alan W. Klimek, P.E., Director
Division of Water Quality

January 22, 2007

Ed Kerwin, Executive Director
Orange Water and Sewer Authority
P.O. Box 366
Carrboro, North Carolina 27510

Subject: Issuance of NPDES Permit NC0025241
Mason Farm WWTP
Orange County

Dear Mr. Kerwin:

Division personnel have reviewed and approved your application for renewal of the subject permit. Accordingly, we are forwarding the attached NPDES discharge permit. This permit is issued pursuant to the requirements of North Carolina General Statute 143-215.1 and the Memorandum of Agreement between North Carolina and the U.S. Environmental Protection Agency dated May 9, 1994 (or as subsequently amended).

The Division concurs with your interpretation of the "seasonal variation" requirement for the annual Effluent Pollutant Scan.

This final permit includes following major changes from the draft permit sent to you on November 22, 2006:

- The quarterly monitoring for 2 years for CN and Hg was added to the permit. If reasonable potential to exceed applicable water quality standards exists, the limits for CN and Hg will be added to the permit (see special condition A. (12)).

If any parts, measurement frequencies or sampling requirements contained in this permit are unacceptable to you, you have the right to an adjudicatory hearing upon written request within thirty (30) days following receipt of this letter. This request must be in the form of a written petition, conforming to Chapter 150B of the North Carolina General Statutes, and filed with the Office of Administrative Hearings (6714 Mail Service Center, Raleigh, North Carolina 27699-6714). Unless such demand is made, this decision shall be final and binding.

Please note that this permit is not transferable except after notice to the Division. The Division may require modification or revocation and reissuance of the permit. This permit does not affect the legal requirements to obtain other permits which may be required by the Division of Water Quality or permits required by the Division of Land Resources, the Coastal Area Management Act or any other Federal or Local governmental permit that may be required. If you have any questions concerning this permit, please contact Sergei Chernikov at telephone number (919) 733-5083, extension 594.

Sincerely,

for Alan W. Klimek, P.E.

cc: Central Files
NPDES Files
Raleigh Regional Office / Surface Water Protection
Aquatic Toxicology Unit
Mr. Roosevelt Childress, EPA Region IV

N. C. Division of Water Quality / NPDES Unit
1617 Mail Service Center, Raleigh, NC 27699-1617
Internet: h2o.enr.state.nc.us

Phone: (919) 733-5083
fax: (919) 733-0719
DENR Customer Service Center: 1 800 623-7748

STATE OF NORTH CAROLINA
DEPARTMENT OF ENVIRONMENT AND NATURAL RESOURCES
DIVISION OF WATER QUALITY

PERMIT

TO DISCHARGE WASTEWATER UNDER THE

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM

In compliance with the provision of North Carolina General Statute 143-215.1, other lawful standards and regulations promulgated and adopted by the North Carolina Environmental Management Commission, and the Federal Water Pollution Control Act, as amended, the

Orange Water and Sewer Authority

is hereby authorized to discharge wastewater from a facility located at the

Mason Farm WWTP
100 Old Mason Farm Road
Chapel Hill
Orange County

to receiving waters designated as Morgan Creek in the Cape Fear River Basin in accordance with effluent limitations, monitoring requirements, and other conditions set forth in Parts I, II, III, and IV hereof.

The permit shall become effective March 1, 2007.

This permit and the authorization to discharge shall expire at midnight on July 31, 2011.

Signed this day January 22, 2007.

for

Alan W. Klimek, P.E., Director
Division of Water Quality
By Authority of the Environmental Management Commission

SUPPLEMENT TO PERMIT COVER SHEET

All previous NPDES Permits issued to this facility, whether for operation or discharge are hereby revoked, and as of this issuance, any previously issued permit bearing this number is no longer effective. Therefore, the exclusive authority to operate and discharge from this facility arises under the permit conditions, requirements, terms, and provisions included herein.

The Orange Water and Sewer Authority is hereby authorized to:

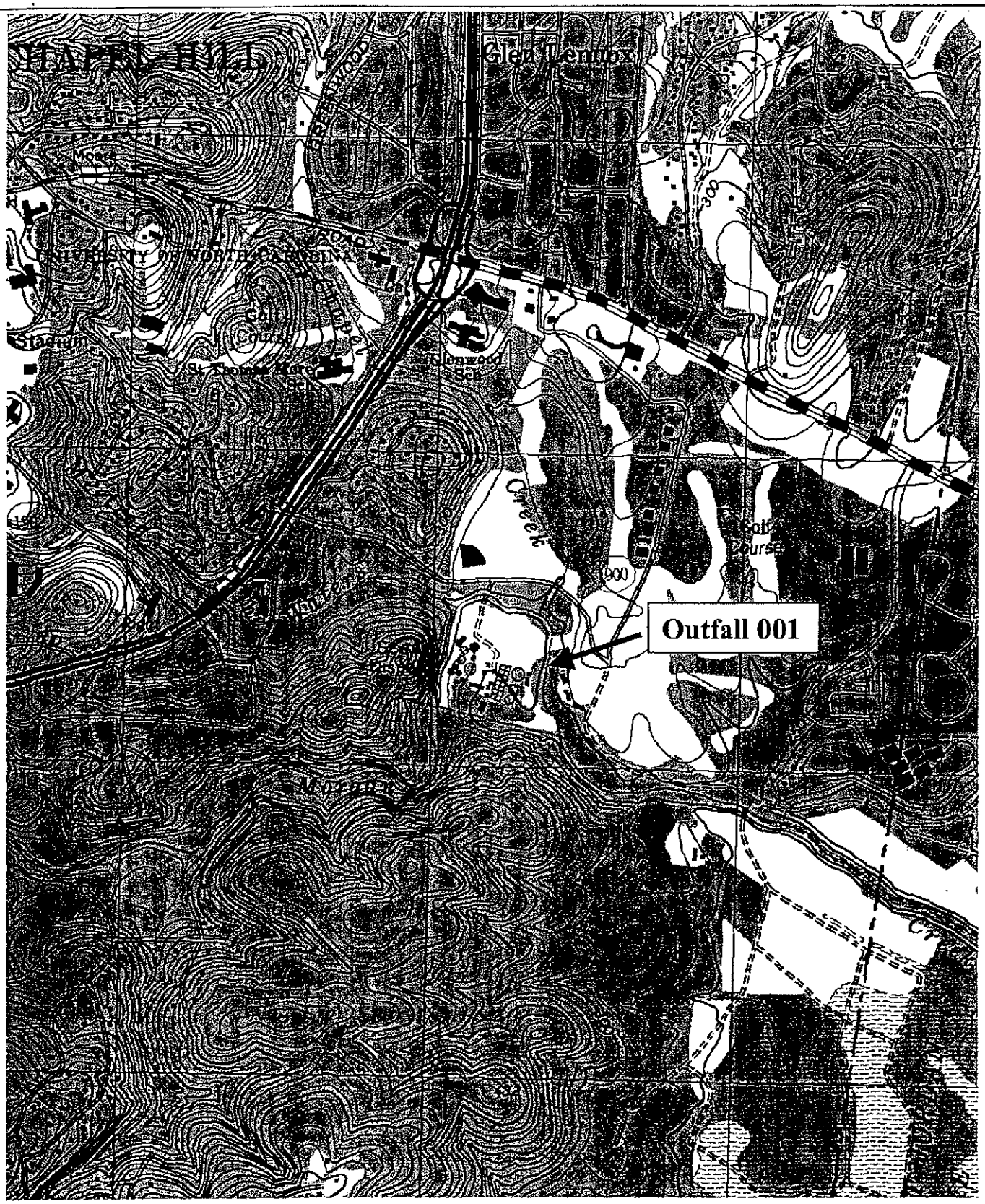
1. Continue to operate an existing 12.0 MGD wastewater treatment facility located at the Mason Farm Wastewater Treatment Plant [100 Old Mason Farm Road, Chapel Hill] in Orange County. This facility discharges via outfall 001 and includes the following wastewater treatment components:

   - Influent screen
   - Grit removal
   - Primary clarifiers
   - Trickling filters
   - Aeration basins
   - Secondary clarifiers
   - Primary sludge fermenter
   - UV disinfection system
   - Anaerobic digester
   - Gravity belt thickeners
   - Sludge holding tank and;

2. After submitting an Engineer's Cetification, operate the facility at the 14.5 MGD design flow.

3. Upon receipt of a wastewater reuse permit from the Non-Discharge Permitting Unit of the Division of Water Quality construct and operate a water reclamation and distribution system to provide beneficial reuse of treated effluent from the treatment plant, as approved and conditioned pursuant to the reuse permit, and

4. Discharge from said treatment works into Morgan Creek, a class WS-IV NSW water in the Cape Fear River Basin, at the location specified on the attached map.



OWASA - Mason Farm WWTP

State Grid/Quad:	D22NE / Chapel Hill	**Latitude:**	35° 53' 43" N
		Longitude:	79° 01' 26" W
Receiving Stream:	Morgan Creek	**Drainage Basin:**	Cape Fear
Stream Class:	WS-IV, NSW	**Sub-Basin:**	03-06-06

North



Facility Location
not to scale

NPDES Permit No. NC0025241
Orange County

A. (1) EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS (12.0 MGD)

Beginning on the effective date of this permit and lasting until expansion above 12.0 MGD or expiration, the Permittee is authorized to discharge treated wastewater from Outfall 001. Such discharges shall be limited and monitored by the Permittee as specified below:

EFFLUENT CHARACTERISTICS	EFFLUENT LIMITATIONS			MONITORING REQUIREMENTS		
	Monthly Average	Weekly Average	Daily Maximum	Measurement Frequency	Sample Type	Sample Location
Total Monthly Flow (MG)	Monitor and Report			Monthly	Recorded or Calculated	Influent or Effluent
Flow	12.0 MGD			Continuous	Recording	Influent or Effluent
CBOD, 5-day, 20°C [1] (April 1 – October 31)	4.0 mg/L	6.0 mg/L		Daily	Composite	Influent & Effluent
CBOD, 5-day, 20°C [1] (November 1 – March 31)	8.0 mg/L	12.0 mg/L		Daily	Composite	Influent & Effluent
Total Suspended Solids[1]	30.0 mg/L	45.0 mg/L		Daily	Composite	Influent & Effluent
NH_3 as N (April 1 – October 31)	2.0 mg/L	6.0 mg/L		Daily	Composite	Effluent
NH_3 as N (November 1 – March 31)	4.0 mg/L	12.0 mg/L		Daily	Composite	Effluent
Temperature				Daily	Grab	Effluent
Conductivity				Daily	Composite	Effluent
pH[2]				Daily	Grab	Effluent
Total Residual Chlorine			18 µg/L	Daily	Grab	Effluent
Fecal Coliform (geometric mean)	200/100 mL	400/100 mL		Daily	Grab	Effluent
Dissolved Oxygen[3]				Daily	Grab	Effluent
Total Phosphorus[4]	0.6 mg/L			Weekly	Composite	Effluent
Total Phosphorus Load[5]	Monitor & Report (pounds/month) Monitor & Report (pounds/year)			Monthly Annually	Calculated Calculated	Effluent Effluent
Total Nitrogen (NO_2+NO_3+TKN)	Monitor & Report (mg/L)			Weekly	Composite	Effluent
TKN (mg/L)	Monitor & Report (mg/L)			Weekly	Composite	Effluent
NO_3-N + NO_2-N (mg/L)	Monitor & Report (mg/L)			Weekly	Composite	Effluent
Total Nitrogen Load[6] (NO_2+NO_3+TKN)	Monitor & Report (pounds/month) Monitor & Report (pounds/year)			Monthly Annually	Calculated Calculated	Effluent Effluent
Total Copper				Quarterly	Composite	Effluent
Total Zinc				Quarterly	Composite	Effluent
Chronic Toxicity[7]				Quarterly	Composite	Effluent
CN[8]				Quarterly	Composite	Effluent
Total Hg[8]				Quarterly	Composite	Effluent

Notes:

1. The monthly average effluent CBOD5 and TSS concentrations shall not exceed 15% of the respective influent value (85% removal).
2. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.
3. The daily average effluent dissolved oxygen concentration shall not be less than 6.0 mg/L.
4. Compliance shall be based upon quarterly averages of weekly samples.

5. See condition A. (4) for calculation instructions.
6. See condition A. (5) for calculation instructions.
7. Chronic Toxicity (*Ceriodaphnia*) at 90 %; February, May, August and November [see condition A. (8)].
8. Monitoring shall be conducted for two years. If reasonable potential to exceed applicable water quality standards exists, the limit will be added to the permit.

There shall be no discharge of floating solids or visible foam in other than trace amounts. Instream monitoring shall be conducted as specified in condition A. (3).

A. (2) EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS (14.5 MGD)

Beginning upon expansion above 12.0 MGD and lasting until expiration, the Permittee is authorized to discharge treated wastewater from Outfall 001. Such discharges shall be limited and monitored by the Permittee as specified below:

EFFLUENT CHARACTERISTICS	EFFLUENT LIMITATIONS			MONITORING REQUIREMENTS		
	Monthly Average	Weekly Average	Daily Maximum	Measurement Frequency	Sample Type	Sample Location
Total Monthly Flow (MG)	Monitor and Report			Monthly	Recorded or Calculated	Influent or Effluent
Flow	14.5 MGD			Continuous	Recording	Influent or Effluent
CBOD, 5-day, 20°C [1] (April 1 – October 31)	4.0 mg/L	6.0 mg/L		Daily	Composite	Influent & Effluent
CBOD, 5-day, 20°C [1] (November 1 – March 31)	8.0 mg/L	12.0 mg/L		Daily	Composite	Influent & Effluent
Total Suspended Solids[1]	30.0 mg/L	45.0 mg/L		Daily	Composite	Influent & Effluent
NH_3 as N (April 1 – October 31)	1.0 mg/L	3.0 mg/L		Daily	Composite	Effluent
NH_3 as N (November 1 – March 31)	2.0 mg/L	6.0 mg/L		Daily	Composite	Effluent
Temperature				Daily	Grab	Effluent
Conductivity				Daily	Composite	Effluent
pH[2]				Daily	Grab	Effluent
Total Residual Chlorine			18 µg/L	Daily	Grab	Effluent
Fecal Coliform (geometric mean)	200/100 mL	400/100 mL		Daily	Grab	Effluent
Dissolved Oxygen[3]				Daily	Grab	Effluent
Total Phosphorus	Monitor & Report (mg/L)			Weekly	Composite	Effluent
Total Phosphorus Load[4]	Monitor & Report (pounds/month) 11,030 pounds/year			Monthly Annually	Calculated Calculated	Effluent Effluent
TKN (mg/L)	Monitor & Report (mg/L)			Weekly	Composite	Effluent
NO_3-N + NO_2-N (mg/L)	Monitor & Report (mg/L)			Weekly	Composite	Effluent
Total Nitrogen (NO_2+NO_3+TKN)	Monitor & Report (mg/L)			Weekly	Composite	Effluent
Total Nitrogen Load[5] (NO_2+NO_3+TKN)	Monitor & Report (pounds/month) 409,448 pounds/year			Monthly Annually	Calculated Calculated	Effluent Effluent
Total Copper				2/month	Composite	Effluent
Total Zinc				2/month	Composite	Effluent
Chronic Toxicity[6]				Quarterly	Composite	Effluent
CN[7]				Quarterly	Composite	Effluent
Total Hg[7]				Quarterly	Composite	Effluent

Notes:

1. The monthly average effluent CBOD5 and TSS concentrations shall not exceed 15% of the respective influent value (85% removal).
2. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.
3. The daily average effluent dissolved oxygen concentration shall not be less than 6.0 mg/L.

4. See condition A. (4) for calculation instructions.
5. See condition A. (5) for calculation instructions.
6. Chronic Toxicity (*Ceriodaphnia*) at 90 %; February, May, August and November; refer to condition A. (9).
7. Monitoring shall be conducted for two years. If reasonable potential to exceed applicable water quality standards exists, the limit will be added to the permit.

There shall be no discharge of floating solids or visible foam in other than trace amounts.
Instream monitoring shall be conducted as specified in Special Condition A (3).

A. (3) INSTREAM MONITORING REQUIREMENTS

Sample Location[1]	Measurement Frequency		Parameters[2]
UP1 DN1 DN2	October - May	Weekly	Temperature Dissolved Oxygen Conductivity Fecal Coliform
	June - September	3/week	
UP1 DN1 DN2	June-September	Once every 2 weeks[3]	Total Phosphorous PO_4 NH_3 Total Kjendahl Nitrogen NOx
DN3 DN4	April-October	Monthly[3]	Temperature Dissolved Oxygen Conductivity Fecal Coliform Secchi depth Chlorophyl-a

Notes:

1. Sample locations:
   UP1 – Upstream location at Morgan Creek at the bridge at the plant entrance
   DN1 – Downstream location #1 at Morgan Creek at the cement ford downstream of outfall
   DN2 – Downstream location #2 at Morgan Creek at the Farrington Road bridge
   DN3 – Downstream location # 3 at Morgan Creek at the mouth of Morgan Creek
   DN4 - Downstream Location # 4 at Jordan Lake at the old railroad grade
   - Sampling of temperature and DO at the surface and one-meter intervals to a depth equal to the Secchi depth. If the Secchi depth is less than 1.0 meters, sample at 0.5-meter intervals.
   - As a participant in the Upper Cape Fear River Basin Association, the instream monitoring requirements as stated above are waived. Should your membership in the association be terminated, you shall notify the Division immediately and the instream monitoring requirements specified in your permit shall be reinstated.
2. Upstream and downstream samples shall be grab samples.
3. Monthly sampling at DN3 and DN4 should be separated at least two weeks within the appropriate month.

A. (4) CALCULATION OF TOTAL PHOSPHORUS LOADS

a. The Permittee shall calculate monthly and annual TP Loads as follows:

i. Monthly TP Load (pounds/month) = TP x TMF x 8.34

where:

TP = the average Total Phosphorus concentration (mg/L) of the composite samples collected during the month
TMF = the Total Monthly Flow of wastewater discharged during the month (MG/mo)
8.34 = conversion factor, from (mg/L x MG) to pounds

ii. Annual TP Load (pounds/year) = Sum of the 12 Monthly TP Loads for the calendar year

b. The Permittee shall report monthly Total Phosphorus results (mg/L and pounds/month) in the discharge monitoring report for that month and shall report each year's annual results (pounds/year) in the December report for that year.

A. (5) CALCULATION OF TOTAL NITROGEN LOADS

a. The Permittee shall calculate monthly and annual TN Loads as follows:

i. Monthly TN Load (pounds/month) = TN x TMF x 8.34

where:

TN = the average Total Nitrogen concentration (mg/L) of the composite samples collected during the month

TMF = the Total Monthly Flow of wastewater discharged during the month (MG/mo)

8.34 = conversion factor, from (mg/L x MG) to pounds

ii. Annual TN Load (pounds/year) = Sum of the 12 Monthly TN Loads for the calendar year

b. The Permittee shall report monthly Total Nitrogen results (mg/L and pounds/month) in the discharge monitoring report for that month and shall report each year's annual results (pounds/year) in the December report for that year.

A. (6) ANNUAL LIMITS FOR TOTAL PHOSPHORUS

a. Total Phosphorus (TP) allocations and TP Load limits are annual limits and are applied for the calendar year.

b. For any given calendar year, the Permittee shall be in compliance with the annual TP Load limit in this Permit, if the Permittee's annual TP Load is less than or equal to said limit.

A. (7) ANNUAL LIMITS FOR TOTAL NITROGEN

a. Total Nitrogen (TN) allocations and TN Load limits are annual limits and are applied for the calendar year.

b. For any given calendar year, the Permittee shall be in compliance with the annual TN Load limit in this Permit, if the Permittee's annual TN Load is less than or equal to said limit.

A. (8) CHRONIC TOXICITY PERMIT LIMIT (Quarterly) @ 12.0 MGD

The effluent discharge shall at no time exhibit observable inhibition of reproduction or significant mortality to *Ceriodaphnia dubia* at an effluent concentration of 90 %.

The permit holder shall perform at a minimum, *quarterly* monitoring using test procedures outlined in the "North Carolina *Ceriodaphnia* Chronic Effluent Bioassay Procedure," Revised February 1998, or subsequent versions or "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions. The tests will be performed *during the months of* February, May, August and November. Effluent sampling for this testing shall be performed at the NPDES permitted final effluent discharge below all treatment processes.

If the test procedure performed as the first test of any single quarter results in a failure or ChV below the permit limit, then multiple-concentration testing shall be performed at a minimum, in each of the two following months as described in "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions.

The chronic value for multiple concentration tests will be determined using the geometric mean of the highest concentration having no detectable impairment of reproduction or survival and the lowest concentration that does have a detectable impairment of reproduction or survival. The definition of "detectable impairment," collection methods, exposure regimes, and further statistical methods are specified in the "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions.

All toxicity testing results required as part of this permit condition will be entered on the Effluent Discharge Monitoring Form (MR-1) for the months in which tests were performed, using the parameter code TGP3B for the pass/fail results and THP3B for the Chronic Value. Additionally, DWQ Form AT-3 (original) is to be sent to the following address:

Attention: NC DENR / DWQ / Environmental Sciences Section
1621 Mail Service Center
Raleigh, North Carolina 27699-1621

Completed Aquatic Toxicity Test Forms shall be filed with the Environmental Sciences Branch no later than 30 days after the end of the reporting period for which the report is made.

Test data shall be complete, accurate, include all supporting chemical/physical measurements and all concentration/response data, and be certified by laboratory supervisor and ORC or approved designate signature. Total residual chlorine of the effluent toxicity sample must be measured and reported if chlorine is employed for disinfection of the waste stream.

Should there be no discharge of flow from the facility during a month in which toxicity monitoring is required, the permittee will complete the information located at the top of the aquatic toxicity (AT) test form indicating the facility name, permit number, pipe number, county, and the month/year of the report with the notation of "No Flow" in the comment area of the form. The report shall be submitted to the Environmental Sciences Branch at the address cited above.

Should the permittee fail to monitor during a month in which toxicity monitoring is required, monitoring will be required during the following month.

Should any test data from this monitoring requirement or tests performed by the North Carolina Division of Water Quality indicate potential impacts to the receiving stream, this permit may be re-opened and modified to include alternate monitoring requirements or limits.

NOTE: Failure to achieve test conditions as specified in the cited document, such as minimum control organism survival, minimum control organism reproduction, and appropriate environmental controls, shall constitute an invalid test and will require immediate follow-up testing to be completed no later than the last day of the month following the month of the initial monitoring.

A. (9) CHRONIC TOXICITY PERMIT LIMIT (Quarterly) @ 14.5 MGD

The effluent discharge shall at no time exhibit observable inhibition of reproduction or significant mortality to *Ceriodaphnia dubia* at an effluent concentration of 90 %.

The permit holder shall perform at a minimum, *quarterly* monitoring using test procedures outlined in the "North Carolina *Ceriodaphnia* Chronic Effluent Bioassay Procedure," Revised February 1998, or subsequent versions or "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions. The tests will be performed *during the months of* February, May, August and November. Effluent sampling for this testing shall be performed at the NPDES permitted final effluent discharge below all treatment processes.

If the test procedure performed as the first test of any single quarter results in a failure or ChV below the permit limit, then multiple-concentration testing shall be performed at a minimum, in each of the two following months as described in "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions.

The chronic value for multiple concentration tests will be determined using the geometric mean of the highest concentration having no detectable impairment of reproduction or survival and the lowest concentration that does have a detectable impairment of reproduction or survival. The definition of "detectable impairment," collection methods, exposure regimes, and further statistical methods are specified in the "North Carolina Phase II Chronic Whole Effluent Toxicity Test Procedure" (Revised-February 1998) or subsequent versions.

All toxicity testing results required as part of this permit condition will be entered on the Effluent Discharge Monitoring Form (MR-1) for the months in which tests were performed, using the parameter code TGP3B for the pass/fail results and THP3B for the Chronic Value. Additionally, DWQ Form AT-3 (original) is to be sent to the following address:

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Raleigh, North Carolina 27699-1621

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Test data shall be complete, accurate, include all supporting chemical/physical measurements and all concentration/response data, and be certified by laboratory supervisor and ORC or approved designate signature. Total residual chlorine of the effluent toxicity sample must be measured and reported if chlorine is employed for disinfection of the waste stream.

Should there be no discharge of flow from the facility during a month in which toxicity monitoring is required, the permittee will complete the information located at the top of the aquatic toxicity (AT) test form indicating the facility name, permit number, pipe number, county, and the month/year of the report with the notation of "No Flow" in the comment area of the form. The report shall be submitted to the Environmental Sciences Branch at the address cited above.

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Should any test data from this monitoring requirement or tests performed by the North Carolina Division of Water Quality indicate potential impacts to the receiving stream, this permit may be re-opened and modified to include alternate monitoring requirements or limits.

NOTE: Failure to achieve test conditions as specified in the cited document, such as minimum control organism survival, minimum control organism reproduction, and appropriate environmental controls, shall constitute an invalid test and will require immediate follow-up testing to be completed no later than the last day of the month following the month of the initial monitoring.

A. (10) EFFLUENT POLLUTANT SCAN

The Permittee shall perform an annual Effluent Pollutant Scan for all parameters listed in the attached table. The Permittee shall perform an annual Effluent Pollutant Scan for all parameters listed in the attached table. The analytical methods shall be in accordance with 40 CFR Part 136 and shall be sufficiently sensitive to determine whether parameters are present in concentrations greater than applicable standards and criteria. Samples shall **represent seasonal variations.** Unless otherwise indicated, metals shall be analyzed as "total recoverable."

Ammonia (as N)
Chlorine (total residual, TRC)
Dissolved oxygen
Nitrate/Nitrite
Kjeldahl nitrogen
Oil and grease
Phosphorus
Total dissolved solids
Hardness
Antimony
Arsenic
Beryllium
Cadmium
Chromium
Copper
Lead
Mercury
Nickel
Selenium
Silver
Thallium
Zinc
Cyanide
Total phenolic compounds

Volatile organic compounds:

Acrolein
Acrylonitrile
Benzene
Bromoform
Carbon tetrachloride
Chlorobenzene
Chlorodibromomethane
Chloroethane
2-chloroethylvinyl ether
Chloroform
Dichlorobromomethane
1,1-dichloroethane
1,2-dichloroethane
Trans-1,2-dichloroethylene
1,1-dichloroethylene
1,2-dichloropropane
1,3-dichloropropylene
Ethylbenzene
Methyl bromide
Methyl chloride
Methylene chloride
1,1,2,2-tetrachloroethane
Tetrachloroethylene
Toluene
1,1,1-trichloroethane
1,1,2-trichloroethane
Trichloroethylene
Vinyl chloride

Acid-extractable compounds:

P-chloro-m-creso
2-chlorophenol
2,4-dichlorophenol
2,4-dimethylphenol
4,6-dinitro-o-cresol
2,4-dinitrophenol
2-nitrophenol
4-nitrophenol
Pentachlorophenol
Phenol
2,4,6-trichlorophenol

Base-neutral compounds:

Acenaphthene
Acenaphthylene
Anthracene
Benzidine
Benzo(a)anthracene
Benzo(a)pyrene
3,4 benzofluoranthene
Benzo(ghi)perylene
Benzo(k)fluoranthene
Bis (2-chloroethoxy) methane
Bis (2-chloroethyl) ether
Bis (2-chloroisopropyl) ether
Bis (2-ethylhexyl) phthalate
4-bromophenyl phenyl ether
Butyl benzyl phthalate
2-chloronaphthalene
4-chlorophenyl phenyl ether
Chrysene
Di-n-butyl phthalate
Di-n-octyl phthalate
Dibenzo(a,h)anthracene
1,2-dichlorobenzene
1,3-dichlorobenzene
1,4-dichlorobenzene
3,3-dichlorobenzidine
Diethyl phthalate
Dimethyl phthalate
2,4-dinitrotoluene
2,6-dinitrotoluene
1,2-diphenylhydrazine
Fluoranthene
Fluorene
Hexachlorobenzene
Hexachlorobutadiene
Hexachlorocyclo-pentadiene
Hexachloroethane
Indeno(1,2,3-cd)pyrene
Isophorone
Naphthalene
Nitrobenzene
N-nitrosodi-n-propylamine
N-nitrosodimethylamine
N-nitrosodiphenylamine
Phenanthrene
Pyrene
1,2,4-trichlorobenzene

Test results shall be reported to the Division in DWQ Form- A MR-PPA1 or in a form approved by the Director within 90 days of sampling. The report shall be submitted to the following address:

NC DENR / DWQ / Water Quality Section
Central Files
1617 Mail Service Center, Raleigh, North Carolina 27699-1617.

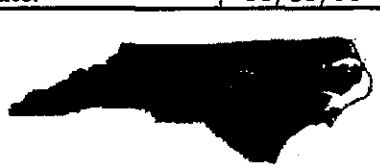
A. (11) PERMIT RE-OPENER: NUTRIENT CONTROLS

Pursuant to N.C. Gen. Stat. Section 143-215.1 and the implementing rules found in Title 15A of the North Carolina Administrative Code, Subchapter 2H, specifically, 15A NCAC 2H.0112(b)(1) and 2H.0114(a), and Part II sections B-12 and B-13 of this Permit, the Director of DWQ may reopen this permit to require supplemental nutrient monitoring of the discharge. The additional monitoring will be to support water quality modeling efforts within the Cape Fear River Basin, and shall be consistent with a monitoring plan developed jointly by the Division and affected stakeholders.

A. (12) CN and Hg MONITORING REQUIREMENTS

The permittee shall compile monitoring data for CN and Hg into report after 2 years of monitoring have been completed and submit the report with all available data to the Division for review.

DENR/DWQ
FACT SHEET FOR NPDES PERMIT DEVELOPMENT
NPDES Permit NC0025241

Facility Information			
Applicant/Facility Name:	Orange Water and Sewer Authority/ Mason Farm WWTP		
Applicant Address:	P.O. Box 366 Carrboro, N.C. 27510		
Facility Address:	100 Mason Farm Road Chapel Hill, N.C.		
Permitted Flow	12.0 MGD; phased to 14.5 MGD		
Type of Waste:	Domestic and industrial		
Facility/Permit Status:	Modification		
Facility Classification	IV		
County:	Orange		
Miscellaneous			
Receiving Stream:	Morgan Creek	Regional Office:	Raleigh
Stream Classification:	WS-IV NSW	USGS Topo Quad:	D22NE/Chapel Hill
303(d) Listed?:	Yes	Permit Writer:	Sergei Chernikov
Subbasin:	03-06-06	Date:	11/06/06
Drainage Area (mi2):	40.7		
Summer 7Q10 (cfs)	1.0		
Winter 7Q10 (cfs):	3.0		
Average Flow (cfs):	42		
IWC (%):	95		
Primary SIC Code:	4952		

SUMMARY
This facility is a major municipal treatment plant operating in Orange County that serves 80,000 in Chapel Hill, Carrboro and UNC. The facility is served by a separate sewer collection system. Facility has a permitted flow of phased flow of 12/14.5 MGD discharging into Morgan Creek (biologically impaired). The upgrade and expansion is expected to be completed in the fall of 2007. For the past 2 years annual average flow varied within the range of 7.7-8.3 MGD. The City of Chapel Hill has an inactive pretreatment program. The permit will require the City to continue to implement its existing pretreatment program.

REASONABLE POTENTIAL ANALYSIS:
Reasonable potential analyses were conducted for Cu and Zn (see attached).

TOXICITY TESTING:

Type of Toxicity Test:	Chronic P/F
Existing Limit:	001: Chronic P/F @ 90%
Recommended Limit:	001: Chronic P/F @ 90%
Monitoring Schedule:	February, May, August, and November

The facility has been consistently passing its WET tests with one exception (see attached).

COMPLIANCE SUMMARY:
DMRs have been reviewed for the period January 2003 through April 2006. Facility has a good compliance record. During review period following NOVs have been issued: 5/31/06 – TP; 08/09/06 – release of untreated sewage; 05/29/03 – TRC.

A compliance evaluation inspection conducted on 03/02/06 found that facility is in compliance.

INSTREAM MONITORING:
Instream monitoring is required for temperature, dissolved oxygen, fecal coliforms, conductivity, TP, PO_4, NH_3, TKN, NOx, Secchi Depth, and Chlorophyl-a. The facility is a

member of the Upper Cape Fear Monitoring Coalition, and all their instream monitoring is conditionally waived for the duration of their membership in the Coalition.

Monitoring data indicates that the facility has limited negative impact on the stream.

PROPOSED CHANGES:
In accordance with the Division's new Permitting Strategy for Cape Fear River Basin the following changes have been made:

1. Monitoring frequencies for TN were increased to Weekly.
2. Monitoring for TKN and NO_3-N + NO_2-N was added to the permit.
3. Nutrient Controls Re-Opener condition was added to the permit (see A. (11)).
4. Total monthly flow reporting was added to the permit
5. Load monitoring for TN and TP was added to the permit (see A. (1)).

The existing monitoring frequency for zinc and copper remains unchanged. Due to the facility's toxicity record, limits for these action level standards will not be implemented, but monitoring will remain.

PROPOSED SCHEDULE FOR PERMIT ISSUANCE:
Draft Permit to Public Notice: November 22, 2006 (est.)
Permit Scheduled to Issue: January 14, 2007 (est.)

STATE CONTACT:
If you have any questions on any of the above information or on the attached permit, please contact Sergei Chernikov at (919) 733-5038 ext. 594.

CHANGES TO THE FINAL PERMIT:

Quarterly monitoring for 2 years for CN and Hg was added. The monitoring will be deleted in the future, if the permittee provides updated effluent data that shows no sampling events above detection level for these parameters. If reasonable potential to exceed applicable water quality standards exists, limits for CN and Hg will be added to the permit.

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Joe Rohrbacher, Paul Pitt, Chris White

DATE: May 14, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Operational Transition and Optimization Assistance

Technical Memorandum – Process Transition Plan

1. INTRODUCTION

Orange Water and Sewer Authority (OWASA) operates the Mason Farm Wastewater Treatment Plant (MFWWTP), an advanced treatment facility which discharges treated effluent to Morgan Creek, a tributary to Jordan Lake and the Cape Fear River. The most recent expansion to the Mason Farm WWTP increased the permitted maximum month capacity of the plant to 14.5 million gallons per day (mgd). The design also provided for a rated hydraulic peak capacity of 43.5 mgd with all facilities in service. As part of the expansion project, the plant was also upgraded to address anticipated nutrient limits associated with the Jordan Lake Nutrient Strategy. The final Jordan Lake rules require that the Mason Farm WWTP meet effluent annual mass limits for Total Nitrogen (TN) and Total Phosphorus (TP) equivalent to performance at concentrations of 3.0 mg/l and 0.23 mg/l, respectively, at the 14.5 mgd maximum month capacity.

OWASA engaged Hazen and Sawyer to perform a Hydraulic and Treatment Capacity Study for the current facilities in light of the proposed additional limitations on discharge of nutrients. A draft report was issued in March 2010 recommending improvements to address plant capacity limitations and nutrient reduction improvements to meet the final Jordan Lake Nutrient Strategy. The report identified configuring the existing activated sludge process to a step feed process as a low capital cost strategy to decrease effluent TN concentrations while reducing operational costs. In addition, the report also identified that current biological phosphorus removal (BPR) performance was poor

and OWASA is heavily relying on chemical phosphorus removal (CPR) to meet their current effluent TP limits.

The purpose of this memorandum is to present a process transition plan to address reconfiguring the existing aeration basins to the proposed step feed BNR process. and to optimize current WWTP operations for TP and TN removal while reducing chemical usage and operational costs at the Mason Farm WWTP. It is fully anticipated that the process transition plan will undergo changes as the plan is being implemented and the WWTP process responds to the changes. Hazen and Sawyer will work closely with the Mason Farm WWTP staff during the transition and this document will be updated as modifications to the plan are identified.

2. GOALS OF PROCESS TRANSITION PLAN

The most important goal for this process transition plan is to maintain permit compliance, specifically with the facility's current permit limits, which are summarized in Table 1. The current plant flowrate is approximately 8 mgd, and annual average TN and TP concentrations of 16.8 mg/L and 0.42 mg/L, respectively, need to be achieved to meet the mass limits summarized in Table 1. The TN mass load limit will be reduced to 134,375 lb/year beginning in 2016 as part of the Jordan Lake Nutrient Management Study.

Secondary goals of the process transition plan include reducing OWASA's dependence on chemical addition for phosphorus removal and to better utilize available carbon in the wastewater to increase denitrification in the secondary process. The average alum (48%) feed rate reported between November 2008 and August 2009 (the period studied during development of the Hydraulic and Treatment Capacity Study) was 950 gallons per day (gpd), which equates to an overall alum dose of 78 mg/L. It is reported that approximately 2/3 of the alum feed is to the filtrate with the remaining 1/3 fed to the mixed liquor suspended solids (MLSS) upstream of distribution to the secondary clarifiers.

In addition to the substantial alum feed OWASA is adding an average of 625 gpd of 20% acetic acid solution to the nutrified sludge (NSL) basins to promote RAS denitrification and biological phosphorus removal. Poor BPR performance has been observed as evidenced by the high alum usage to meet OWASA's effluent TP limits. The process transition plan will evaluate the need for acetic acid addition.

The design of the recent upgrade and expansion project anticipated that denitrification filters would be utilized to achieve compliance with the lower total nitrogen limits with little denitrification occurring in the secondary process. The process transition plan will develop a strategy for modifying current operations to provide for step feed biological nutrient removal treatment to reduce effluent TN concentrations while maintaining compliance with the current permit. In addition, the plan will address potential reductions in alkalinity feed associated with increased denitrification in the secondary process.

Table 1

Mason Farm WWTP Current Permitted Effluent Limits

	Value	Period	Sampling Frequency
Flow (mgd)	14.5	Monthly	Continuous
$cBOD_5$ (mg/L) (April-October)	4.0	Monthly	Daily
$cBOD_5$ (mg/L) (November-March)	8.0	Monthly	Daily
TSS (mg/L)	30.0	Monthly	Daily
NH_3-N(mg/L) (April-October)	1.0	Monthly	Daily
NH_3-N(mg/L) (November-March)	2.0	Monthly	Daily
Total Kjeldahl Nitrogen (mg/L)	Report	-	Weekly
Nitrate/Nitrite (mg/L)	Report	-	Weekly
Total Phosphorus (mg/L)	Report	-	Weekly
Total Phosphorus (lbs/year)	10,188[1]	Annual	Monthly
Total Nitrogen (mg/L)	Report	-	Weekly
Total Nitrogen (lbs/year)	409,448	Annual	Monthly
Dissolved Oxygen (mg/L)	> 6.0	Monthly	Daily
pH	6 – 9	Monthly	Daily
Fecal Coliform (cfu/100 mL)	200	Monthly	Daily
Total Residual Chlorine (max. µg/L)	18	Daily	Daily

[1]Effective Calendar Year 2010

3. CHEMICAL PHOSPHORUS REMOVAL

Significant quantities of alum are currently being fed at the Mason Farm WWTP to control phosphorus levels. The majority of this feed is reported to be to the solids dewatering process

filtrate. Observations indicate that BPR efficiency is poor and that limited phosphate is measured in the filtrate. The addition of excessive alum to the filtrate may result in enhanced primary clarifier performance, which could reduce the available carbon to the activated sludge system for denitrification. Excess alum addition may also lead to phosphate limiting conditions in the NSL basins, which may be detrimental to BPR performance.

3.1 CHEMICAL PHOSPHORUS REMOVAL JAR TESTING

Chemical phosphorus removal jar testing was performed on April 28, 2010 to determine the efficiency of alum addition for phosphorus removal. Chemical jar tests were performed on filtrate, mixed liquor suspended solids (MLSS) and secondary clarifier effluent (SCE). The jar testing indicated relatively poor removal of phosphorus through alum addition to the MLSS and secondary effluent; however initial ortho-phosphorus concentrations were low (0.77 mg/L for MLSS and 0.41 mg/L for SCE), and competing reactions at low ortho-phosphorus concentrations often lead to high required aluminum to phosphorus requirements.

The stoichiometric molar dosage of aluminum to phosphorus (Al:P molar ratio) is 1.0. The calculated Al:P molar ratio for the MLSS ranged from 6.7 to 9.7, and the calculated Al:P molar ratio for the SCE ranged from 4.6 to 7.3. Typical observed Al:P molar ratios at other similar plants with similar ortho-phosphate concentrations range between 2 and 5. Additional jar testing will be performed on the MLSS and SCE to confirm the required alum doses for optimized chemical usage.

More efficient utilization of alum was observed in the filtrate sample, where the Al:P molar ratio ranged from 1.3 to 1.9. The filtrate ortho-phosphate concentration was 38 mg/L prior to chemical addition, which is low for an anaerobically digested sludge and likely indicates limited biological phosphorus removal is occurring. Typical alum doses are close to stoichiometric requirements (1:1) at ortho-phosphate concentrations above 1 mg/L; however due to the elevated pH of the filtrate sample (8.4) at the time of the jar testing less efficient removal was observed. The major conclusions drawn from the chemical jar testing of alum are as follows:

- Low MLSS and secondary effluent ortho-phosphate concentrations are likely a result of alum feed to the filtrate and MLSS, and BPR efficacy is limited.

- Alum addition to the MLSS and secondary effluent is inefficient. Additional jar testing will be performed to confirm this observation.

- Filtrate phosphorus precipitation efficiency is somewhat reduced due to the high filtrate pH.

3.2 CHEMICAL PHOSPHORUS OPTIMIZATION - FILTRATE

Measured filtrate TP and ortho-phosphate (PO_4-P) concentrations vary significantly. Table 2 summarizes the available filtrate nutrient data between 2005 and 2009. Due to the limited current BPR performance, very little ortho-phosphate is currently returned to the liquid stream process through the filtrate. The initial filtrate ortho-phosphate concentration during jar testing was approximately 40 mg/L. Note this value is similar to the average and median observed values.

Target filtrate alum doses were developed based on the results of the chemical phosphorus removal jar testing. The target alum dose is based on an ortho-phosphate concentration of 40 mg/L and aluminum to phosphorus molar ratio (Al:P molar ratio) of 1.6, which was the average observed value during jar testing. The resulting alum dose is approximately 600 mg/L. The resulting alum feed rate (assuming 48% alum solution) is 30 gpd at a filtrate flow rate of 30,000 gpd. This is equivalent to a feed rate of 1 gallon of 48% alum solution per 1,000 gallons of filtrate.

Table 2

Available Filtrate Nutrient Data Summary (2005-2009)

	TKN, mg/L	NH3-N, mg/L	TP, mg/L	PO4-P, mg/L
Number of Samples	12	14	14	9
Minimum	655	852	88	8.2
Mean	1,191	1,093	251	51
Median	1,223	1,070	208	31
90th Percentile	1,380	1,291	414	148
Maximum	1,399	1,440	658	153

3.3 CHEMICAL PHOSPHORUS OPTIMIZATION – LIQUID STREAM

The alum feed rate to the liquid stream process was developed based on modifying the alum feed to the filtrate to fully remove the ortho-phosphate from the filtrate without overfeeding alum to this point in the process. The initial dose recommended herein assumes continued poor BPR performance as is currently being experienced. Improvements in BPR will allow further reductions in alum dosages. Alum feed requirements to the liquid stream were calculated to include phosphorus removal through assimilation and a target effluent ortho-phosphate concentration less

than 0.1 mg/L. Phosphorus assimilation was assumed to be 2.4% of the mixed liquor volatile suspended solids (MLVSS) wasted per day, which is a typical value for non-biological phosphorus removal systems.

Table 3 develops the recommended initial liquid stream alum feed requirement. An initial alum dose of 55 mg/L is recommended and corresponds to a daily feed rate of 688 gpd (assuming 48% alum solution) at the current 8 mgd design flow, or 86 gallons of 48% alum solution per million gallons treated. The initial alum feed rate is based on an Al:P molar ratio of 4.0 at ortho-phosphate concentrations below 1.0 mg/L. This is relatively high compared to observations at other facilities. The alum dose should be gradually reduced provided effluent TP limits are met. It is recommended that the target alum dose be decreased by 5 mg/L every week until measured effluent TP concentrations approach the required limit. Target alum doses should also be decreased by 5 mg/L every two weeks once reliable biological phosphorus removal is re-established. Reestablishment of BPR will increase the amount of soluble phosphorus release in the digesters, increasing filtrate phosphorus. Adjustments to the filtrate alum feed may be required, and it is recommended that 1 gallon of 48% alum be added to the filtrate for every 40 mg/L of ortho-phosphate in and 1,000 gallons of filtrate.

Table 3

Liquid Stream Target Alum Requirement

Parameter	Value
Flow, MGD	8.0
Influent PO_4-P Concentration, mg/L	5.0
Effluent PO_4-P Concentration, mg/L	< 0.10
Influent TP Loading, lb/d	334
PO_4-P Assimilation by Biomass, lb/d	150
Effluent PO_4-P Loading, lb/d	~ 0
TP Removed Chemically, lb/d	184
Al:P Molar Ratio, PO4-P >1	1.0
Al:P Molar Ratio, PO4-P <1	4.0
Alum Dose, mg/L	55
Alum Flow, gpd	**688**

The Hydraulic and Treatment Capacity Study recommended providing for chemical phosphorus

removal feed upstream of the filters to provide further chemical optimization, improve phosphorus removal reliability and reduce effluent alkalinity to reduce reclaimed water sulfuric acid feed requirements. The preliminary jar test data indicates better CPR efficiency, as expected, when alum is fed to secondary effluent upstream of the filters. It is recommended that 20 mg/L of the liquid stream target alum dose be provided upstream of the filters once this feed point is installed.

4. BIOLOGICAL PHOSPHORUS REMOVAL

The Mason Farm WWTP utilizes the OWASA process for biological phosphorus removal. This process relies on the addition of fermentate and acetic acid to the RAS in the NSL basins to provide carbon for biological phosphorus release. The released soluble phosphorus is assimilated by poly-phosphate accumulating organisms (PAOs) under aerobic conditions in the aeration basins (luxury phosphorus uptake). BPR performance has recently been unreliable, resulting in substantial chemical phosphorus feed, despite the addition of substantial acetic acid to the process.

4.1 BIOLOGICAL PHOSPHORUS UPTAKE AND RELEASE TESTING

Biological phosphorus uptake and release (PUR) testing was performed on April 27, 2010 to determine the efficiency of the BPR process at the Mason Farm WWTP. The testing consisted of batch tests in a 3.5 liter reactor filled with return activated sludge (RAS) collected upstream on the NSL basins influent channel. Four separate tests were performed to determine the impacts of various carbon sources on BPR performance. The PUR testing was performed for carbon sources including acetate, fermentate (gravity belt thickener filtrate) and acetate + fermentate. A control reactor without carbon addition was also run.

The results from the PUR testing indicate relatively poor BPR was occurring during the testing. Table 4 compares the results from this testing with results from other Hazen and Sawyer PUR tests and the range of values cited in the WERF publication Factors Influencing the Reliability of Enhanced Biological Phosphorus Removal (2005).

Table 4

Phosphate Release and Uptake Testing Results

	Acetate	Fermentate	Fermentate + Acetate	H&S Observed Values	WERF Observed Values
P Release Rate (mg P/g VSS/hr)	0.61	1.53	1.79	6.9 - 9.0	6.4 - 24.6
P Uptake Rate (mg P/g VSS/hr)	0.60	0.91	1.13	2.2 – 3.6	2.4 – 5.9
Release to Uptake Ratio	1.02	1.68	1.58	2.5 – 3.4	2.3 – 4.0
P Release/COD Uptake (mg P/ mg COD)	0.18	-[1]	-[1]	0.21 – 0.30	0.22 – 0.51
Average MLVSS (mg/L)	5,600	7,450	6,720	1,000 -1,500	1,850 – 2,600

[1]Note these values could not be properly estimated due to biodegradation of fermentate COD

4.2 FERMENTATE IMPACTS

Supplemental sampling indicates the primary fermenter effluent (pre-thickening) contains approximately 1,200 mg/L of volatile fatty acids (VFAs) as chemical oxygen demand (COD). The volatile fatty acids are distributed (based on COD contribution) as follows:

- Acetic Acid – 40%
- Propionic Acid – 40%
- Butyric Acid – 20%

The primary fermenter efficiency is estimated between 0.03 and 0.05 lbs VFA COD /lb COD solids and 0.1 to 0.2 lbs VFA COD /lb volatile suspended solids (VSS) based on supplemental sampling data from August 2009. Typical fermenter efficiencies cited in the literature are between 0.03 to 0.10 lbs VFA COD /lb COD solids and 0.1 to 0.2 lbs VFA COD /lb VSS (Latimer, et. al. 2007). The primary fermenter efficiency observed at the Mason Farm WWTP is typical of other primary fermenter installations. Continued fermentate addition to the NSL basins is recommended to provide carbon for RAS denitrification and BPR. Primary fermenter operation at a solids retention time (SRT) between 2 and 3 days should be maintained for continued efficient fermentation.

The fermentate (gravity belt thickener filtrate) VFA concentration is approximately 300 mg/L due to

the fact that the primary fermenter effluent is diluted by waste activated sludge and thickener washwater. The fermentate directly contributes approximately 900 lb COD/day of VFAs to the NSL basins. Approximately 7.5 to 10.7 mg COD is required to remove one 1 mg/L of phosphorus biologically (Grady, et. al., 1999, page 538); therefore fermentate addition to the NSL basins could potentially account for the biological removal of 85 – 120 lbs of phosphorus per day assuming efficient and reliable BPR operation.

Further breakdown and fermentation of biodegradable carbon under anaerobic conditions present in the NSL basins provides further carbon for BPR. It is recommended that one additional NSL cell be put into operation to provide additional anaerobic capacity to reduce/eliminate acetic acid addition. Increasing the detention time in the NSL basin will allow further biodegradation and fermentation of biodegradable carbon in the RAS and fermentate, potentially increasing BPR efficiency.

4.3 ACETIC ACID IMPACTS

OWASA feeds approximately 600 gpd of 20% acetic acid (HAC) to the Mason Farm WWTP. This contributes approximately 1,100 lb/d volatile fatty acids (VFAs) as chemical oxygen demand (COD). As previously discussed 7.5 to 10.7 mg COD is required to remove one 1 mg/L of phosphorus biologically; therefore acetic acid addition to the NSL basins could potentially account for the biological removal of 100 – 150 lbs of phosphorus per day assuming efficient and reliable BPR operation.

The combined acetate/fermentate volatile fatty acids are distributed (based on COD contribution) as follows:

- Acetic Acid – 73%
- Propionic Acid – 18%
- Butyric Acid – 7%

Current research suggests that similar amounts of acetic acid and propionic acid are optimal to BPR performance, and having too great an amount of acetic acid or propionic acid can reduce BPR efficiency due to glycogen accumulating organism (GAO) proliferation. Poor BPR efficiency has been observed at the Mason Farm WWTP and may possibly be caused, in part, by this

phenomenon. Reduced available soluble phosphorus (due to chemical phosphorus removal) may also limit the phosphate available to the phosphate accumulating organisms (PAOs), further selecting for GAOs.

The cost of feeding acetic acid for BPR was compared to the cost of alum and sodium hydroxide (to compensate for lost alkalinity resulting from alum addition). The approximate cost of feeding 20% acetic acid addition is $4.50/lb phosphorus removed based on a 20% acetic acid cost of $0.83/gallon. The cost of chemical phosphorus removal is $3.00/lb phosphorus removed based on a 48% alum cost of $1.11/gallon and 25% sodium hydroxide cost of $0.65/gallon. Chemical phosphorus costs are based on a aluminum to phosphorus (Al:P) molar ratio of 1:1 (stoichiometric ratio). Chemical phosphorus removal is expected to be less expensive than acetic acid addition on a mass of phosphorus removed basis.

It is recommended that OWASA gradually reduce and eliminate the feed of acetic acid to the NSL basins to investigate whether BPR performs more efficiently without acetic acid addition. It is suggested that acetic acid feed rates be initially lowered to 350 gallons per day once step feed is established and the nitrate load to the NSL basin decreases. The process should be observed, and in particular overall chemical phosphorus feed should be monitored. The alum feed rate should be adjusted as necessary to meet effluent TP limits.

5. DENITRIFICATION OPTIMIZATION

The Hydraulic and Treatment Capacity Study recommended operation of the existing aeration basins in a step feed BNR configuration to increase TN removal through the Mason Farm WWTP Secondary Process to help comply with the upcoming Jordan Lake Nutrient Strategy mass load allocations. Increasing the removal of TN through the secondary system will reduce the dependence on the denitrification effluent filters to meet the limits. Additional TN removal through the secondary process is also expected to significantly reduce supplemental carbon, alkalinity and aeration requirements.

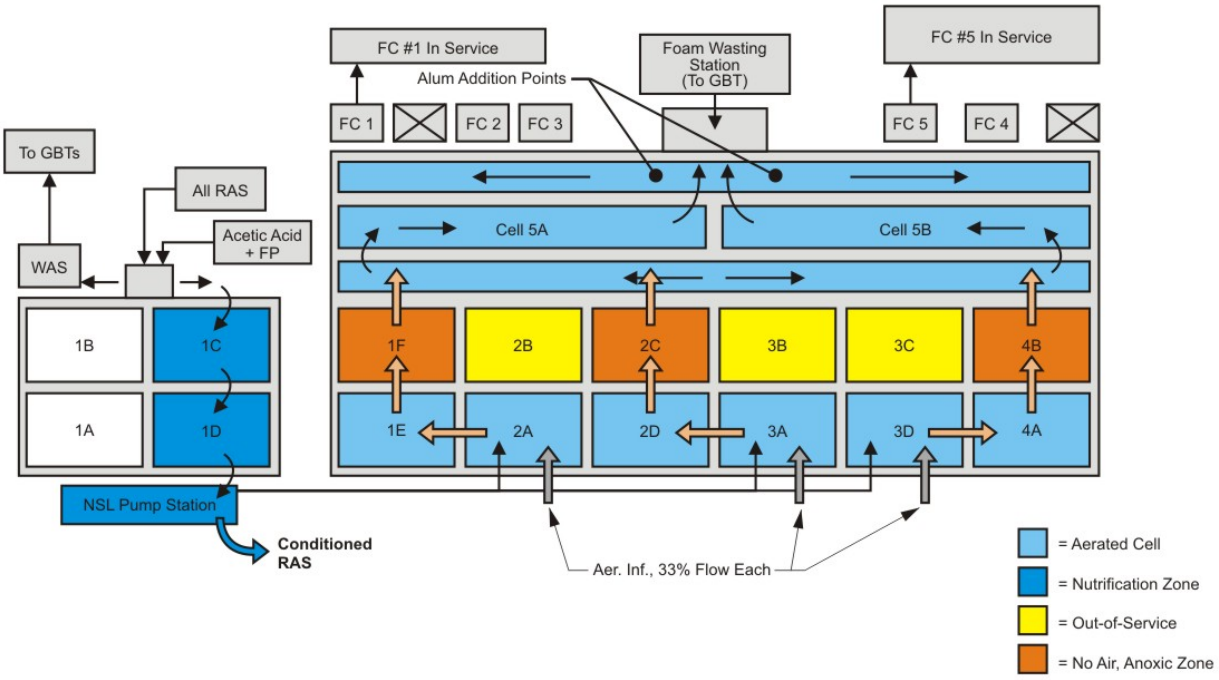
The existing aeration basins already have the flexibility to be configured to allow for step feed operation, so no capital improvements are necessary to operate in step feed mode. Although more stringent TN mass load limits will not need to be met until 2016, it is prudent that OWASA optimize TN reduction through the secondary process now while the TN limits are less stringent to determine

the impact of step feed operation on effluent TN concentrations and address any capital or operational issues that may be identified as a result of operating in the step feed BNR process mode. OWASA will also benefit from potential reductions in chemical usage for alkalinity adjustments and aeration costs by implementing the step feed configuration prior to 2016.

The Mason Farm WWTP is currently operated in the "4:3" configuration, with one train of three cells out of service. The third cell in each train is operated as an anoxic zone, although limited denitrification is occurring in this cell due to the lack of available carbon. Figure 1 provides a schematic of the existing configuration.

Figure 1

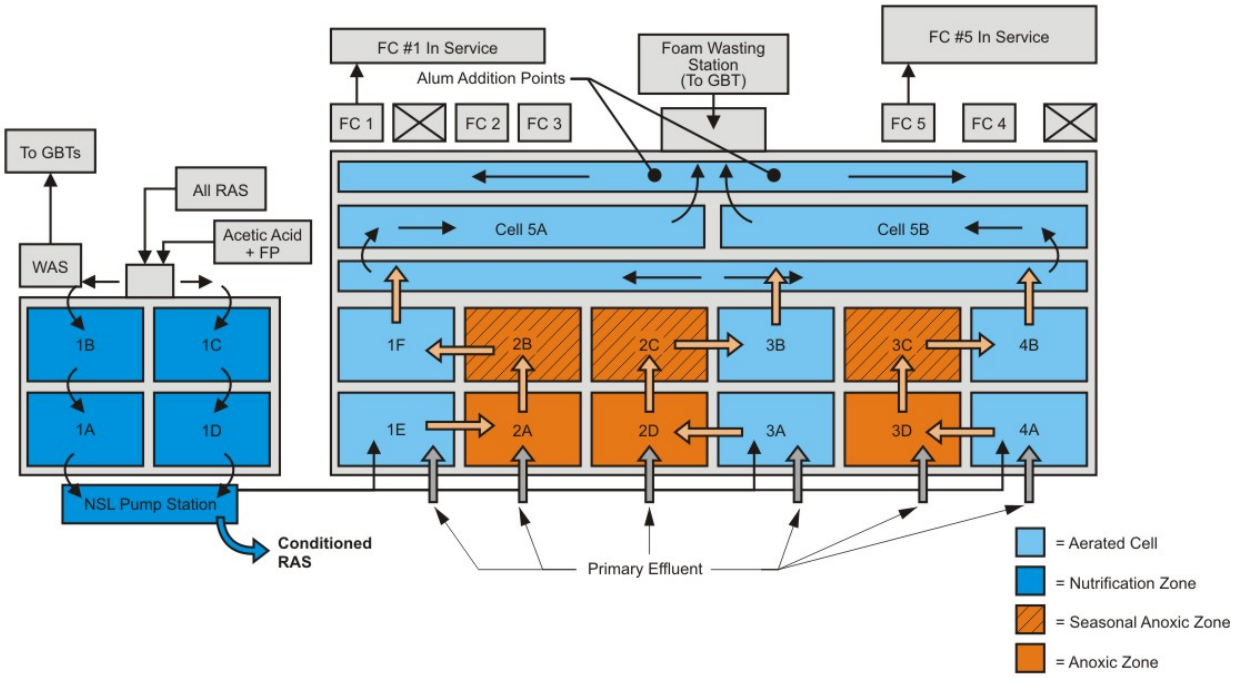
Current Process Configuration



The recommended step feed alternative includes operation of the aeration basin in the "3:4" configuration. The middle two cells of each train would be operated anoxically in this configuration, and primary effluent would be split 50/50 to the first and second cells in each train. NSL basin effluent would be discharged to the first cell in each train. The third cell would be operated aerobically during maximum month/minimum temperature design conditions. Figure 2 presents the proposed step feed configuration.

Figure 2

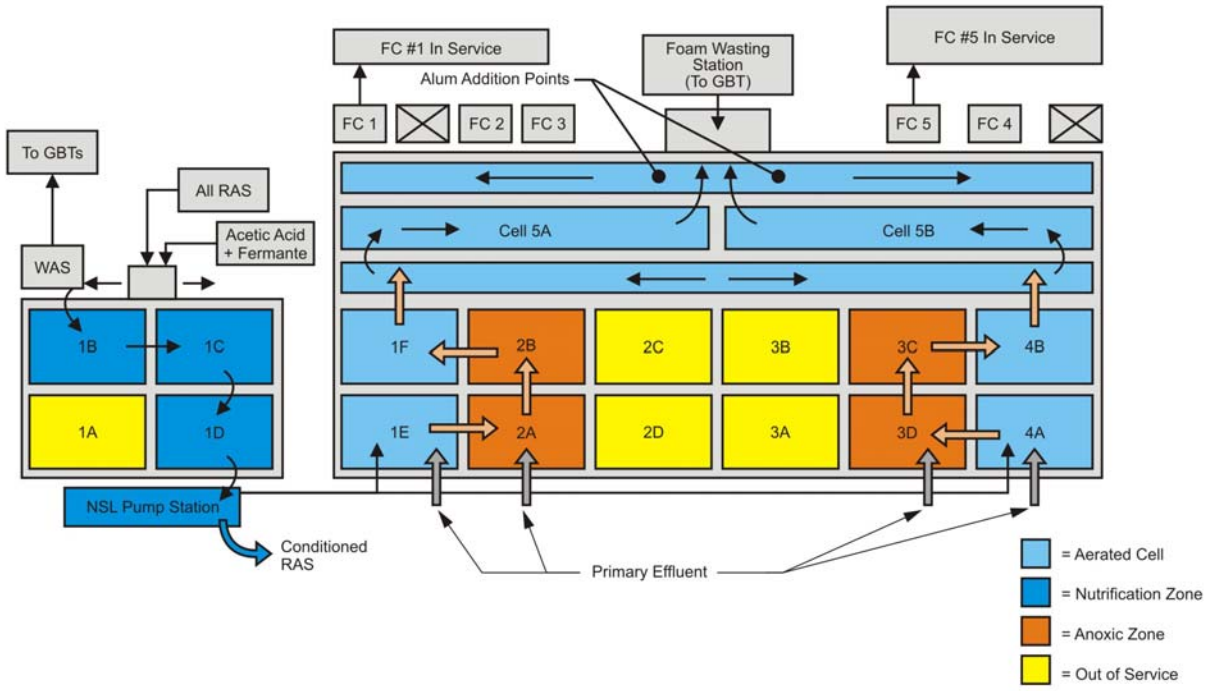
Proposed Step Feed Process Configuration



Since the current plant flow is approximately 8 mgd (approximately 2/3 of the 11.6 mgd annual average design flow, only two trains are required to be in service. It is recommended that the northernmost (Cells 1E, 2A, 2B and 1F) and southernmost (Cells 4A, 3D, 3C and 4B) trains initially be placed into service due to limited jet aeration capacity in Cell 3B of the middle train. Figure 3 provides a schematic of the proposed initial step feed configuration to treat current flows.

Figure 3

Proposed Initial Step Feed Configuration to Treat Current Flows



Process simulations were performed to estimate effluent TN concentrations and MLSS concentrations at current flows. The simulations predicted an effluent TN concentration of 12 mg/L in the step feed BNR configuration assuming a RAS flow rate of 4.8 mgd (60% of the 8 mgd influent flow). The effluent TN concentration is predicted to decrease to 10 mgd assuming a RAS flow rate of 8 mgd (100% of the 8 mgd influent flow). Operating in the step feed configuration with two trains in service and increasing the RAS rate to 100% of the influent flow has the potential to reduce current effluent TN concentrations to 10 mg/L or below under average conditions and 14 mg/L or below under maximum month / minimum temperature conditions. Note that operation under minimum temperature conditions assumes aerobic operation of the second anoxic zone (third cell) in eachtrain. A third step feed train could be placed into operation during cold weather conditions to maintain the second and third anoxic zones and provide greater TN removal; however the potential aeration limitation in Cell 3B may need to be addressed. Cold weather step feed operation will be further addressed once operating data from the initial step feed operation period is evaluated.

6. ALKALINITY OPTIMIZATION

Approximately 1,400 gpd of 25% sodium hydroxide is added at the Mason Farm WWTP to maintain pH, and the average secondary effluent alkalinity is approximately 75 mg/L as CaCO3. Providing additional denitrification in the secondary process will reduce the supplemental alkalinity requirements to maintain adequate pH for nitrification. An alkalinity balance was performed for the proposed step feed configuration, and the step feed operation is estimated to reduce sodium hydroxide (25%) requirements to 900 gpd to maintain a secondary effluent alkalinity of 75 mg/L as CaCO3. Alkalinity feed rates should be lowered as step feed trains are placed on line and RAS flow rates are increased.

Additional alkalinity reduction could be achieved by reducing the target secondary effluent alkalinity concentration to 50 mg/L as CaCO3. Many facilities operate with secondary effluent alkalinity concentrations of 50 mg/L or lower and pH between 6.5 and 7.0 with no noticeable reduction in nitrification capability. Operation at low pH can adversely impact BPR operation, so care must be taken to prevent pH from falling below the 6.5 range. The ability to reduce the target secondary effluent alkalinity to 50 mg/L as CaCO3 will reduce 25% sodium hydroxide feed requirements to an estimated 450 gpd at current flows.

7. PROCESS TRANSITION SCHEDULE

Table 5 presents a timeline of the process transition and optimization activities recommended herein. The process timeline is based on an optimization plan starting date of June 1, 2010. Note that the schedule presented is an initial estimate and should be adjusted as field conditions require. Table 6 summarizes reduction in the chemical feed rate through the process transition plan.

Table 5 – Process Transition Plan Implementation Schedule

Date	Days	Transition Description
6/1/2010	0	Reduce alum feed to filtrate to 600 mg/L (1 gallon 48% alum solution per 1,000 gallons filtrate). Monitor and adjust weekly as BPR improves.
6/1/2010	0	Adjust alum feed to MLSS to 55 mg/L (86 gallons 48% alum solution per mgd treated).
6/15/2010	14	Convert aeration basins to step feed mode (Cells 1E, 1F, 2A, 2B, 3C, 3D, 4A and 4). Take Cells 2C, 2D, 3A and 3B out of service.
6/29/2010	28	Increase RAS rate to 100% of influent flow.
7/6/2010	35	Reduce sodium hydroxide feed to 900 gpd or as necessary to maintain secondary effluent alkalinity of 75 mg/L as $CaCO_3$.
7/6/2010	35	Reduce acetic acid feed rates to 350 gpd. Adjust alum feed rate as necessary to meet effluent TP limits.
7/6/2010	35	Adjust alum feed to MLSS to 50 mg/L (78 gallons 48% alum solution per mgd treated). Continue reducing rate by 5 mg/L (approximately 8 gallons 48% alum solution per mgd treated) weekly until further reduction results in increased effluent TP concentrations
7/20/2009	49	Place 3rd NSL cell online. Eliminate acetic acid feed.
7/20/2009	49	Reduce sodium hydroxide feed to 450 gpd or as necessary to maintain secondary effluent alkalinity of 50 mg/L as $CaCO_3$ while maintaining stable nitrification and pH between 6.5 and 7.0.
Once Available	NA	Reduce MLSS alum feed by 20 mg/L (31 gallons 48% alum solution per mgd treated) once ability to feed alum to the secondary effluent is provided. Feed 20 mg/L of alum to secondary effluent. Adjust alum feed to MLSS as required to meet effluent TP limit.

Table 6 – Process Transition Plan Total Chemical Feed Rates

Elapsed Days	48% Alum Feed (gpd)	20% Acetic Acid Feed (gpd)	25% Sodium Hydroxide (gpd)
Current Typical Usage	950	600	1,400
0	720	600	1,400
35	650	350	900
49	650	0	450

8.0 REFERENCES

Grady, C. P. L.; Daigger, G. T.; Lim, H. C.; (1999) Biological Wastewater Treatment. 2nd ed. New York: Marcel Dekker, Inc.

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Latimer, R., Pitt, P. A., van Niekerk, A., Houlk, T., Deacon, S. (2007) Review of Primary Sludge Fermentation Performance in South Africa and the USA. Proceedings of the 80th Annual Water Environment Federation Technical Exhibition and Conference, San Diego, CA, October 13-17, 2007, pages 1651–1671.

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Ron Taylor, Joe Rohrbacher, Paul Pitt, Chris White

DATE: September 8, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Operational Transition and Optimization Assistance

Technical Memorandum – Process Transition Results

1. INTRODUCTION

Orange Water and Sewer Authority (OWASA) operates the Mason Farm Wastewater Treatment Plant (MFWWTP), an advanced treatment facility which discharges treated effluent to Morgan Creek, a tributary to Jordan Lake and the Cape Fear River. The most recent expansion to the Mason Farm WWTP increased the permitted maximum month capacity of the plant to 14.5 million gallons per day (mgd). The design also provided for a rated hydraulic peak capacity of 43.5 mgd with all facilities in service. As part of the expansion project, the plant was also upgraded to address anticipated nutrient limits associated with the Jordan Lake Nutrient Strategy. The upgrade included construction of deep bed denitrification filters and acetic acid feed facilities to provide Total Nitrogen (TN) removal. The Jordan Lake Nutrient Strategy was finalized in August 2009 and requires that the Mason Farm WWTP meet effluent annual mass limits for TN and Total Phosphorus (TP) equivalent to performance at concentrations of 3.0 mg/l and 0.23 mg/l, respectively, at the 14.5 mgd maximum month capacity. These nutrient limits are more stringent than those forecasted during the recent expansion project. Compliance with the TP mass limit is required in calendar year 2010, and the TN mass limit will need to be achieved starting in 2016.

OWASA engaged Hazen and Sawyer to perform a Hydraulic and Treatment Capacity Study for the current facilities in light of the proposed additional limitations on discharge of nutrients. A draft report was issued in March 2010 recommending improvements to address plant capacity limitations and nutrient reduction improvements to meet the final Jordan Lake Nutrient Strategy.

The report identified reconfiguring the existing activated sludge process to a step feed process as a low capital cost strategy to decrease secondary effluent TN concentrations while reducing operational costs. In addition, the report also identified that current biological phosphorus removal (BPR) performance was poor and OWASA is heavily relying on chemical phosphorus removal (CPR) to meet their current effluent TP limits.

In May 2010, Hazen and Sawyer submitted a Process Transition Plan Technical Memorandum describing the series of steps to reconfigure the existing aeration basins to the step feed BNR process in order to optimize WWTP operations for TP and TN removal while reducing costs. The primary goal of the process transition plan was to assess the potential additional TN removal provided by step feed operation while maintaining compliance with the facility's current permit limits. Based on a flowrate of 8 mgd, annual average TN and TP concentrations of 16.8 mg/L and 0.42 mg/L, respectively, need to be achieved to meet the current permit limits, which are summarized in Table 1. A secondary goal of the process transition plan was to reduce the amount of chemical addition at the MFWWTP to decrease operational costs.

The purpose of this memorandum is to discuss the implementation and results of the process transition for the period between June 1, 2010 to August 31, 2010. Supporting evaluations including nutrient profile analyses, bench-scale phosphate uptake and release tests and chemical phosphorus removal jar tests will also be discussed. Additional recommendations for plant optimization will be provided to assist OWASA in reducing operational costs while maintaining permit compliance.

Table 1

Mason Farm WWTP Current Permitted Effluent Limits

	Value	Period	Sampling Frequency
Flow (mgd)	14.5	Monthly	Continuous
$cBOD_5$ (mg/L) (April-October)	4.0	Monthly	Daily
$cBOD_5$ (mg/L) (November-March)	8.0	Monthly	Daily
TSS (mg/L)	30.0	Monthly	Daily
NH_3-N(mg/L) (April-October)	1.0	Monthly	Daily
NH_3-N(mg/L) (November-March)	2.0	Monthly	Daily
Total Kjeldahl Nitrogen (mg/L)	Report	-	Weekly
Nitrate/Nitrite (mg/L)	Report	-	Weekly
Total Phosphorus (mg/L)	Report	-	Weekly
Total Phosphorus (lbs/year)	10,188[1]	Annual	Monthly
Total Nitrogen (mg/L)	Report	-	Weekly
Total Nitrogen (lbs/year)	409,448	Annual	Monthly
Dissolved Oxygen (mg/L)	> 6.0	Monthly	Daily
pH	6 – 9	Monthly	Daily
Fecal Coliform (cfu/100 mL)	200	Monthly	Daily
Total Residual Chlorine (max. µg/L)	18	Daily	Daily

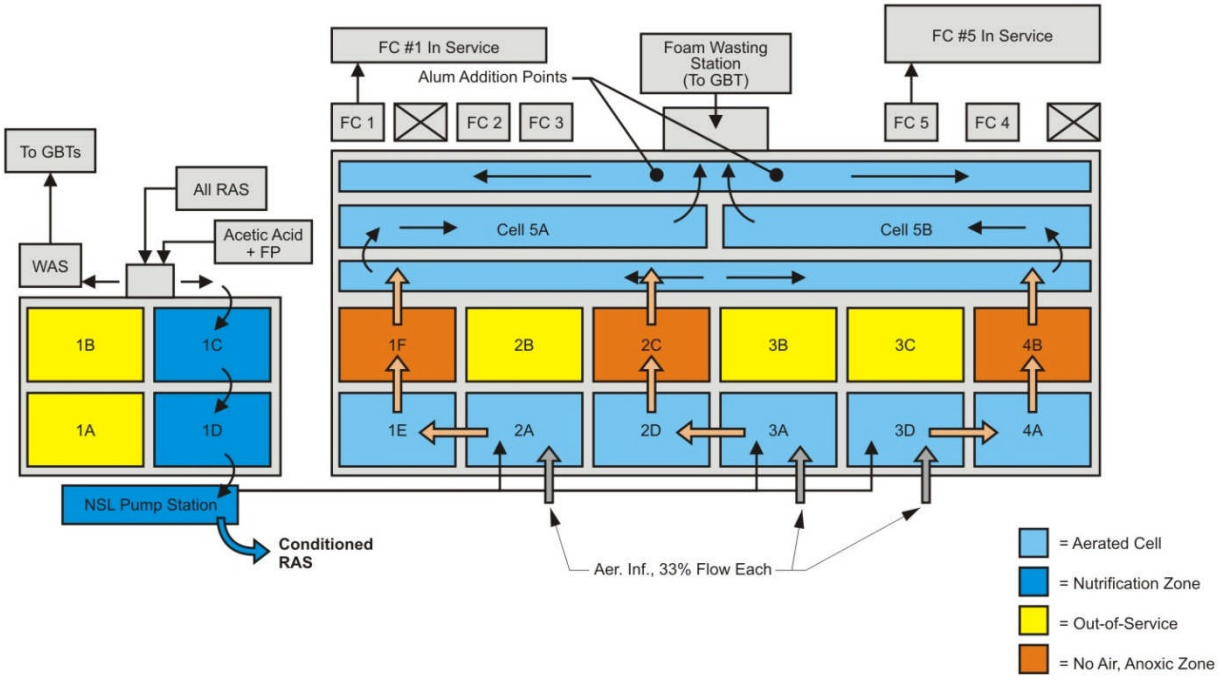
[1]Effective Calendar Year 2010

2. Initial Conditions and Performance

The aeration basins at the MFWWTP have the flexibility to operate in various flow configurations including varying the number of cells in service per train and independent control of aeration for each cell. Prior to the implementation of the transition plan, the MFWWTP operated in the "4:3" configuration, with one train of three cells out of service. Each train of three cells included two aerated cells followed by a third anoxic cell. Basin effluent was collected in aerated effluent channels prior to the alum dose point and distribution to the secondary clarifiers. Figure 1 provides a schematic of this configuration.

Figure 1

4:3 Process Configuration



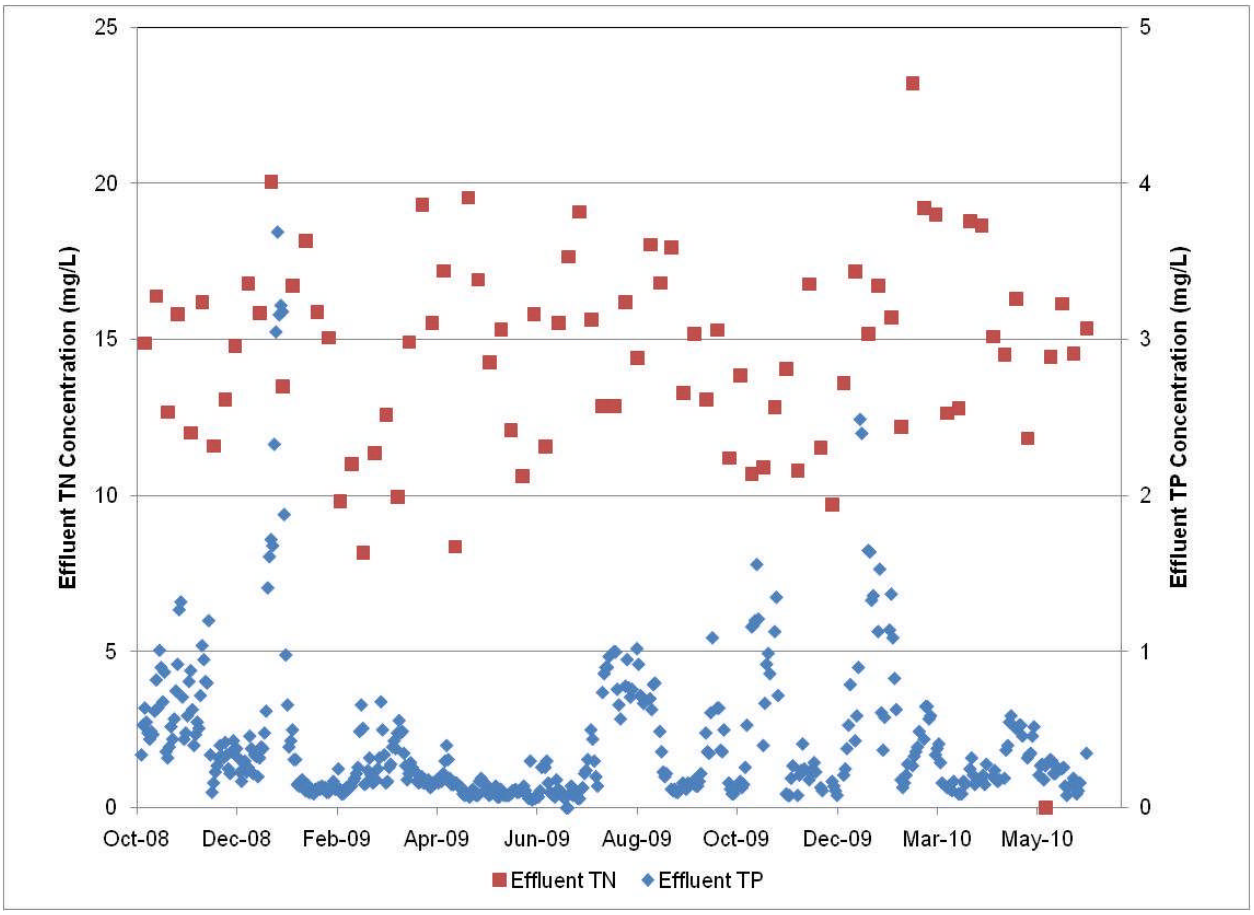
Nutrient removal performance from November 2008 through May 2010 (prior to the process transition) is shown in Figure 2. Effluent TN and TP concentrations averaged approximately 15 mg/L and 0.41 mg/L, respectively, during this period. The average plant effluent flow was approximately 7.1 mgd. Current permit limits were met during this period, but significant alum feed to the dewatering filtrate and mixed liquor suspended solids (MLSS) upstream of the secondary clarifiers was required to meet the effluent TP mass limit.

Daily alum (48%), sodium hydroxide (25%) and acetic acid (20%) feed rates from November 2008 to August 2009 were evaluated. The average alum feed rate during this period was 925 gallons per day (gpd). The plant operating records do not indicate the distribution of alum to the MLSS and the filtrate, but the plant operations staff indicates that approximately 2/3 of the alum feed may have been to the filtrate during this period. Chemical feed data was unavailable from September 2009 through March 2010, but operational data from April and May 2010 indicate that the alum feed rate was approximately 870 gpd prior to the process transition. During this period reported alum feed rates to the filtrate varied widely as a result of rotary press operation, but feed rates to the MLSS were approximately 660 gpd.

Acetic acid (20%) is fed to the nutrified sludge (NSL) basins to promote RAS denitrification and biological phosphorus removal. The average acetic acid feed rate prior to the process transition was 620 gpd. Sodium hydroxide (25%) is fed to the aeration basin influent channel to prevent pH depression during nitrification. The typical feed rate prior to the process transition was 1,360 gpd.

Figure 2

Historical Nutrient Removal Performance



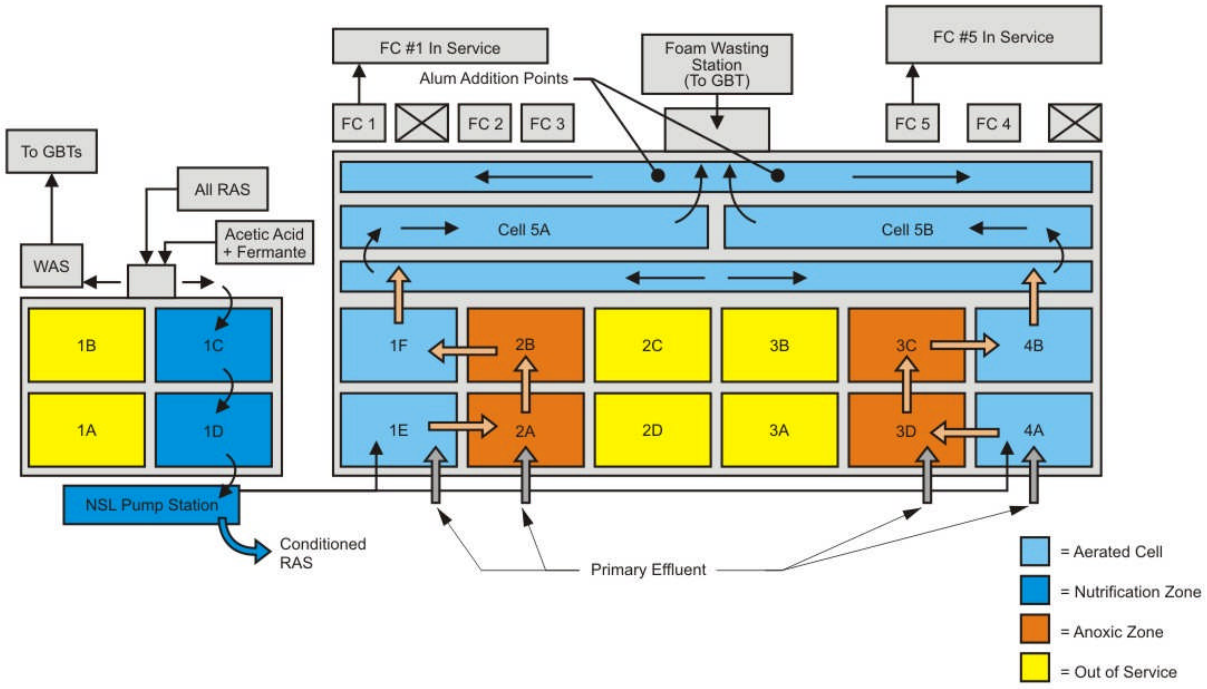
3. Step Feed Implementation

The step feed alternative proposed in the PER and process transition plan included operation of the aeration basin in the "3:4" configuration. The middle two cells of each train are operated anoxically in this configuration, and primary effluent is evenly split to the first and second cells in each train. NSL basin effluent is discharged to the first cell in each train. The third cell would be operated aerobically during maximum month loading/minimum temperature design

conditions to maintain full nitrification. Since the current plant flow is approximately 8 mgd (2/3 of the 11.6 mgd annual average design flow, only two trains are currently required to be in service to match the future design conditions. The northernmost (Cells 1E, 2A, 2B and 1F) and southernmost (Cells 4A, 3D, 3C and 4B) trains were initially placed into service due to limited jet aeration capacity in Cells 2C and 3B of the middle train. Figure 3 presents the step feed configuration implemented during the transition.

Figure 3

Initial Step Feed Configuration



4. Transition Process Monitoring

Samples throughout the liquid process train were collected, field filtered, and analyzed by Hazen and Sawyer on April 28, July 12, and August 10, 2010 to compare process performance prior to and after the transition to step feed operation. Table 2 presents a list and description of the sample locations, and Figure 4 identifies these points on a site plan. In addition, existing online instrumentation, including the ChemScan nutrient analyzer mounted at the filters, was also utilized for process monitoring and optimization.

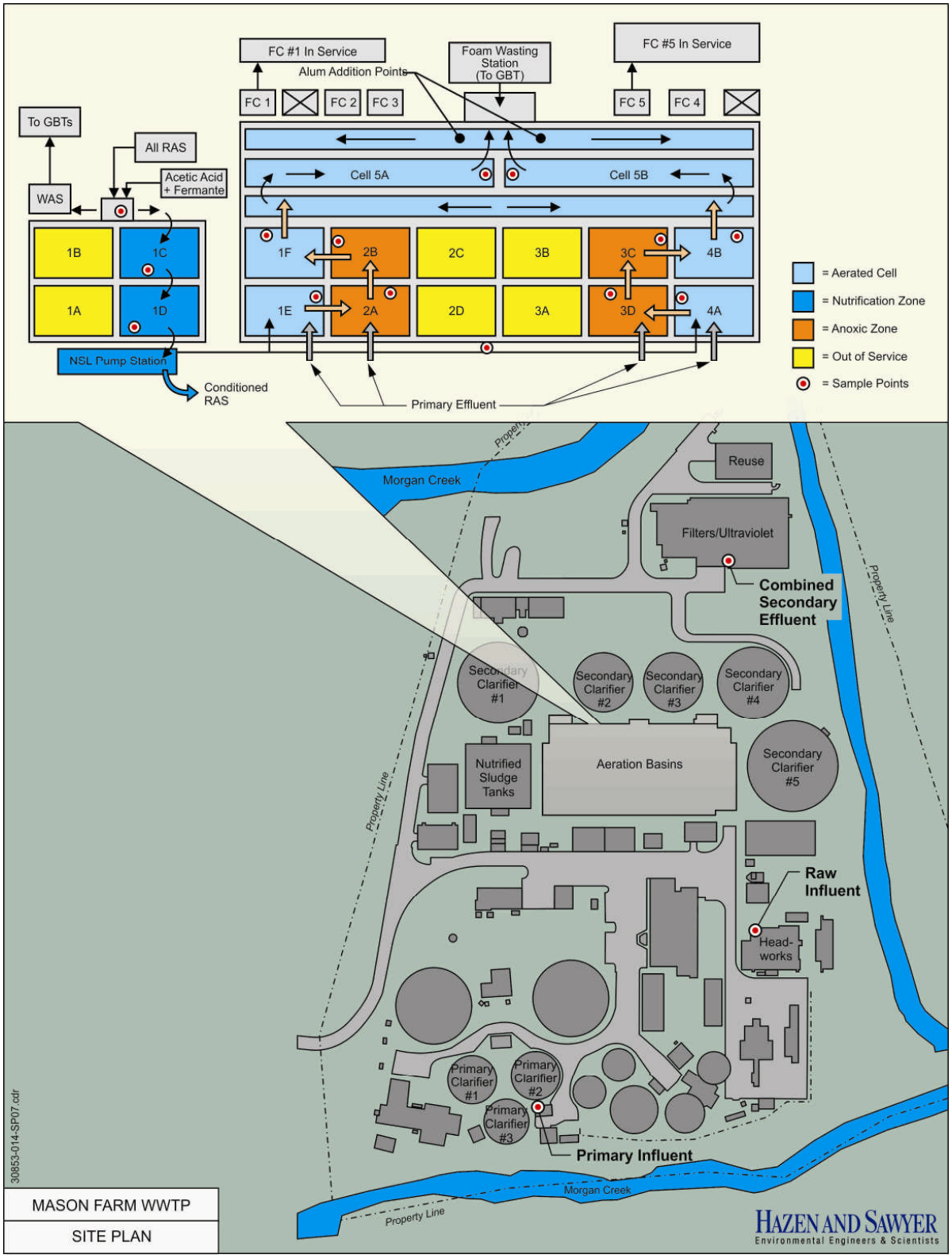
Table 2

Sample Points

Raw Influent	
Primary Influent	
Aeration Basin Influent Channel	
End Cell 1E (Aerated)	End Cell 4A (Aerated)
End Cell 2A (Anoxic)	End Cell 3D (Anoxic)
End Cell 2B (Anoxic)	End Cell 3C (Anoxic)
End Cell 1F (Aerated)	End Cell 4B (Aerated)
End Cell 5A (Aerated)	End Cell 5B (Aerated)
Combined Secondary Effluent	
Combined RAS/Nutrified Influent	
Nutrified Sludge Basin End Cell 1C	
Nutrified Sludge Basin End Cell 1D	

Figure 4

Sampling Site Plan



Figures 5 provides ammonia (NH_3-N) and nitrate (NO_3-N) profiles through the process prior to the implementation of the transition plan. Figures 6 and 7 present these nitrogen profiles after the transition to the step feed configuration. The nutrient profiles indicate substantially enhanced TN removal through the secondary process under the step feed configuration. Secondary effluent nitrate concentrations decreased by two-thirds (approximately 10 mg/L) upon the transition to step feed.

Figure 5

April 28, 2010 Nitrogen Profile (Before Transition)

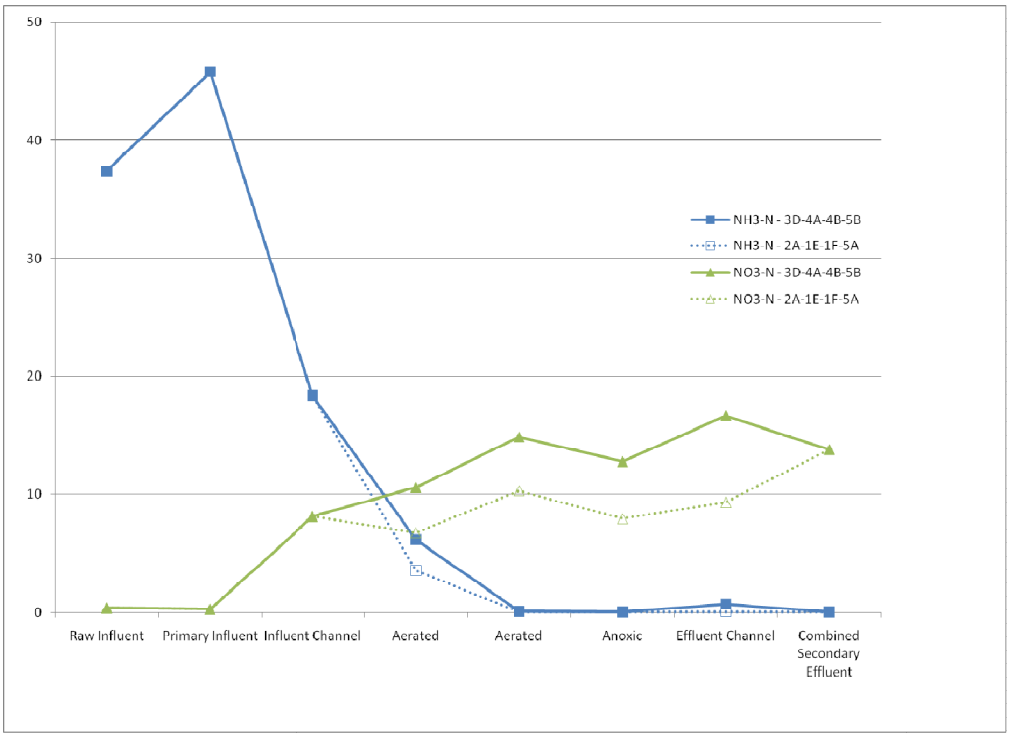


Figure 6

July 12, 2010 Nitrogen Profile (After Transition)

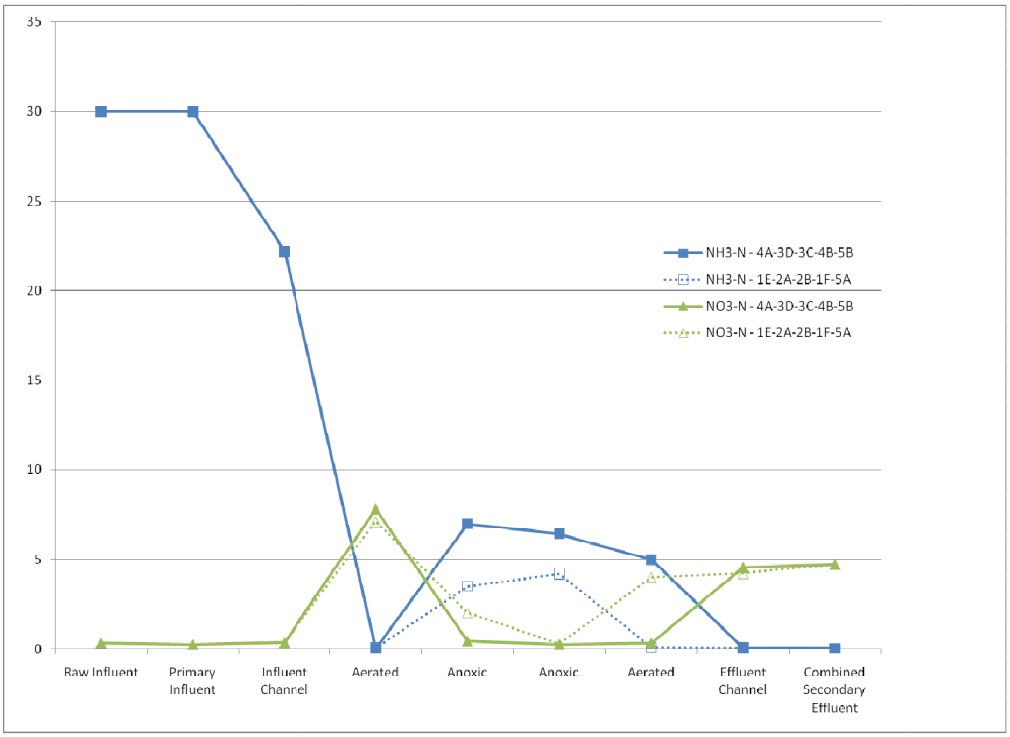
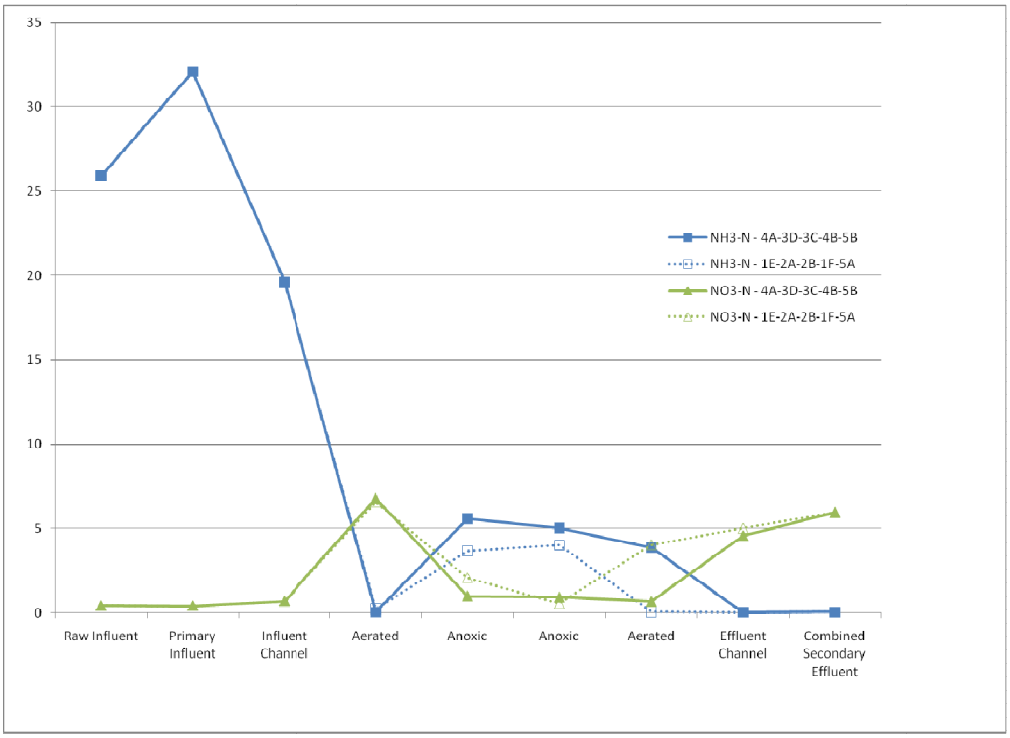


Figure 7

August 10, 2010 Nitrogen Profile (After Transition)



While both step-feed trains provided similar overall nitrogen removal, significantly less nitrification was observed in Cell 4B compared to Cell 1F. Figures 8 and 9 present the measured dissolved oxygen (DO) concentrations through the process on July 12 and August 10, 2010, respectively. The DO profiles indicate that Cell 4B was operated at a very low DO concentration (less than 0.5 mg/L). The reduced aeration capacity in Cell 4B is suspected to be due to clogging of the jet aeration equipment, particularly since Cell 4B had previously been operated as an anoxic cell and solids have likely settled in the aeration headers. Aeration system maintenance for Cell 4B should be performed prior to cold weather operation in order to ensure sufficient DO in Cell 4B and increase nitrification reliability.

Figure 8

July 12, 2010 DO Profile

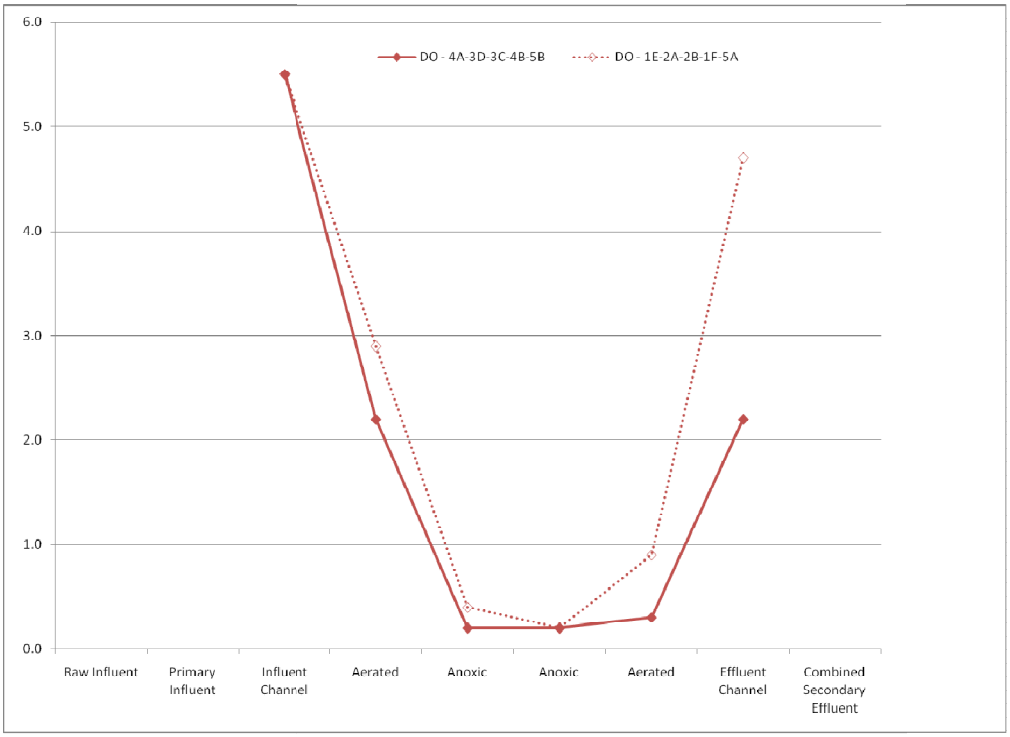
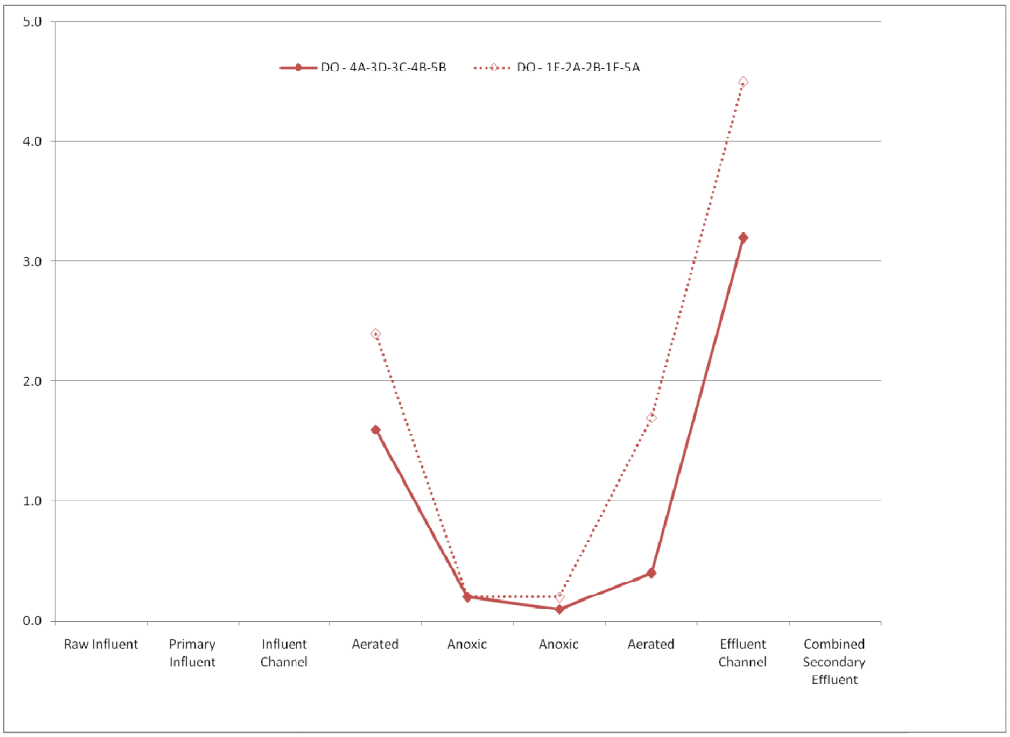


Figure 9

August 10, 2010 DO Profile



Figures 10, 11 and 12 illustrate the ortho-phosphate (PO_4-P) concentration profiles before and after the transition to step feed. As shown in Figure 10, limited ortho-phosphate release was observed through the NSL basins. Likewise no aerobic uptake was observed through the aeration basins. These observations indicate little to no biological phosphorus removal (BPR) was occurring prior to the process transition. Ortho-phosphate release was observed in the NSL basins and anoxic zones after the transition to step feed, indicating the presence of poly-phosphate accumualting organisms (PAOs), which are responsible for BPR. Aerobic uptake was also observed after the transition. The observed ortho-phosphate release in the NSL cells was relatively low (particularly on August 10), indicating that BPR may still be unstable.

The reduction of nitrate in the return activated sludge (RAS), which is returned to the NSL cells, may have increased BPR performance once step feed was implemented. Denitrification of RAS in the NSL cells competes for carbon with the BPR process, and a reduction in nitrate to the NSL cell results in more VFAs being available to the PAOs.

Figure 10

April 28, 2010 PO_4-P Profile (Before Transition)

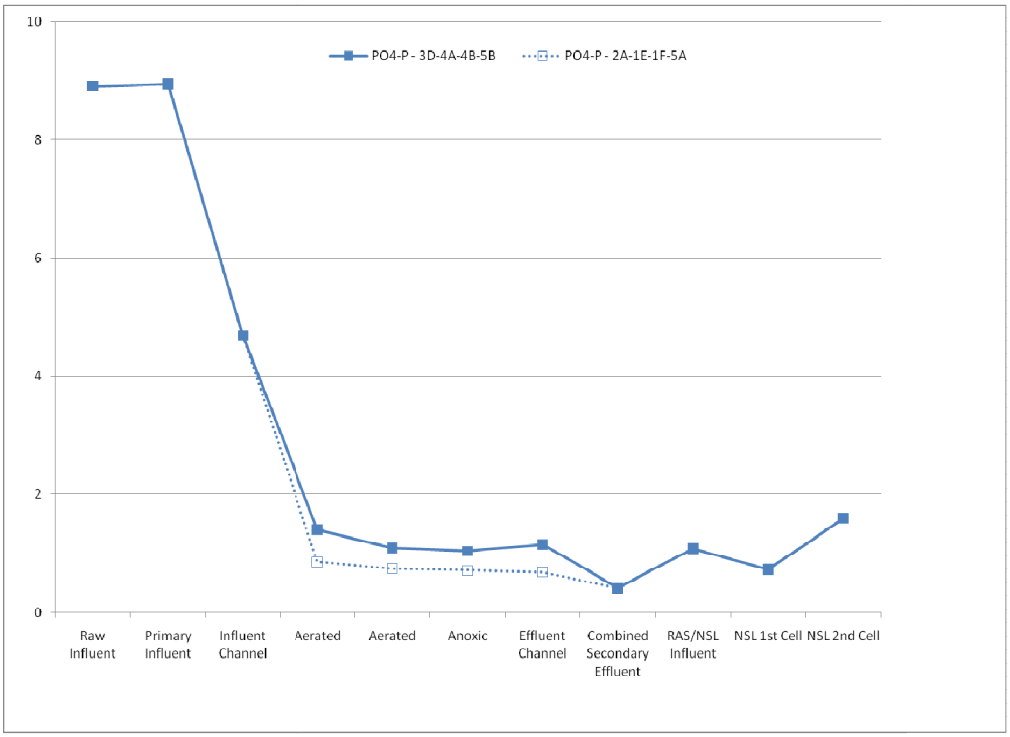


Figure 11

July 12, 2010 PO_4-P Profile (After Transition)

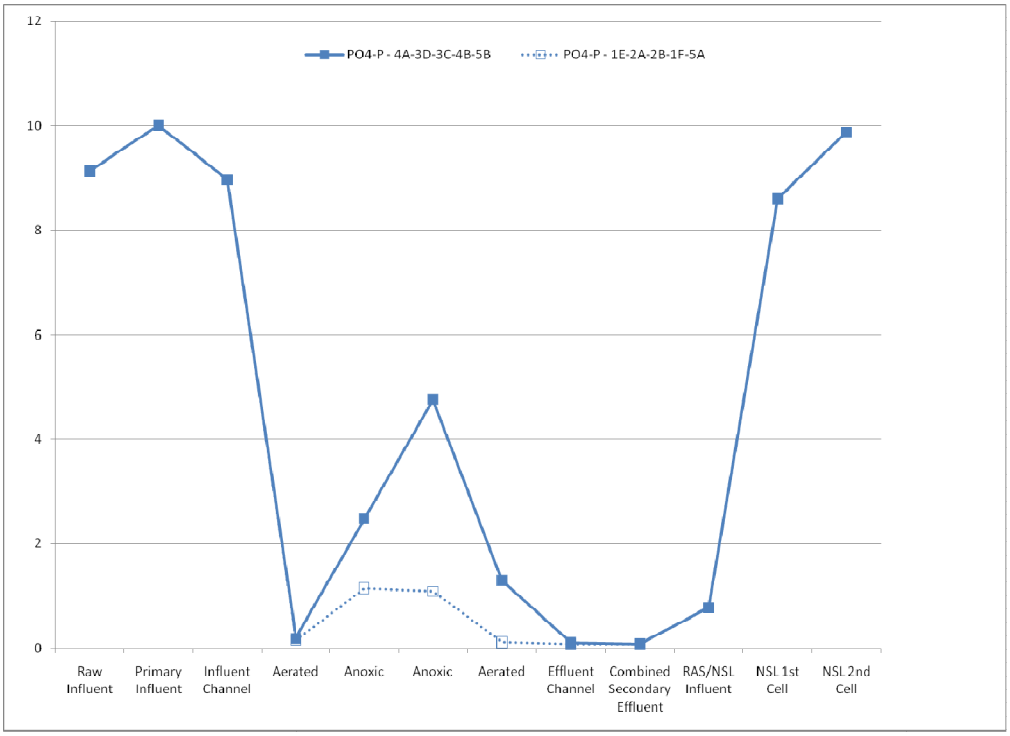
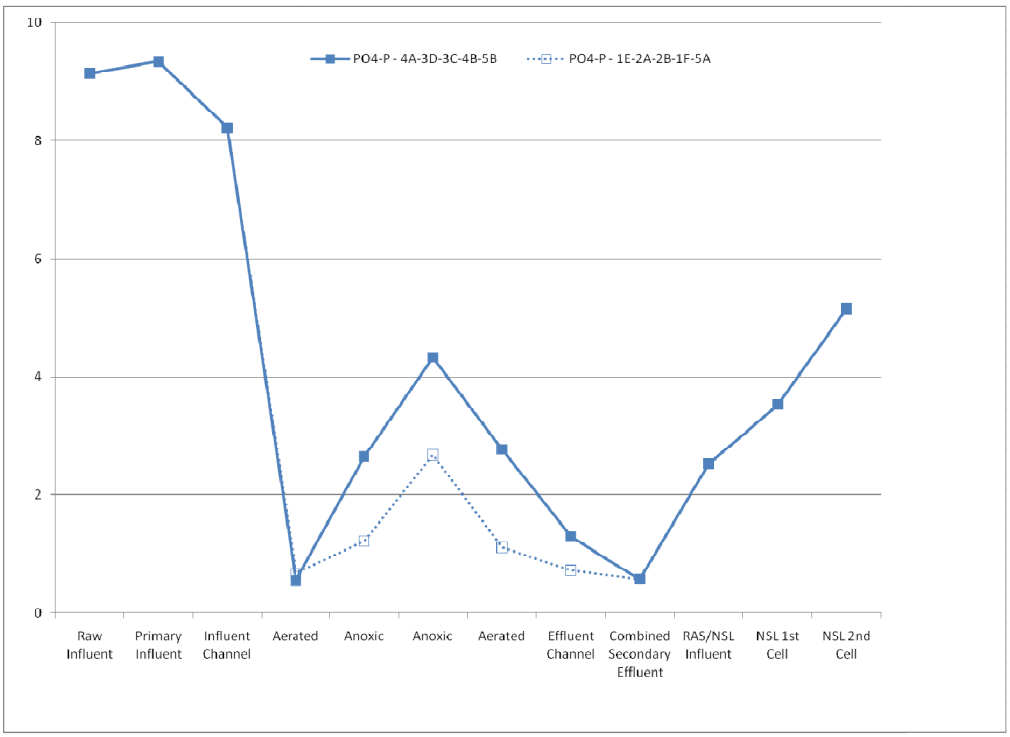


Figure 12

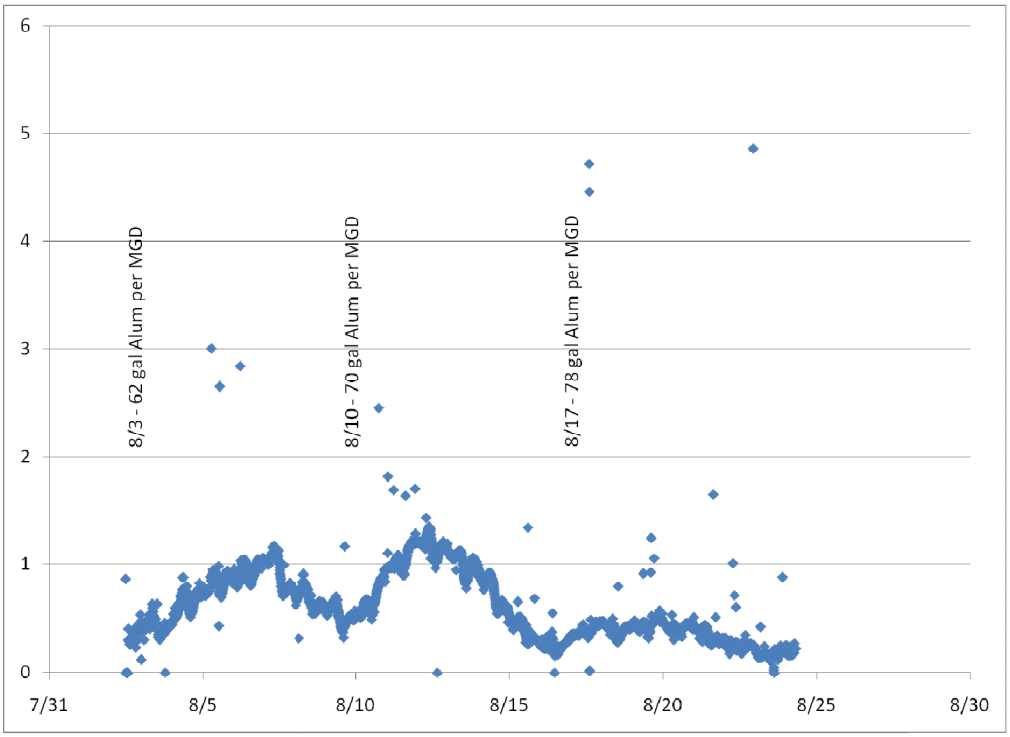
August 10, 2010 PO_4-P Profile (After Transition)



A Hach Phosphax ortho-phosphate analyzer was temporarily installed on August 2, 2010 in Cell 5A to closely monitor ortho-phosphate at the aeration basin effluent prior to alum feed to assist in optimizing alum feed. The Phosphax analyzer collected a sample every 10 minutes and locally displayed data trends for 1 day and 1 week periods. Hazen and Sawyer downloaded the data on a weekly basis. The ortho-phosphate trend during the month of August is displayed in Figure 13. The alum feed rate to the MLSS prior to the transition was approximately 100 gallons per day per mgd (gpd/mgd) treated. An alum feed rate of 78 gpd/mgd treated has resulted in consistent effluent ortho-phosphate concentrations. The feed rate was lowered to between 60-70 gpd/mgd during a portion of the transition but elevated effluent ortho-phosphate concentrations were observed.

Figure 13

Online PO_4-P Analyzer, Cell 5A



5. PHOSPHORUS RELEASE/UPTAKE TESTING

The Mason Farm WWTP utilizes the OWASA process for biological phosphorus removal. This process relies on the addition of fermentate and acetic acid to the RAS in the NSL basins to provide carbon for biological phosphorus release. The released soluble phosphorus is assimilated by poly-phosphate accumulating organisms under aerobic conditions in the aeration basins (luxury phosphorus uptake). The results from the biological phosphorus uptake and release (PUR) testing performed on April 27, 2010 indicated relatively poor BPR was occurring during the testing and supports the nutrient profile data prior to the transition. Table 3 compares the results from this testing with results from other Hazen and Sawyer PUR tests and the range of values cited in the WERF publication Factors Influencing the Reliability of Enhanced Biological Phosphorus Removal (2005).

Table 3

Phosphate Release and Uptake Testing Results

	Acetate	Fermentate	Fermentate + Acetate	H&S Observed Values	WERF Observed Values
P Release Rate (mg P/g VSS/hr)	0.61	1.53	1.79	6.9 - 9.0	6.4 - 24.6
P Uptake Rate (mg P/g VSS/hr)	0.60	0.91	1.13	2.2 – 3.6	2.4 – 5.9
Release to Uptake Ratio	1.02	1.68	1.58	2.5 – 3.4	2.3 – 4.0
P Release/COD Uptake (mg P/ mg COD)	0.18	-[1]	-[1]	0.21 – 0.30	0.22 – 0.51
Average MLVSS (mg/L)	5,600	7,450	6,720	1,000 -1,500	1,850 – 2,600

[1]Note these values could not be properly estimated due to biodegradation of fermentate COD

The combined acetate/fermentate volatile fatty acids are largely comprised of acetic acid (73% as COD), based on VFA analysis. Current research suggests that similar amounts of acetic acid and propionic acid are optimal for BPR performance, and having too great an amount of acetic acid or propionic acid can reduce BPR efficiency due to glycogen accumulating organism (GAO) proliferation. This phenomenon may have contributed, in part, to the poor BPR efficiency observed at the MFWWTP. Reduced available soluble phosphorus (due to chemical phosphorus removal) may also limit the phosphate available to the PAOs, further selecting for GAOs.

Additional phosphorus release/uptake testing was performed throughout the transition plan to determine the prevalence of PAOs and overall BPR efficiency. MLSS samples were collected and sent to Dr. Andrew Shuler at the University of New Mexico. Samples were collected at the initiation of the transition on June 2 and subsequently on July 13 and August 4, 2010. An increase in BPR efficiency has been observed since the transition to step feed as indicated by observed increases in the phosphorus content of the biomass, phosphorus release to acetate uptake ratios and anaerobic phosphorus release (normalized to VSS concentration). A memorandum from Dr. Shuler and supporting data are provided in Appendix A.

6. CHEMICAL PHOSPHORUS REMOVAL JAR TESTING

OWASA currently adds alum to the MLSS upstream of the secondary clarifiers to reduce ortho-phosphate concentrations and meet the current effluent TP limit. Chemical phosphorus removal jar testing was performed on April 28, July 12 and August 10, 2010 to determine the efficiency of alum and sodium aluminate addition for phosphorus removal. Chemical jar tests were performed on filtrate (April 28 only), MLSS and secondary clarifier effluent (SCE). The jar testing protocol is provided in Appendix B. Results of the jar testing are summarized in Appendix C.

The stoichiometric aluminum requirement for phosphorus precipitation is 1 mol Al per 1 mol of phosphorus (Al:P molar ratio = 1). This equates to a dose of 9.6 mg/L alum or 2.6 mg/L sodium aluminate per 1 mg/L phosphorus removed. Phosphorus precipitation with aluminum species is most effective in a pH range of 6.0 to 6.5. Operation at a pH above this range will reduce chemical phosphorus removal efficiency since aluminum will react with excess hydroxide to form aluminum hydroxides. Observations at other wastewater facilities indicate a 1:1 molar ratio is fairly typical when reducing ortho-phosphate down to about 1 mg/L with alum. Further reduction in ortho-phosphate concentration results in substantially greater chemical demands due to competing reactions, and molar ratios between 3:1 and 5:1 are often observed to reduce ortho-phosphate to 0.3 mg/L or below. Molar ratios greater than 10:1 are routinely observed to reduce ortho-phosphate from an initial concentration of 0.1 mg/L.

Jar testing with 48% alum solution was performed on April 28th, 2010, prior to the start of the transition plan. Alum was added to the MLSS, SCE and filtrate. The jar testing indicated relatively poor removal of phosphorus in the MLSS and secondary effluent with initial ortho-phosphorus concentrations of 0.77 mg/L for MLSS and 0.41 mg/L for SCE. Al:P molar ratios ranged from 4.6 to 7.3 for the SCE and 6.7 to 9.7 for the MLSS. More efficient utilization of the alum was observed in the filtrate sample (initial ortho-phosphorus concentration of 38 mg/L), where the Al:P molar ratio ranged from 1.3 to 1.9; however, the pH of the filtrate (8.2-8.4) was substantially greater than the optimal range for phosphate precipitation with alum, and Al:P molar ratios of closer to 1:1 would be expected if the pH was 6 to 7. This observation also likely precludes the effective use of sodium aluminate for filtrate treatment.

Chemical jar tests with 48% alum and 38% sodium aluminate solutions were performed on MLSS and SCE during the process transition period on July 12 and August 10, 2010. Since

dewatering was not regularly practiced during this period, filtrate testing was omitted. The calculated Al:P molar ratio for the MLSS and SCE varied widely during the July 12 testing. Initial ortho-phosphorus concentrations were very low (<0.1 mg/L for MLSS and <0.2 mg/L for SCE) and were at the lower detection limits of the test kits; therefore the data collected during the July 12 testing was determined to be unreliable. During this phase of the process transition, reductions of alum fed to the MLSS had not occurred and the MFWWTP was operating with a 48% alum dose of approximately 100 gallons per mgd treated. This high alum dose likely impacted the jar test results.

Chemical jar tests were repeated on August 10, 2010, and the results are illustrated in Figures 14 and 15. Initial ortho-phosphorus concentrations were 0.5 and 0.7 mg/L for MLSS tests and 0.5 and 0.6 mg/L for SCE tests. Molar Al:P ratios for alum were calculated between 4 and 10 for addition to SCE and 7 to 12 for addition to MLSS. Molar Al:P ratios for sodium aluminate were calculated between 9 and 19 for addition to SCE and 12 to 24 for addition to MLSS. The addition of acid or base was implemented to hold pH to approximately 7 for the MLSS tests. The pH of SCE jar tests was more difficult to maintain constant with pH values varying from 7.2 to 7.7 for 48% alum tests and 7.5 to 8.1 for 38% sodium aluminate tests.

Figure 14
Secondary Clarifier Effluent Dose Response

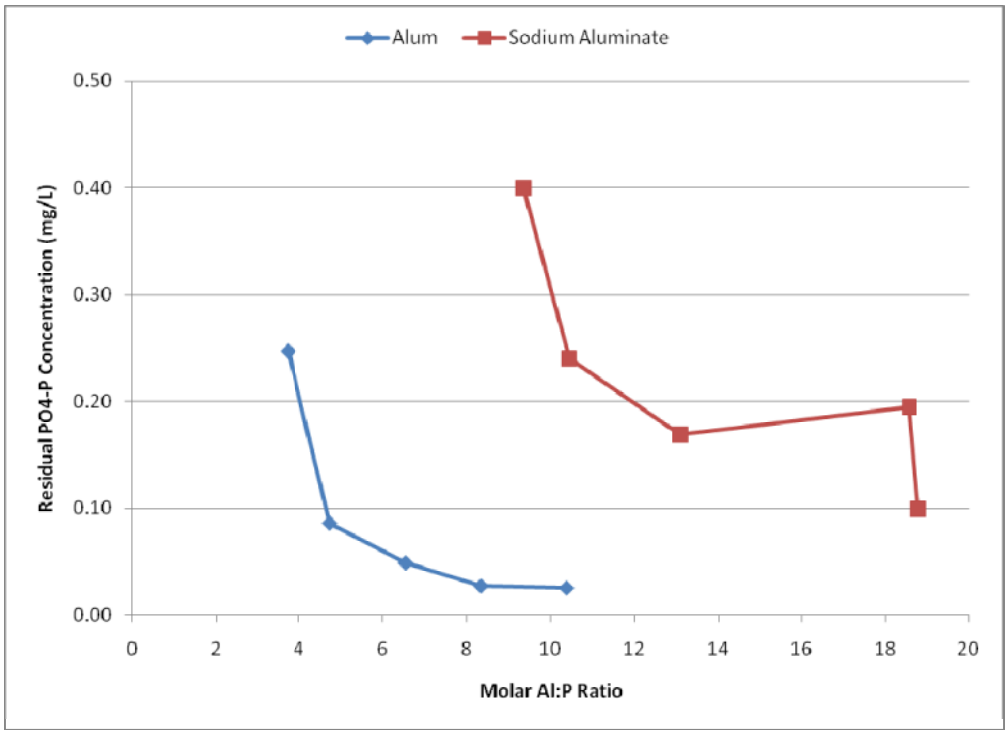
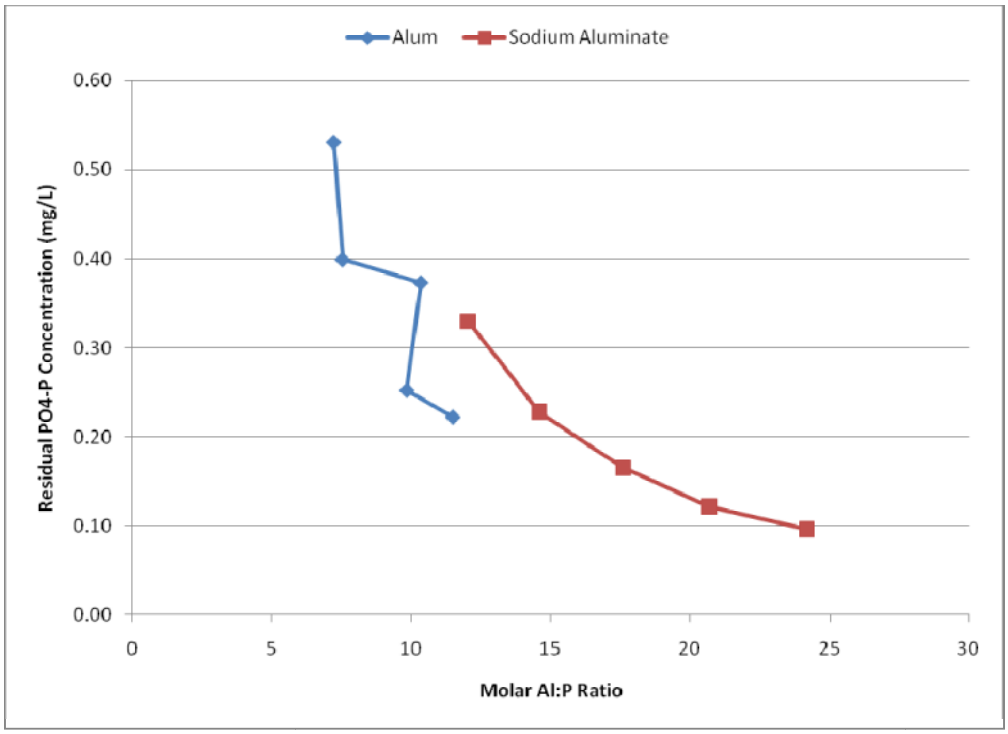


Figure 15

Mixed Liquor Dose Response



The observed Al:P ratio for alum addition to the secondary effluent is fairly typical given the low initial and final (<0.03 mg-P/L at higher doses) ortho-phosphate concentrations. Chemical feed requirements to MLSS are often greater than that required to secondary effluent for an equivalent amount of phosphorus removal due to greater potential for competing reactions in the MLSS. As expected the jar testing indicated better utilization of chemical when applied to the secondary effluent instead of the MLSS. It is noted that excessive coagulant addition to secondary clarifier effluent could cause filter blinding, and a careful balance must be struck between the multi-point feed of coagulant to the MLSS, where utilization is less efficient, and a tertiary process such as a filter.

The required aluminum dose for addition of sodium aluminate was approximately twice that required for alum on a lb Al per lb P removed basis. The increase in alkalinity and pH associated with sodium aluminate addition was compensated by pH adjustment with 1.0 N HCl during the jar testing. The pH of the reactors where sodium aluminate was added was approximately 0.3 units greater than the alum test reactors after pH adjustment. This slight increase in reactor pH could be a contributing factor for the increased sodium aluminate

requirement.

Full-scale addition of sodium aluminate to the MLSS and SCE may provide greater phosphorus removal efficiency than indicated during the jar testing. Recent jar tests performed at other facilities have also suggested that the use of sodium aluminate for phosphorus removal is less efficient than alum due to increasing pH at higher doses, Several WWTPs in North Carolina have recently switched from alum and ferric chloride to sodium aluminate addition. The facilities that are currently adding sodium aluminate have reported greater phosphorus removal efficiency compared to alum addition, which may be due to greater BPR efficiency attributed to increased pH. Sodium aluminate addition to the filtrate will likely be less efficient than alum due to the high filtrate pH.

7. PROCESS TRANSITION OBSERVATIONS

Table 4 presents the timeline presented in the process transition plan memorandum and initiated on June 1, 2010. Figure 16 summarizes reduction in the chemical feed rates throughout the implementation of the process transition plan.

Table 4

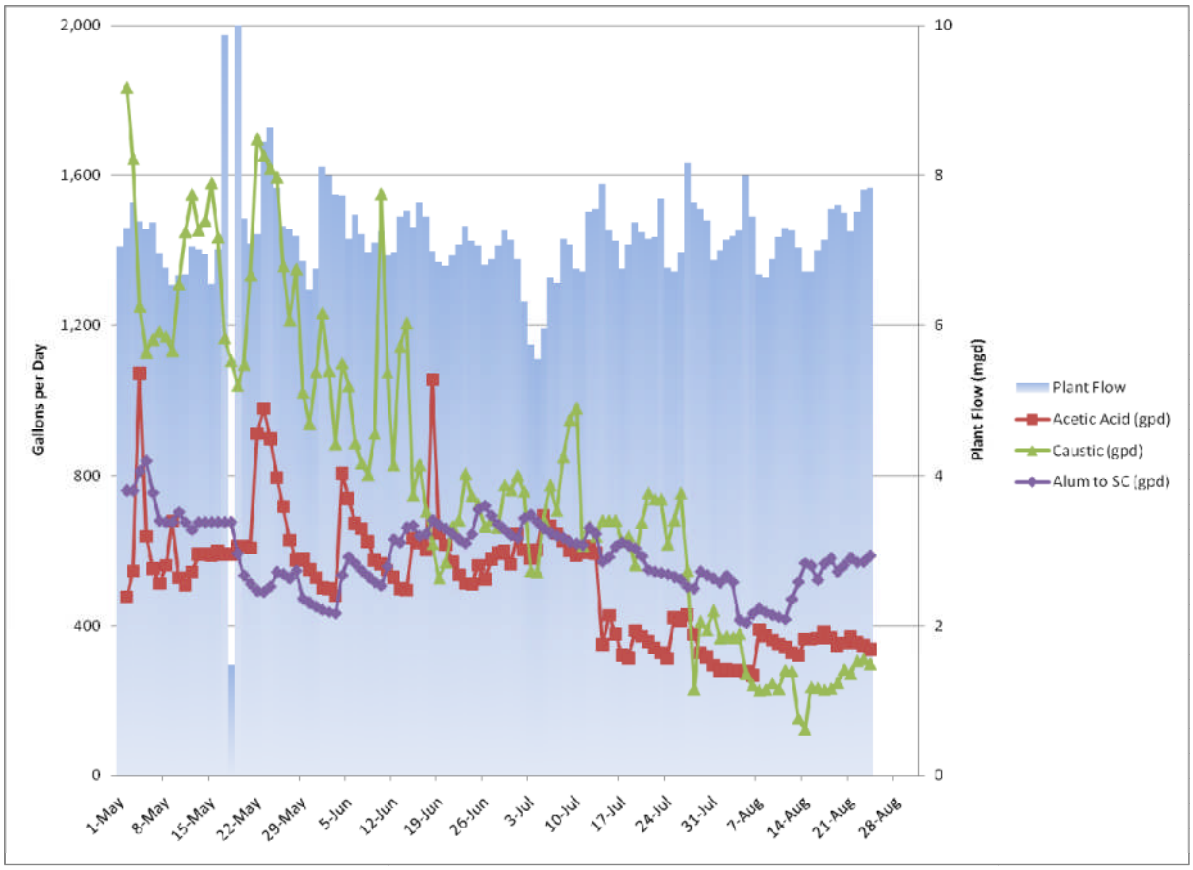
Process Transition Plan Implementation Schedule

Date	Days	Transition Description	Observations
6/1/2010	0	Convert aeration basins to step feed mode (Cells 1E, 1F, 2A, 2B, 3C, 3D, 4A and 4). Take Cells 2C, 2D, 3A and 3B out of service. Reduce sodium hydroxide feed to 1,100 gpd or as necessary to maintain secondary effluent alkalinity of 75 mg/L as $CaCO_3$.	Sodium hydroxide feed was reduced to ~900 gpd.
6/15/2010	14	Increase RAS rate to 100% of influent flow. Reduce sodium hydroxide feed to 900 gpd or as necessary to maintain secondary effluent alkalinity of 75 mg/L as $CaCO_3$.	RAS rate was increased to approximately 100% of influent flow. Sodium hydroxide feed was reduced to ~700 gpd.
6/29/2010	28	Hazen and Sawyer to perform additional nutrient profiles, phosphorus uptake/release testing and chemical phosphorus removal jar testing	Hazen and Sawyer performed testing on July 12 due to limited alum pump turndown to the rotary press.
7/13/2010	42	Adjust alum feed to MLSS to 55 mg/L (86 gallons 48% alum solution per mgd treated). Continue reducing rate by 5 mg/L (approximately 8 gallons 48% alum solution per mgd treated) weekly until further reduction results in increased effluent TP concentrations	Alum feed rates: 7/6 – ~100 gal / mgd treated 7/13 – 86 gal / mgd treated 7/20 – 78 gal / mgd treated

7/13/2010	42	Reduce acetic acid feed rates to 350 gpd or as determined through additional testing. Adjust alum feed rate as necessary to meet effluent TP limits.	Acetic acid feed reduced to 350 gpd on 7/13. Acetic acid feed rate prior to reduction was 600 gpd
7/27/2010	56	Place 3rd NSL cell online. Eliminate acetic acid feed. (Optional – Dependent on results of additional testing)	3rd NSL Cell was not placed online as it was deemed unnecessary for process performance goals and due to the risk of developing filamentous organism population.
7/27/2010	56	Reduce sodium hydroxide feed to 450 gpd or as necessary to maintain secondary effluent alkalinity of 50 mg/L as $CaCO_3$ while maintaining stable nitrification and pH between 6.5 and 7.0.	Sodium hydroxide feed was reduced to ~400 gpd and further reduced to ~300 gpd on 8/5. Alum feed rates: 7/27 – 70 gal / mgd treated 8/3 – 62 gal / mgd treated
8/10/2010	70	Hazen and Sawyer to perform additional nutrient profiles, phosphorus uptake/release testing and chemical phosphorus removal jar testing	Alum feed rates: 8/10 – 70 gal / mgd treated 8/17 – 78 gal / mgd treated
Once Available	NA	Reduce MLSS alum feed by 20 mg/L (31 gallons 48% alum solution per mgd treated) once ability to feed alum to the secondary effluent is provided. Feed 20 mg/L of alum to secondary effluent. Adjust alum feed to MLSS as required to meet effluent TP limit.	Secondary effluent alum application point is not yet available.

Figure 16

Chemical Feed Rates



By operating the existing aeration basins in a step feed BNR configuration, increased TN removal through the MFWWTP Secondary Process helps comply with the upcoming Jordan Lake Nutrient Strategy mass load allocations. Increasing the removal of TN through the secondary system reduces the dependence on the denitrification effluent filters to meet the limits. Additional TN removal through the secondary process is also expected to significantly reduce supplemental carbon, alkalinity and aeration requirements once the new TN limits are in force.

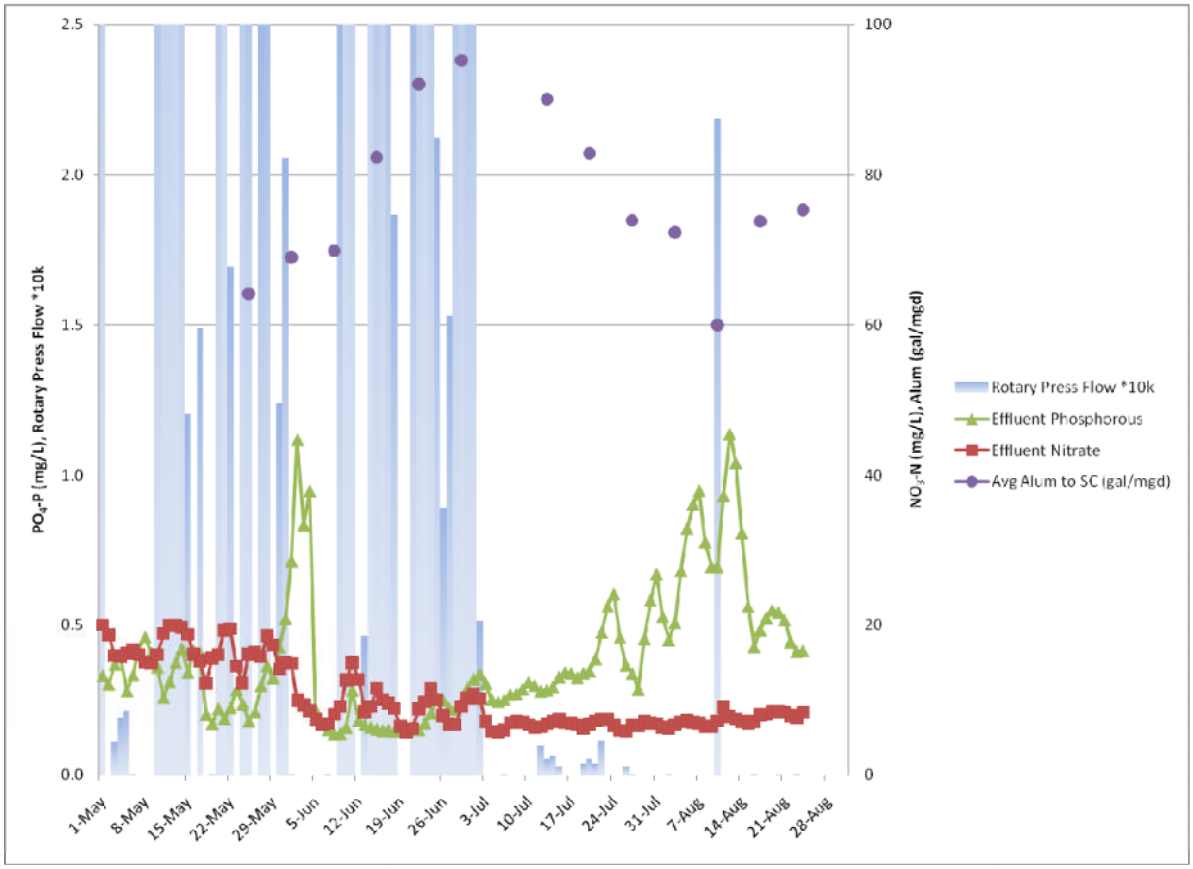
Figure 17 presents effluent ortho-phosphate and nitrate concentrations throughout the transition period as measured by the effluent nutrient analyzer (Chemscan). Effluent nitrate concentrations were reduced in half from approximately 16 mg/L before the process transition to 8 mg/L afterwards. Ortho-phosphate data obtained through the online instrumentation appeared to be approximately 0.1 mg/L higher than grab samples analyzed by Hazen and Sawyer. It is recommended that sample line cleanliness and calibration of the equipment be

confirmed. Also, it should be noted that the online parameter measured is ortho-phosphate as phosphorus (PO_4-P) versus the phosphorus indicated on the discharge monitoring report is total phosphorus (TP) and includes the digestion step in analysis. Plant staff should be informed of this difference to avoid potential reporting errors.

Rotary press operation and average weekly alum feed rates are also indicated on Figure 17. Operation of the rotary press led to increased effluent nitrate concentrations due to the return of ammonia in the filtrate and subsequent oxidation. Filtrate flow equalization should be considered if regular rotary press operation is anticipated once the effluent TN limit is reduced in 2016.

Figure 17

Effluent Nutrient Concentrations



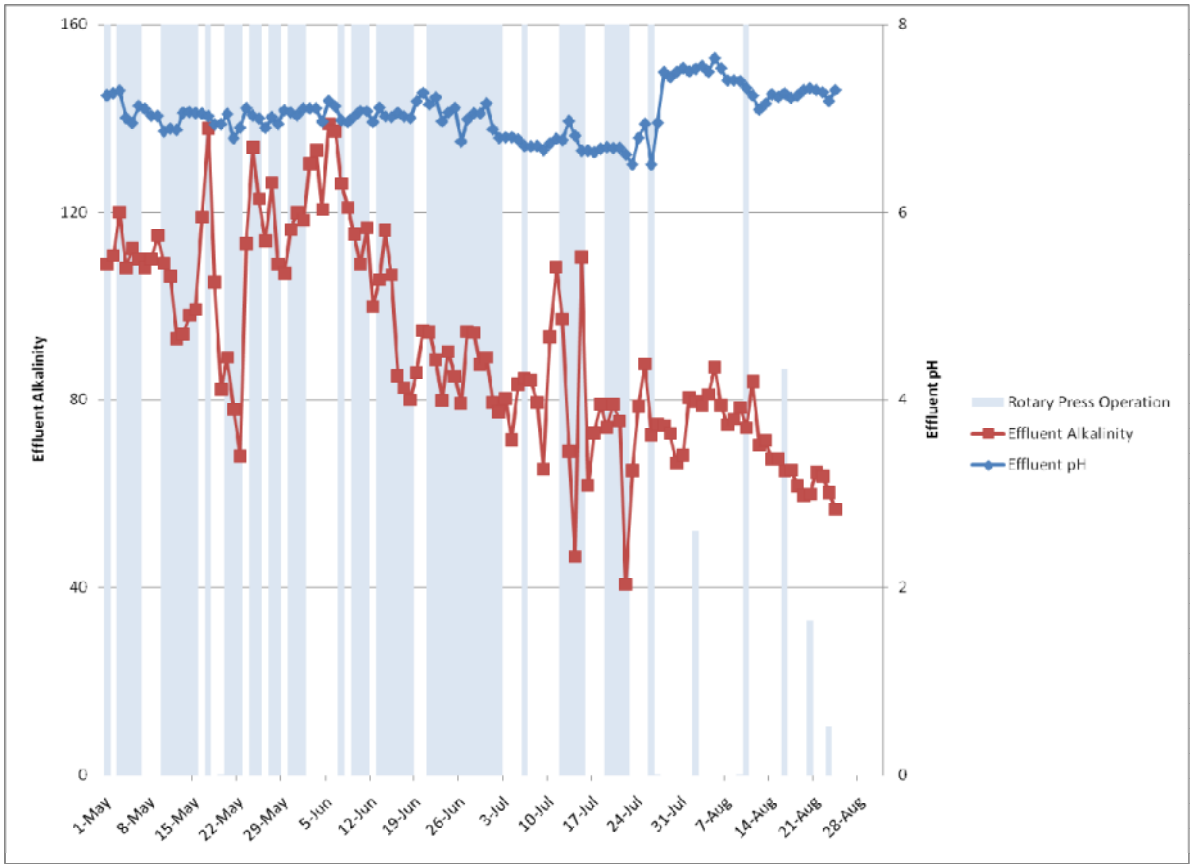
Approximately 1,400 gpd of 25% sodium hydroxide had been added at the MFWWTP to maintain pH prior to the process transition, and the average secondary effluent alkalinity has been approximately 75 mg/L as $CaCO_3$. The additional denitrification in the secondary process

being achieved by the process modifications reduces the supplemental alkalinity requirements to maintain adequate pH for nitrification. The step feed operation was estimated to reduce sodium hydroxide (25%) requirements to 900 gpd to maintain a secondary effluent alkalinity of 75 mg/L as CaCO3. It was recommended in the process transition plan that additional alkalinity reduction could be achieved by reducing the target secondary effluent alkalinity concentration to 50 mg/L as $CaCO_3$. Operating at this reduced secondary effluent alkalinity will also decrease sulfuric acid feed to meet the reuse system alkalinity target range once the effluent filters are run in denitrification mode.

Sodium hydroxide (caustic soda) feed has been reduced by 80% to below 300 gpd during the process transition. The current effluent alkalinity is approximately 60 mg/L as $CaCO_3$. Effluent pH has remained above 7. Figure 18 displays the effluent alkalinity and pH throughout the transition period. Rotary press operation is also indicated in the figure.

Figure 18

Effluent Alkalinity and pH



BPR efficiency is reduced at low pH, and operation at values less than approximately 6.5 at other facilities in the region has contributed to decreased phosphorus removal performance (Rohrbacher, 2008). Furthermore, research indicates that GAOs take up acetate faster than PAOs when anaerobic zone pH is less than 7.25, resulting in reduced PAO proliferation and GAO predominance at lower pH values. PAOs were also found to consume acetate at a greater rate than GAOs at anaerobic zone pHs greater than 7.5 (Filipe, et al. 2001a). Phosphate uptake is also pH dependent, and it is necessary to maintain a pH greater than 7.0 in the aerobic zone since PAOs are sensitive to the pH of the aerobic zones, but GAOs are not (Filipe, 2001b). Reducing the target secondary effluent alkalinity can result in significant operational savings, but care should be exercised to prevent operation at low pH (below 6.5) to reduce the potential for low pH inhibition of the PAOs and nitrifiers. OWASA has been successful during the process transition plan in drastically reducing sodium hydroxide feed while maintaining sufficient pH for reliable BPR and nitrification.

8. POTENTIAL COST SAVINGS

Overall chemical feed rate reductions throughout the transition period are indicated in Table 5. Table 6 develops the potential cost savings due to the reduction in chemical usage under current flows and permit limits, and Table 7 presents potential operational savings over the 20-year period beginning with implementation of the Jordan Lake TN standard. Note that these tables do not include the potential reduction in sulfuric acid feed for reclaimed water alkalinity adjustment. Chemical costs are based on the actual prices paid by OWASA at the time of the process transition and include tax and fuel surcharge (where applicable). Future chemical requirements are based on an annual average flow of 9.5 mgd over the period of 2016 to 2035. Table 7 assumes 20% acetic acid feed to the denitrification filters.

Table 5

Chemical Feed Rates

	May Average (gpd)	August Average (gpd)	Reduction
48% Alum	627[1]	507[1]	19%
25% Sodium Hydroxide	1,352	267	80%
20% Acetic Acid	634	338	47%

[1]Excludes alum feed to filtrate (227 gpd avg. in May, ~ 0 gpd in August)

Table 6

Potential Chemical Feed Savings – Current Flows

	Estimated Savings (gpd)	Current Cost ($/gallon)	Annual Savings
48% Alum	120	$0.72	$31,500
25% Sodium Hydroxide	1,090	$0.50[1]	$199,000
20% Acetic Acid	296	$0.86	$92,900
Total Annual Savings	-	-	$323,400

[1]Based on delivery of 50% sodium hydroxide and dilution to 25%

Table 7

Potential Chemical Feed Savings – Future Flows (Acetic Acid to Filters)

	Estimated Savings (gpd)	Current Cost ($/gallon)	Annual Savings
48% Alum	160	$0.72	$42,000
25% Sodium Hydroxide	1,450	$0.50[1]	$265,000
20% Acetic Acid	1,640	$0.86	$515,000
Total Annual Savings	-	-	$822,000

[1]Based on delivery of 50% sodium hydroxide and dilution to 25%

The average annual savings in chemical costs over the 20-year period from 2016 (implementation of the Jordan Lake TN limit) through 2035 is estimated to be $822,000 per year based on current chemical costs and performance during the transition period. The majority of the savings is attributed to decreased acetic acid usage.

9. ONGOING/FUTURE OPTIMIZATION RECOMMENDATIONS

The process transition to step feed was successful and resulted in both significant nitrogen and chemical usage reduction. It is recommended that OWASA continue operation in this mode to save on operational costs and gain experience with the revised flow path prior to implementation of the stricter TN standards in 2016. Although the work included under the current process transition scope is complete upon finalization and acceptance of this technical memorandum, several additional optimization/improvement steps should be considered by

OWASA and are described below.

- Cold Weather Operation Optimization - Operation under cold weather conditions may necessitate aerobic operation of the second anoxic zone (third cell) in each train to maintain full nitrification. Alternatively, the third step feed train could be placed into operation during cold weather conditions to maintain the second and third anoxic zones and provide greater TN removal. At increased flow and load conditions the third train will need to be operated year-round, and the second anoxic zone may need to be operated as aerobic volume in cold weather. It is recommended that OWASA closely monitor performance of the step feed process during cold weather. Operation of the second anoxic zone as aerobic volume should be piloted to determine the potential impact on nitrification and TN removal.

- Aeration Improvements – Process monitoring during the transition indicates limited aeration in Cell 4B. The condition and operation of the jet aeration and ancillary equipment in this zone should be investigated and corrected as necessary if not done so already. In addition, the potential aeration limitation in Cells 2C and 3B (center step feed train) may need to be addressed if cold weather step feed operation requires operation of the third train and/or flows and load increase and require activation of the third step feed train.

- Operation of the Third NSL Cell – Operation of the third NSL cell to provide additional fermentation and VFA production for BPR could be piloted if phosphorus release in the current two-cell configuration decreases due to reduced carbon availability to decrease the potential feed of acetic acid. Operation of the third NSL cell should be deferred while reliable BPR performance (as indicated by substantial release and uptake) is occurring.

- Operation of the Denitrification Filters in a Trim Mode – Operation of the denitrification filters in a trim mode prior to 2016 will familiarize OWASA staff with this operation before strict effluent TN limits will need to be met. In addition, various supplemental carbon sources could be piloted during this initial period to assist OWASA in determining which products may be used during full-scale operation starting in 2016 and allow for design and construction of improvements to incorporate various carbon sources.

- Glycerol Pilot to Step Feed Process – The process transition confirmed the substantial reduction in alkalinity feed attributed to denitrification in the secondary process as compared to removing nitrate across the filters, where the released alkalinity cannot assist the pH control of the secondary process. Furthermore the increase in alkalinity across the filters adversely impacts compliance with the reclaimed water alkalinity requirements. The potential for additional denitrification in the step feed process through supplemental carbon addition should be explored. Addition of glycerin products to anoxic zones has been successful in providing cost-effective denitrification. One potential strategy to allow OWASA to further reduce nitrate in the secondary process is to add glycerin at the second anoxic cell of the step feed process to supplement the carbon available in the primary influent sent to the first anoxic zone. This strategy could allow deferral of operation of the filters in a denitrification mode. Consideration should be given to adjusting the distribution of flow to provide more primary influent to the first aerobic cell if supplemental carbon is added in the secondary process.

- Provide an Alum Feed Point to the Secondary Clarifier Effluent – The chemical phosphorus removal testing indicated significantly better utilization when alum was added to the secondary clarifier effluent in comparison to the MLSS. OWASA should make improvements to provide an alum dose point to a common location upstream of the filters to optimize alum efficiency. It is noted that excessive coagulant addition to secondary clarifier effluent could cause filter blinding, and a careful balance must be struck between the multi-point feed of coagulant to the MLSS, where utilization is less efficient, and a tertiary process such as a filter.

- Installation of Additional Nutrient Analyzers – Additional nutrient analyzer equipment would provide OWSA greater process monitoring capability and flexibility. It is recommended that the ortho-phosphate analyzer installed prior to alum feed to the MLSS during the transition be made permanent to assist in alum feed optimization. Addition of an ortho-phosphate analyzer at the NSL effluent will allow OWASA to monitor the phosphorus release in the NSL zones and troubleshoot BPR performance. Nitrate and ammonia analyzers could be installed at the aeration basins to allow optimization of step feed configuration and flow splits to optimize TN removal while maintaining sufficient nitrification.

10. REFERENCES

Filipe, C.D.M.; Daigger, G.T.; and Grady, L., Jr. (2001a) pH as a Key Factor in the Competition Between Glycogen-Accumulating Organisms and Phosphorus-Accumulating Organisms. Water Environ. Res., 73, 223-232

Filipe, C.D.M.; Daigger, G.T.; and Grady, L., Jr. (2001b) Effects of pH on the Rates of Aerobic Metabolism of Phosphate-Accumulating and Glycogen-Accumulating Organisms. Water Environ. Res., 73, 213-222

Rohrbacher, J.; Bilyk, K.; Latimer, R.; Taylor, R.; and Pitt, P. (2008) Troubleshooting Biological Phosphorus Removal – The Middle Atlantic Experience. Proceedings of the 81st Annual Water Environment Federation Technical Exhibition and Conference (WEFTEC). Chicago, Illinois. 1790-1808.

Water Environment Research Foundation (2005) Factors Influencing the Reliability of Enhanced Biological Phosphorus Removal; Alexandria, Virginia.

MEMORANDUM

To: Christopher White, Hazen and Sawyer

From: Andrew Schuler, University of New Mexico

cc: Joe Rohrbacher, Hazen and Sawyer
Paul Pitt, Hazen and Sawyer

Date: 8/29/10

Analyst: Yunjie Tu, University of New Mexico

Re: Summary of OWASA data through 8/4/10 sampling

This brief memorandum summarizes measurements to date on samples from the OWASA plant sent for analyses at the University of New Mexico to evaluate parameters related to enhanced biological phosphorus removal (EBPR) and polyphosphate accumulating organisms (PAOs). Batch tests were conducted to measure rates of phosphorus release and acetate uptake while modifications are being made at the OWASA plant. These measurements are presented in the attached Excel file ("summary OWASA UNM 82910.xls"). Points of interest are summarized in Table 1 and the following graphs. Intermediate batch test measurements (e.g., at 60 minutes) may be of interest in later analyses, but this document focuses on measurements at the end of the batch tests (120 minutes).

Table 1: Measurements at 120 minutes in batch tests

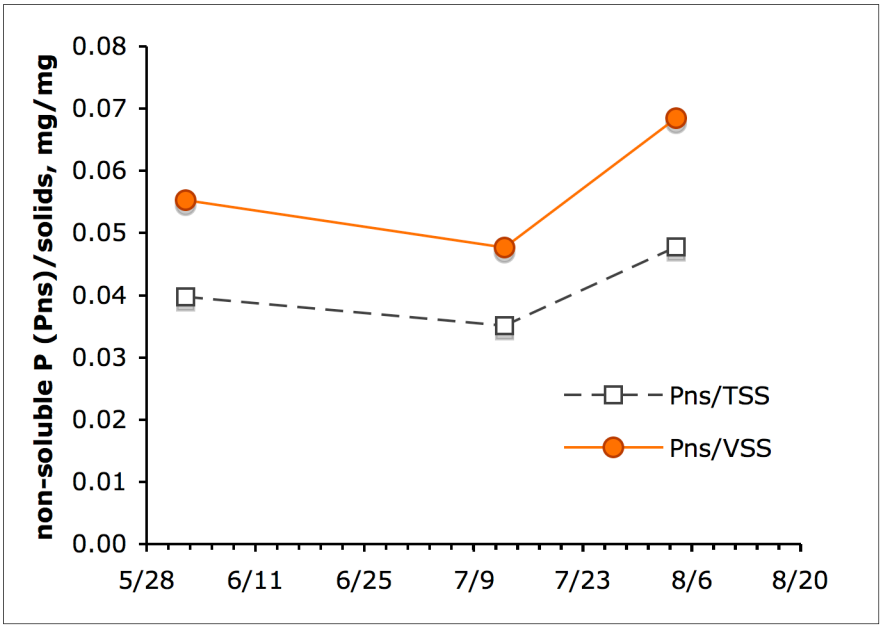
		Pulse acetate addition				**Slow acetate addition**			
	Pns /VSS	**Prel**	**Ac upt**	**Prel/Ac**	**Prel /Pns**	**Prel**	**Ac upt**	**Prel/Ac**	**Prel /Pns**
Date	mg /mg	mg/L	mg/L	mol P /mol C	mg /mg	mg/L	mg/L	mol P /mol C	mg /mg
6/2/10	0.055	13.8	68.0	0.20	0.08	11.6	44.5	0.25	0.07
7/13/10	0.048	18.9	55.9	0.33	0.15	18.5	48.0	0.37	0.15
8/4/10	0.068	23.0	42.4	0.53	0.12	23.1	33.6	0.67	0.12

Abbreviations:
Ac upt = anaerobic acetate uptake
C = carbon
P = phosphorus
Pns = non-soluble P
Prel = anaerobic P release
VSS = volatile suspended solids

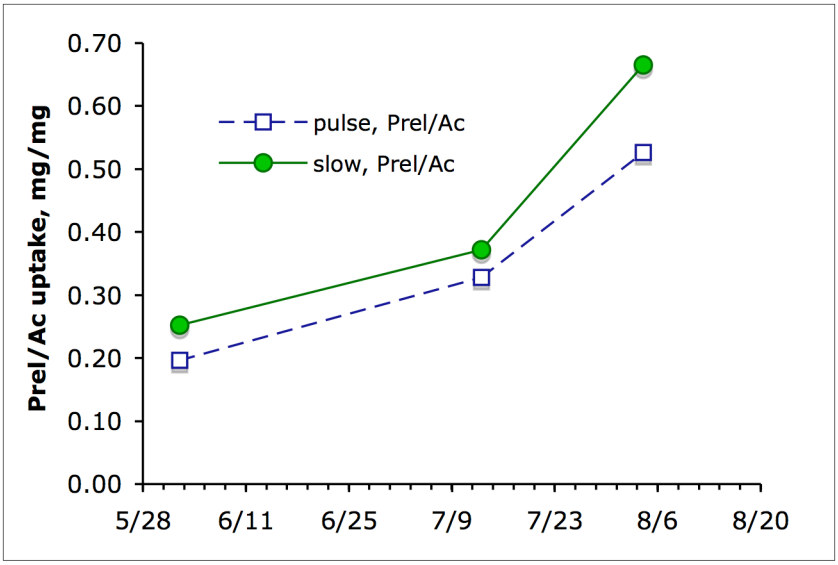
Observations

Major points

1. Polyphosphate content in biomass is increasing, based on increasing Pns/VSS values. This suggests PAO content is increasing.

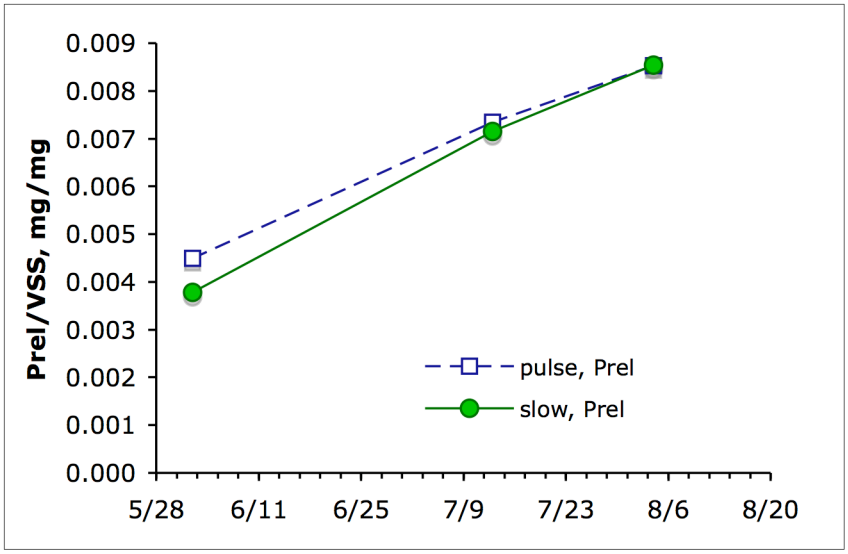


2. Prel/acetate uptake ratio is increasing, which suggests that PAO activity is increasing relative to GAO activity. "Pulse" and "slow" tests are batch tests where acetate is either added as a pulse at the beginning of the test, giving generally high acetate concentrations, or it was added slowly throughout the test, resulting in generally low acetate concentrations. This was done as part of an investigation of high acetate concentrations favor glycogen accumulating organisms (GAOs) and low acetate concentrations favor PAOs. The results support this hypothesis, with higher P release/acetate ratios in the slow acetate addition tests.

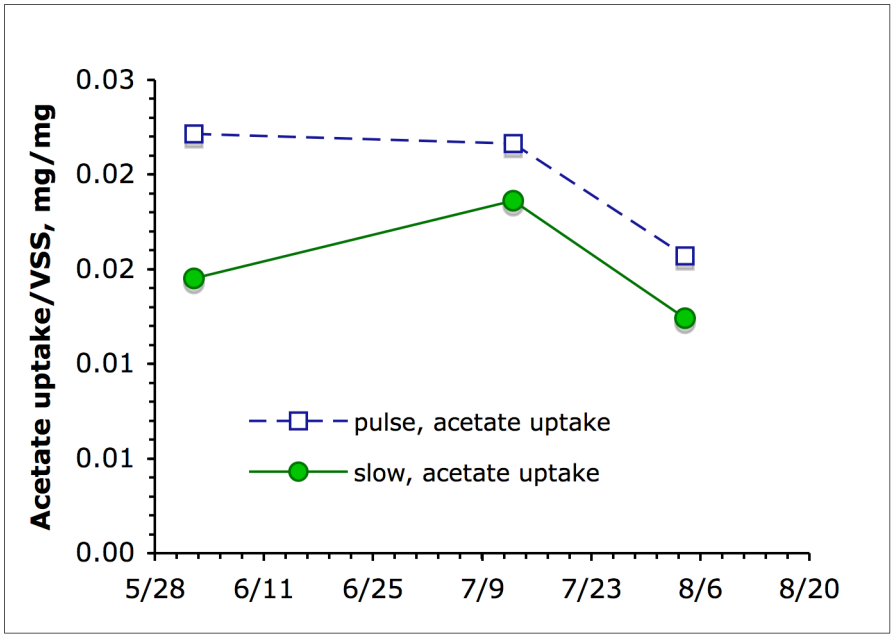


Minor points

3. Anaerobic P release (normalized to VSS below) is increasing ~ linearly with time, which indicates increasing PAO activity.



4. Anaerobic acetate uptake measurements yielded somewhat anomalous results – anaerobic acetate uptake actually decreased in the last test. This could be because GAO activity is dropping, but if our theory is correct, we would expect the "slow" test (where GAOs are expected to be less active) to not show this effect – however, the figure below shows acetate uptake decreased in the slow test as well. The next tests should be enlightening on this point – if PAO content continues to increase, we'd expect anaerobic acetate uptake to begin to increase.



APPENDIX B

The purpose of bench-scale jar tests is to determine an effective coagulant dose to supplement the biological processes. Various coagulant dosages are introduced to the jars containing wastewater under conditions of rapid mixing. Once the coagulation reactions are initiated and particle aggregation occurs, the velocity of the mixers is decreased to encourage particle collisions during flocculation. After mixing is stopped, the particles settle and the phosphorous concentration of the supernatant is measured.

Equipment

Phipps & Bird jar testing apparatus with 6 jars

1 – pH meter and probe

1 – 10 mL pipette and pipette tips

0.1 – 1.0 mL pipette and pipette tips

Glass fiber filters (1.2 um pore size)

Hach spectrophotometer and analysis kits

Procedure

1. Collect ~13 liters of wastewater (mixed liquor or secondary effluent).
2. Measure PO_4-P on the wastewater sample before starting the bench work.
3. Thoroughly mix the wastewater sample and add to the jars. Ensure even distribution of the sample to the jars by filling each jar approximately halfway before adding the final amount of sample. Remix the sample periodically to maintain solids in suspension.
4. Add coagulant to each jar and maintain one jar as a control with no coagulant added.

5. Start rapid mixing at 300 rpm for 30 seconds.

6. Reduce speed to 40 rpm for 15 minutes for secondary effluent samples and 10 minutes for mixed liquor samples.

7. Turn off the mixers, wait for the flocs to settle and proceed to filter the samples with a glass fiber filter. Note: During the mixed liquor test, as soon as the solids settle, remove a clarified portion from the top of the jar to separate the activated sludge from the clarified samples to decrease the possibility of phosphorous release during the filtering step.

8. Analyze samples and record temperature, pH, PO_4-P, and alkalinity.

APPENDIX C

Table C-1: MLSS Jar Test Results, Alum

Date	Coagulant Dose (mg/L)	PO_4^{3-} - P (H&S)	Temperature (°C)	pH	Alkalinity	Al:P Mole Ratio
April 28	0	0.77	22.1	7.30	123.2	-
	4.8	0.69	22.2	7.28		6.7
	9.6	0.66	21.6	7.26		9.4
	14.4	0.59	21.2	7.25	114.6	8.6
	19.2	0.56	21.0	7.25		9.7
	24	0.503	20.8	7.20	121.2	9.4
July 12	0	0.09	28.1	7.1		-
	10	0.09	27.9	7.1		-348
	20	0.12	27.8	7.1		-83
	30	0.10	27.6	7.1		-447
	40	0.07	27.6	7.2		245
	50	0.090	27.7	7.2		5215
August 10	0	0.675	28.8	7.0	57.1	-
	10	0.531	29.0	6.9		7.2
	20	0.399	28.8	6.9		7.6
	30	0.373	28.7	6.9	59.6	10.4
	40	0.252	28.4	6.9		9.9
	50	0.222	28.7	6.9	59.7	11.5

Table C-2: MLSS Jar Test Results, Sodium Aluminate

Date	Coagulant Dose (mg/L)	PO_4^{3-} - P (H&S)	Temperature (°C)	pH	Alkalinity	Al:P Mole Ratio
July 12	0	0.06	28	7.5		-
	5	0.045	27.9	7.4		126
	10	0.078	27.9	7.4		-210
	15	0.039	27.9	7.4		270
	20	0.046	27.9	7.4		540
	25	0.046	28	7.4		675
August 10	0	0.487	29.5	7.0	60.8	-
	5	0.33	29.4	7.1		12
	10	0.228	29.4	7.1		15
	15	0.165	29.2	7.2	62.2	18
	20	0.121	29.2	7.2		21
	25	0.096	29.2	7.2	65.4	24

Table C-3: SCE Jar Test Results, Alum

Date	Coagulant Dose (mg/L)	PO_4^{3-} - P (H&S)	Temperature (°C)	pH	Alkalinity	Al:P Mole Ratio
April 28	0	0.412	20.5	7.40	130	
	4.8	0.312	20.5	7.36		5.0
	9.6	0.195	20.5	7.30		4.6
	14.4	0.125	20.3	7.24	125	5.2
	19.2	0.087	20.4	7.18		6.2
	24	0.067	20.4	7.16	122	7.3
July 12	0	0.156	29.0	7.2		
	10	0.056	29.0	7.2		10
	20	0.004	28.9	7.2		14
	30	0.023	28.8	7.2		24
	40	0.051	28.7	7.2		40
	50	0.029	28.8	7.3		41
August 10	0	0.527	29.6	7.2	59.9	
	10	0.247	29.5	7.3		3.7
	20	0.086	29.4	7.4		4.7
	30	0.049	29.3	7.5	58.2	6.5
	40	0.027	29.1	7.5		8.3
	50	0.025	29.2	7.7	58.7	10.4

Table C-4: SCE Jar Test Results, Sodium Aluminate

Date	Coagulant Dose (mg/L)	PO_4^{3-} - P (H&S)	Temperature (°C)	pH	Alkalinity	Al:P Mole Ratio
July 12	0	0.184	28.7	7.3		
	5	0.349	28.6	7.4		-11
	10	0.08	28.6	7.4		36
	15	0.106	28.6	7.4		73
	20	0.101	28.6	7.4		91
	25	0.114	28.6	7.4		135
August 10	0	0.602	30.5	7.5	64.4	
	5	0.4	30.5	7.6		9.4
	10	0.24	30.5	7.7		10.4
	15	0.169	30.5	7.8	66.1	13.1
	20	0.195	30.5	8.1		18.6
	25	0.099	30.6	7.9	64.2	18.8

Table C-5: Filtrate Jar Test Results, Alum

Date	Coagulant Dose (mg/L)	PO_4^{3-} - P (H&S)	Temperature (°C)	pH	Alkalinity	Al:P Mole Ratio
April 28	0	38.4	22.4	8.38	3032	
	48	34.45	22.8	8.35		1.3
	96	31.55	23.1	8.30		1.5
	144	30.3	22.7	8.24	2840	1.9
	192	25.85	22.8	8.19		1.6
	240	24.7	22.9	8.17	2500	1.8

Summary of Test Results and Proposal for Work
OWASA EBPR Study
A. Schuler
6/18/10

1. Summary

An activated sludge sample from the OWASA plant taken on 6/1/10 and analyzed on 6/2 and 6/3/10 at the University of New Mexico exhibited the following characteristics:
- Very low PAO content by Neisser staining and FISH
- Very low PAO content according to P release measured in an anaerobic batch test.
- Very low PAO/GAO ratio according to the P release/acetate uptake ratio measured in anaerobic batch tests with 2 different rates of acetate addition.

2. Background

The OWASA plant is currently undergoing a series of modifications to improve EBPR and decrease alum addition for phosphorus removal. A sample was sent to A. Schuler at the University of New Mexico for analyses, including fluorescent in-situ hybridization (FISH) for PAOs, Neisser staining (which I understand is already being done), and anaerobic phosphorus release/total P and phosphorus release/acetate uptake assays.

We have ongoing research in our laboratory looking at the P release/acetate uptake ratio in particular. This is follow-on work began in my research at Berkeley, which demonstrate that the anaerobic P release/acetate uptake ratio can be used as an indicator of relative PAO and GAO activity (Schuler and Jenkins (2003)), where the following range of values was suggested:

Table 1

Anaerobic P/Ac ratio (mol/C-mol)	
0.25	GAO dominated
0.25–0.5	Intermediate
0.5	PAO dominated
0 to 8	Total observed range

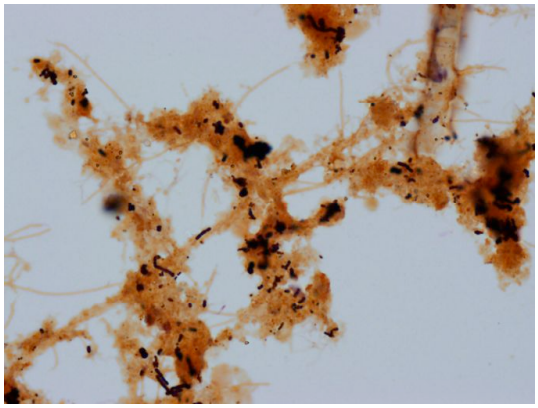
3. Methods

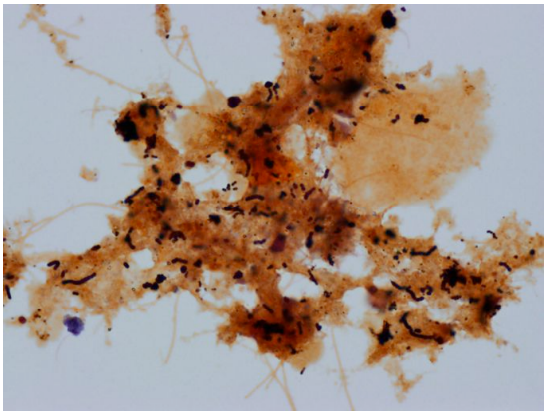
On 6/2/10 we received an activated sludge sample from the OWASA plant for analyses of EBPR characteristics. The following analyses/procedures were performed:

A. Neisser Stain
B. FISH with the "PAO mix" set of probes for most *Accumulibacter* (PAO462, PAO651,and PAO846), and a general probe for Eubacteria
C. A batch test was conducted where the sample was warmed to room temperature, stripped with N2 gas, 240 mg/L acetate was added, and phosphorus and acetate concentrations were measured over 3 hours ("pulse test")
An additional test performed where acetate was added very slowly with a syringe pump ("slow test"). This ties into the research of my doctoral student (Yunjie Tu), who is looking at the effects of substrate concentrations on PAO and GAO uptake rates.

4. Results

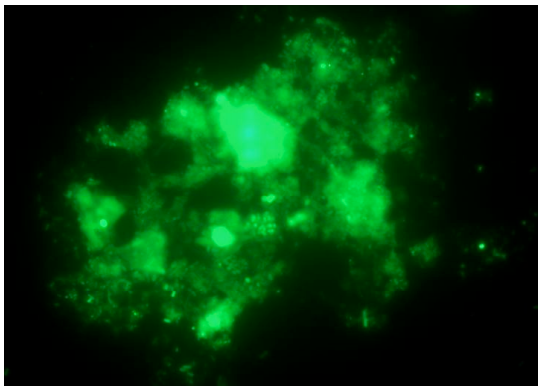
A. Typical Neisser stain results are shown in the figures below (1000x).



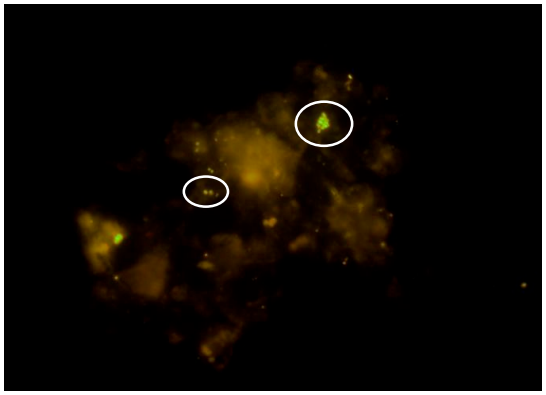


These figures demonstrate very low PAO content (dark purple cells) and very low tetrad content (a characteristic morphology of some, but not all, GAOs).

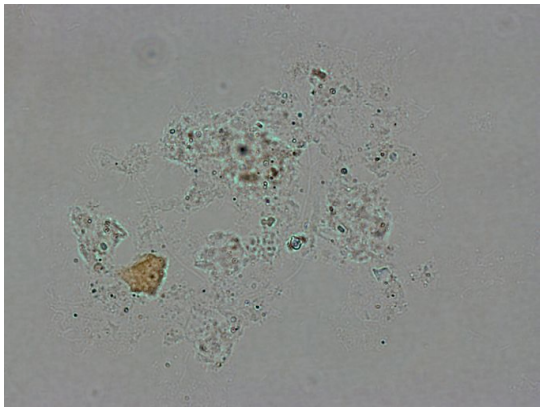
B. Typical fluorescent in-situ hybridization (FISH, 1000x).



EUB probe: most bacteria



PAO mix probes: Accumulibacter



Brightfield (no phase contrast currently available at this magnification)

The FISH images demonstrate very low PAO content – a few PAO positive cells are circled in the second image.

C. Batch tests.
The 3 hour anaerobic batch tests yielded very low amounts of P release, indicating low PAO content. Although the non-soluble P (Pns)/TSS ratio was relatively high at about 4 % (OWASA data), of the 170 mg Pns/L, only 13 to 15 mg P/L was released in the batch tests 8 to 9% of Pns), which was consistent with the high Pns being due to precipitated P rather than polyphosphate storage.

The P release/acetate uptake ratio was also low: in the "pulse" test, the ratio was about 0.20 Pmol/Cmol, which indicated a GAO-dominated culture (Table 1), although since both populations were low, this may not mean much. Similar results were found in the "slow" test. The P release/acetate uptake ratio was higher in the slow addition test, which supports a hypotheses that we are currently demonstrating in our lab system about competition between PAOs and GAOs (this is an important part of Yunjie's thesis – please don't share broadly).

5. Proposed work
I propose that we continue with the P release/acetate uptake and P release/Pns tests to evaluate the total PAO activity and the relative PAO/GAO activity as your plant progresses through its modifications. This would give a good measure of PAO increase and also whether GAOs are contributing to observed activities. This would be very interesting to us, as it ties in well with Yunjie's research and gives him a chance to do full scale verification with a plant that is actively changing its operation.

We can also fix samples for FISH, which can be imaged later for PAOs (easy) and possibly for GAOs if we want to buy more probes. As we discussed, the PAO probe doesn't really tell you anything that a Neisser stain doesn't, but having these images can be good for publishing, and fixing is pretty easy.

We could do both of these analyses periodically as the modifications are made to the system – perhaps 5 or so sampling events.

References
Schuler, A.J. and Jenkins, D. (2003) Enhanced biological phosphorus removal from wastewater by biomass with varying phosphorus contents, Part I: Experimental methods and results. Water Environ. Res. 75(6), 485-498.

MEMORANDUM

To: Christopher White, Hazen and Sawyer

From: Andrew Schuler, University of New Mexico

cc: Joe Rohrbacher, Hazen and Sawyer
Paul Pitt, Hazen and Sawyer

Date: 8/29/10

Analyst: Yunjie Tu, University of New Mexico

Re: Summary of OWASA data through 8/4/10 sampling

This brief memorandum summarizes measurements to date on samples from the OWASA plant sent for analyses at the University of New Mexico to evaluate parameters related to enhanced biological phosphorus removal (EBPR) and polyphosphate accumulating organisms (PAOs). Batch tests were conducted to measure rates of phosphorus release and acetate uptake while modifications are being made at the OWASA plant. These measurements are presented in the attached Excel file ("summary OWASA UNM 82910.xls"). Points of interest are summarized in Table 1 and the following graphs. Intermediate batch test measurements (e.g., at 60 minutes) may be of interest in later analyses, but this document focuses on measurements at the end of the batch tests (120 minutes).

Table 1: Measurements at 120 minutes in batch tests

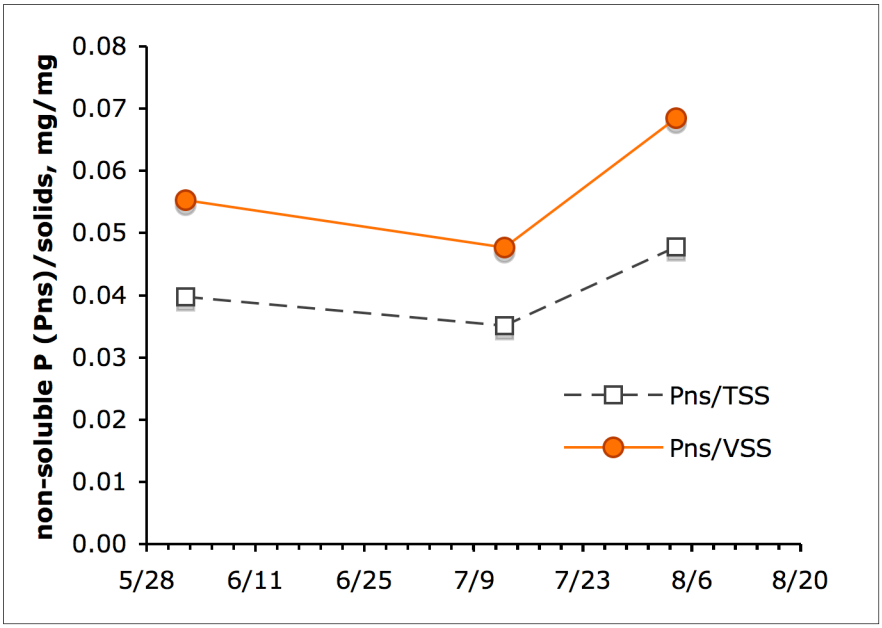
		Pulse acetate addition				**Slow acetate addition**			
	Pns /VSS	**Prel**	**Ac upt**	**Prel/Ac**	**Prel /Pns**	**Prel**	**Ac upt**	**Prel/Ac**	**Prel /Pns**
Date	mg /mg	mg/L	mg/L	mol P /mol C	mg /mg	mg/L	mg/L	mol P /mol C	mg /mg
6/2/10	0.055	13.8	68.0	0.20	0.08	11.6	44.5	0.25	0.07
7/13/10	0.048	18.9	55.9	0.33	0.15	18.5	48.0	0.37	0.15
8/4/10	0.068	23.0	42.4	0.53	0.12	23.1	33.6	0.67	0.12

Abbreviations:
Ac upt = anaerobic acetate uptake
C = carbon
P = phosphorus
Pns = non-soluble P
Prel = anaerobic P release
VSS = volatile suspended solids

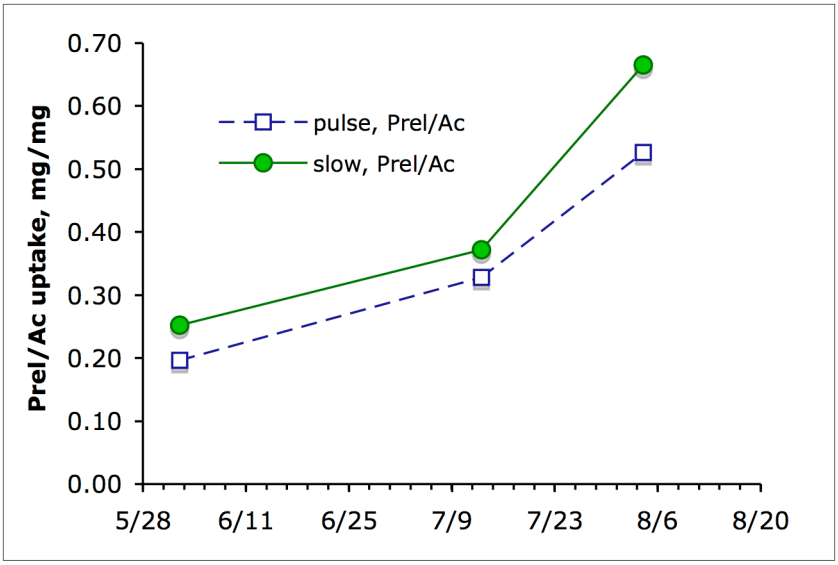
Observations

Major points

1. Polyphosphate content in biomass is increasing, based on increasing Pns/VSS values. This suggests PAO content is increasing.

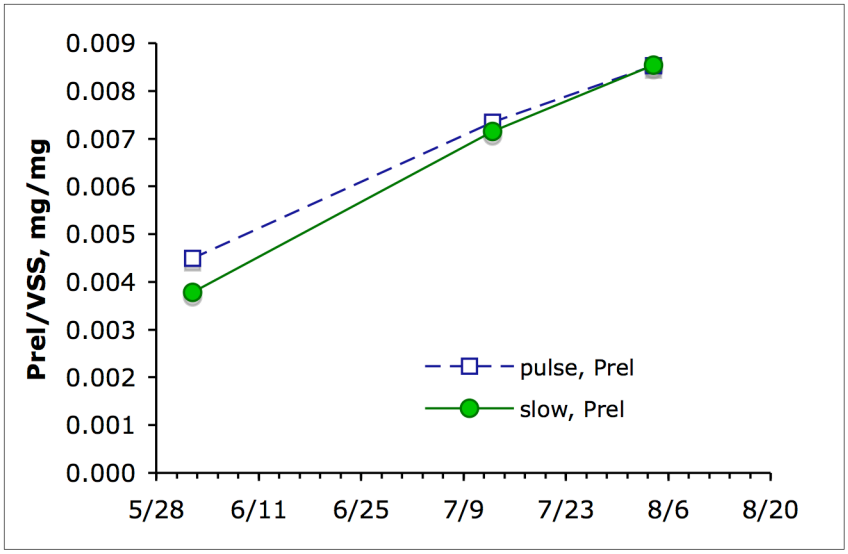


2. Prel/acetate uptake ratio is increasing, which suggests that PAO activity is increasing relative to GAO activity. "Pulse" and "slow" tests are batch tests where acetate is either added as a pulse at the beginning of the test, giving generally high acetate concentrations, or it was added slowly throughout the test, resulting in generally low acetate concentrations. This was done as part of an investigation of high acetate concentrations favor glycogen accumulating organisms (GAOs) and low acetate concentrations favor PAOs. The results support this hypothesis, with higher P release/acetate ratios in the slow acetate addition tests.

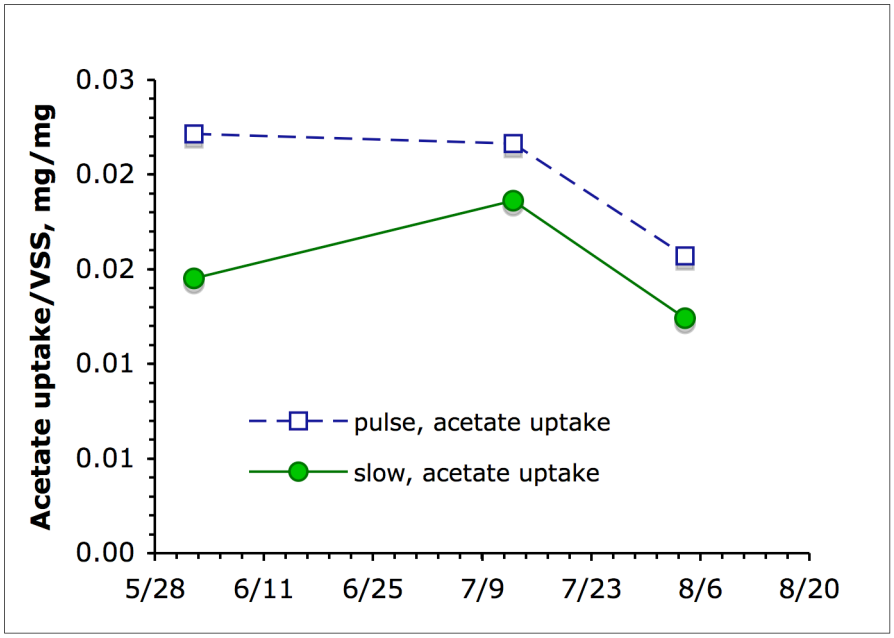


Minor points

3. Anaerobic P release (normalized to VSS below) is increasing ~ linearly with time, which indicates increasing PAO activity.



4. Anaerobic acetate uptake measurements yielded somewhat anomalous results – anaerobic acetate uptake actually decreased in the last test. This could be because GAO activity is dropping, but if our theory is correct, we would expect the "slow" test (where GAOs are expected to be less active) to not show this effect – however, the figure below shows acetate uptake decreased in the slow test as well. The next tests should be enlightening on this point – if PAO content continues to increase, we'd expect anaerobic acetate uptake to begin to increase.



Technical Memorandum

PREPARED FOR:	Orange Water and Sewer Authority (OWASA)
PREPARED BY:	Hazen and Sawyer, P.C.
REVIEWED BY:	Ron Taylor (Hazen and Sawyer)
DATE:	October 24, 2010 Revised June 20, 2011
SUBJECT:	Mason Farm WWTP Aeration Basins Odor Modeling

1.0 INTRODUCTION AND PURPOSE

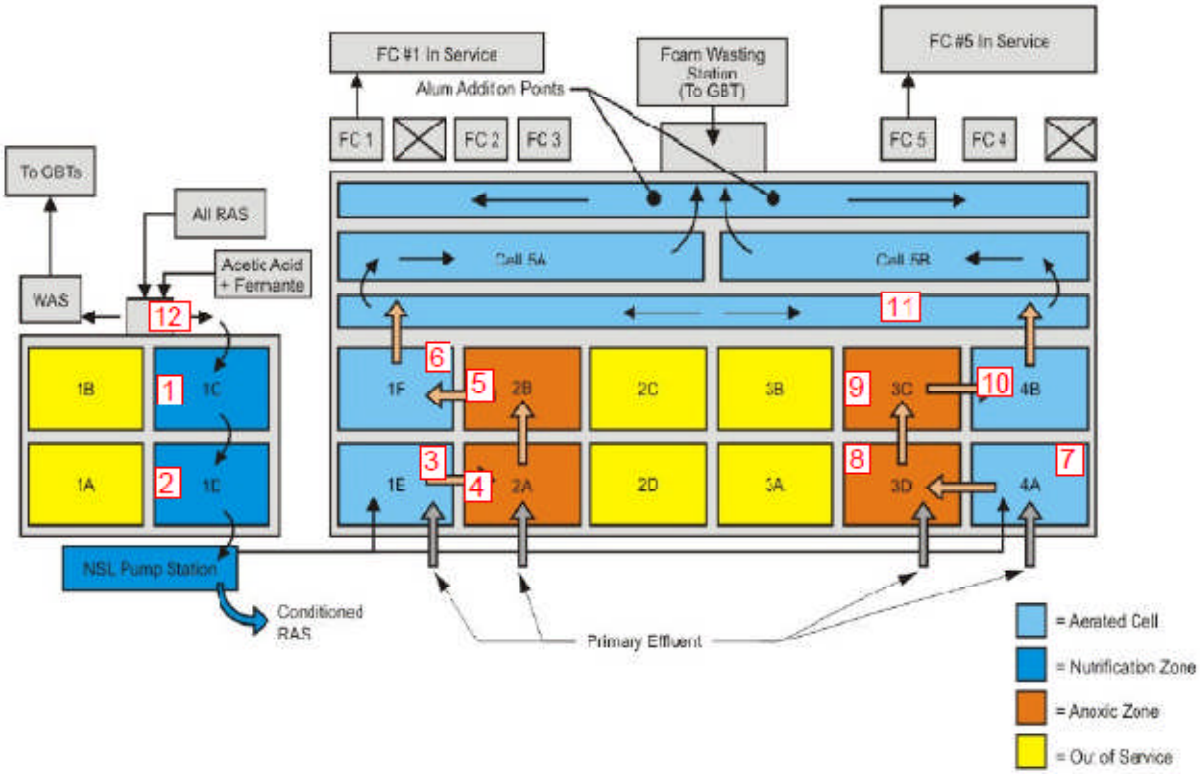
The Orange Water and Sewer Authority (OWASA) owns and operates the Mason Farm WWTP. This Technical Memorandum addresses odor emissions from the Mason Farm WWTP as they may be impacted by recommended changes to the treatment process configuration. Black and Veatch had previously been contracted to perform a comprehensive odor control evaluation and Brown and Caldwell has been contracted to design odor control improvements for the Mason Farm WWTP. Phase 3 of the odor control improvements includes activated carbon odor control adsorbers and formed aluminum truss supported covers for capturing and treating foul air from portions of the aeration basins and nutrified sludge (NSL) basins. Hazen and Sawyer recently completed a Hydraulic and Treatment Capacity Study for the Mason Farm WWTP, which recommends a biological process change to step feed BNR and a new fine bubble diffused aeration system. The diffused aeration system upgrade will replace the existing coarse bubble diffuser system with a higher efficiency system that will use less air for biological treatment and, therefore, will generate less air requiring odor control. The proposed change to the biological process could impact the odor emission rates from the various aeration basin zones, which could impact odors from the facility. Therefore, OWASA has requested that H&S reevaluate potential odors from the aeration basins based on the recommended process changes using dispersion modeling.

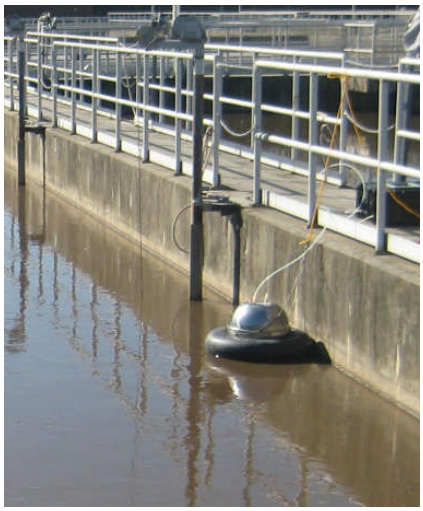
The purpose of this study was to reevaluate potential odors from the upgraded aeration basins based on the recommended process changes using air dispersion modeling. This Technical Memorandum outlines the results of the sampling, dispersion modeling and provides recommendations for changes required to the proposed odor control system.

2.0 SAMPLING

Field sampling was conducted at the Aeration Basins on September 8, 2010 and on November 2, 2010. Samples were collected from different zones in the Aeration Basins and NSL basins. The locations where samples were collected are shown in **Figure 1**.

Figure 1 - Sample Locations





All samples collected were from area sources. Area sources were sampled by placing an equilibrium flux chamber directly over the water surface to eliminate any dilution due to intrusion of ambient air. The chamber was connected to a vacuum chamber via Tygon tubing, collecting air samples in 10 liter Tedlar bags.

Sensory samples were collected in 10 liter Tedlar bags and sent via overnight air to a sensory laboratory where the intensities of the odor samples were determined. Samples are analyzed within 24 hours. Prior to shipping the samples, the H_2S concentration in each bag was analyzed.

For sensory samples, total odor levels are determined by members of a sensory panel, all of whom have been screened for their sensory perception. The panel is comprised of members with a spectrum of sensory perceptions, thereby attempting to represent a typical community. A “forced choice” dynamic olfactometer is used. The olfactometer is the instrument used to present odor samples to the sensory panel. The olfactometer presents 3 samples to each panelist: the diluted odor sample plus two odor free samples. The panelist is forced to choose which sample contains the odor. This process continues and the dilutions get less and less (the odor gets stronger and stronger) until the undiluted sample is reached. Samples are presented to each panelist at an airflow rate of 20 L/min. The sensory evaluation was performed by St. Croix Sensory in St. Croix, Minnesota.

The following values are determined from the panel evaluations:

1. Dilution/Threshold (D/T) Ratio: The D/T ratio is effectively used to quantify the strength of the odor at the source. The D/T ratio indicates the number of dilutions required to reach a threshold of 1. A D/T of 1 is defined when 50% of the panel members can no longer witness the odor. Therefore, a source with a D/T of 1,000 indicates that it requires 1,000 dilutions to reach a D/T of 1. The point at which none of the panel members can witness the odor is corresponds to a D/T of 0.1 (log scale).

2. Odor Persistency (Dose-Response): The strength of the odor alone will not determine the impact of odor in the atmosphere. Some odors will be more pervasive than others and will tend to linger longer. The dose-response slope provides information as to the pervasiveness of the odors. The flatter the slope, the higher the tendency for the odor to linger. When hydrogen sulfide (H_2S) is the primary odorant, the expected dose-response slope is approximately -0.5. Where the slope is flatter (between -0.5 and 0), other more pervasive odorants may be present, such as mercaptans and reduced sulfur compounds. Where the slope is steeper (less than -0.5), less pervasive odorants may be present, such as ammonia.

   To determine the dose-response slope, the perceived intensity of the odor is plotted against the log of the dilution ratio (D/T). A butanol intensity measurement is used to determine the "perceived intensity" and the olfactomer is used to determine the dose-response slope. A linear regression of the plotted data is performed and the slope, y-intercept and r^2 (an indicator of how closely the regression fits the data) values are determined by the laboratory. The Hazen and Sawyer standard for this regression analysis is that the r^2 value be greater than 0.9 (an r^2 value of 1.0 indicates a perfect fit).

The results of the field sampling are shown in **Table 1** below.

Table 1 – Field Sampling Results

Location	Sample #	D/T	H_2S (ppm(v))
Cell 1C (NSL)	1	70, 1800	<0.20
Cell 1D (NSL)	2	210, 460	<0.20
Cell 1E (1st Aerobic)	3	280, 490	<0.20
Cell 2A (1st Anoxic)	4	520, 1300	<0.20
Cell 2B (2nd Anoxic)	5	210, 260	<0.20
Cell 1F (2nd Aerobic)	6	240, 380	<0.20
Cell 4A (1st Aerobic)	7	640	0.003
Cell 3D (1st Anoxic)	8	440	0.006
Cell 3C (2nd Anoxic)	9	1000	0.010
Cell 4B	10	530	0.005
Effluent Channel	11	210, 650	<0.20
NSL Influent Channel	12	1100, 2000	0.25

Some discussion of the field sampling results is warranted. The H_2S data collected in the two sampling events were tested at differing detection limits by the testing laboratory. In all cases, the H_2S levels are very low. The sampling results (D/Ts) were compared to previous sampling events at the WWTP, conducted by both Black and Veatch and Hazen and Sawyer, to see if they are consistent with previous sampling events. The sample comparison is shown in Table 2.

Table 2 – Comparison of D/T Sampling Data with Previous Studies at the WWTP

Zone	**Current (2010)**	B&V (2007)	H&S (2003)[1]
NSL Influent Channel	**1100, 2000**	--	--
NSL Zone 1C	**70, 1800**	--	--
NSL Zone 1D	**210, 460**	--	--
NSL Effluent Channel	**--**	--	--
Basin Influent Channel	**--**	1500	1596
Zone 1E (1st aeration)	**280, 490**	--	1596
Zone 2A (anoxic)	**520, 1300**	1300	655, 493
Zone 2B (anoxic)	**210, 260**	320	2340
Zone 1F (2nd aeration)	**240, 380**	150	714, 527
Zone 4A (1st aeration)	**640**	--	--
Zone 3D (anoxic)	**440**	--	--
Zone 3C (anoxic)	**1000**	--	--
Zone 4B (2nd aeration)	**530**	--	--
Mixed Liquor Effluent Channel (Zones 5A and 5B)	**210, 650**	--	624

[1]Analyzed at 0.5 LPM presentation rate and converted to 20 LPM presentation rate.

Variations in sample results would be expected based on changing influent wastewater characteristics, season of the year and due to the process changes that occurred between sampling events. Engineering judgment is applied to determine the D/T values to use for further analysis. D/T values were selected for each zone of the treatment process as follows:

- NSL Influent Channel and NSL Zone 1C: The recent sampling data for these two zones indicated that the D/T values were very similar. This correlates with expectations in that these two areas are very similar and odor concentrations are expected to be similar as well. During H&S's first round of sampling, the D/T for NSL zone 1C was 70, which was extremely low, and not inline with the other sampling data for those zones. Therefore, the 70 D/T value was discarded. The D/T recommended to be used for these zones was an average of the other D/T values. **D/T = 1550**.

- NSL Zone 1D and NSL Effluent Channel: Although it was expected that this zone would have a similar odor concentration as zone 1C and the NSL influent channel, this was consistently not the case in both sampling events. Therefore, the lower odor concentration in this zone was considered valid and the D/T recommended for this zone was an average of the two D/T sampled values. **D/T = 335**.

- Basin Influent Channel: This channel was not sampled by H&S, therefore the B&V sampled value of 1500 was used. This appears consistent with the aerated environment of the influent channel. **D/T = 1500**.

- Aeration Basins (1st and 2nd aeration; 1st and 2nd anoxic): During the recent sampling, these values were in some cases higher and some cases lower than the previous sampling. However, similar trends were evident when samples between different zones were compared. Therefore, all of these samples were considered valid and the D/T used for these zones was an average of all three sampled values in that process zone. The sole exception to that was the sampled value from Zone 4B, which was determined to not have adequate dissolved oxygen concentration during the September 2010 sampling event, and, therefore, that value was discarded.
    - Zone 1E/4A (1st aerobic): **D/T = 470**
    - Zone 2A/3D (1st anoxic): **D/T = 753**
    - Zone 2B/3C (2nd anoxic): **D/T = 490**
    - Zone 1F/4B (2nd aerobic): **D/T = 383**

- Mixed Liquor Effluent Channel: During the September 2010 H&S sampling event, it was determined that this sample had not been taken at a representative area of the channel. Therefore, this value was discarded and resampled. The new sampling value was considered to be representative of the conditions existing in the effluent channels. **D/T = 210**.

3.0 EXHAUST RATES AND ODOR EMISSION RATES

The first step in evaluating the impact of odor emissions from the aeration basins is to determine the exhaust rates and odor emission rates. The odor concentration, or D/T, alone can not be used to determine the odor impact of a facility. The total emissions into the atmosphere is dependent on the mass rate of release, which is a product of both concentration and the exhaust rate.

The exhaust rates were first determined based on the proposed process changes and aeration rates. For aerated zones, the exhaust rate is the maximum aeration rate, in this case the max day airflow rate. For anoxic zones, the exhaust rate was calculated based on the surface area, amount of turbulence and the evaporation rate.

The odor emission rate (OER) is the product of the D/T concentration multiplied by the exhaust rate. The OER is the mass flow rate of the odors entering the atmosphere. Remember, the OER can be high either due to a high D/T or a high exhaust rate, since it is a product of the two factors. The OER is generally used to determine which odors are the worst problem at the site and is used as an input in the dispersion modeling discussed in Section 4.0 below.

OWASA requested that this analysis be conducted at both projected 2020 airflow rates and projected 2030 airflow rates to determine whether the odor control for the basins may benefit from a further phasing approach. A summary of the 2020 and 2030 maximum day aeration rates for both fine bubble aeration and jet aeration is shown in **Table 3** below. To apply a level of conservatism in the modeling, the maximum day aeration rates assume the third aeration cell will be aerated although this will only be necessary under worst case conditions. This condition

is likely to occur in cold weather months when flows approach levels projected for the Year 2030.). It should be noted that future increases in influent loads will increase oxygen demands in the treatment process and this has been accounted for; however, increased loads are not anticipated to materially impact D/T values used in the analysis.

Table 3 – Comparison of Airflow Rates

Cell	2020 Fine Bubble	2030 Fine Bubble	Jet Mixing (2030)
Total	14,900	19,600	35,200
NSL 1C/B	0	0	0
NSL 1A/D	0	0	0
AB Inf Chan	75	98	176
AB 1	5,453	7,174	12,883
AB 2	0	0	0
AB 3	4,038	5,312	9,539
AB 4	3,367	4,430	7,955
Eff Chan 1	477	627	1,126
5A/5B	1,222	1,607	2,886
Eff Chan 2	268	353	634

The airflow rates for fine bubble diffused aeration are much lower than the requirements for jet aeration. The revised exhaust rates and OERs are summarized in **Table 4** and **Table 5** below for 2020 and 2030 projected airflow rates.

Table 4 – Summary of Exhaust Rates and Odor Emission Rates Based On 2020 Air Flows

Area	Aerobic or Anoxic?	D/T	Exhaust Rate (per Zone)	Area (SF)	Qty	OER/10^6
NSL Influent Channel	Anoxic	1550	80	400	1	0.12
NSL Zones 1C, 1B	Anoxic	1550	500	2500	2	1.55
NSL Zones 1D, 1A	Anoxic	335	500	2500	2	0.34
NSL Effluent Channel	Anoxic	335	66	330	1	0.02
Basin Influent Channel	Aerobic	1500	73	1925	1	0.11
Zones 1E, 3A and 4A (1st AB Cell)	Aerobic	470	2,321	2500	3	3.27
Zones 2A, 2D and 3D (2nd AB Cell)	Anoxic	753.33	500	2500	3	1.13
Zones 2B, 2C and 3C (3rd AB Cell)	Anoxic	490	500	2500	3	0.74
	Aerobic	383.33	1,346	2500	3	1.55
Zones 1F, 3B and 4B (4th AB Cell)	Aerobic	383.33	1,777	2500	3	2.04
Mixed Liquor Effluent Channel (Zones 5A and 5B)	Aerobic	210	2,534	8132	1	0.53

Table 5 – Summary of Exhaust Rates and Odor Emission Rates Based On 2030 Air Flows

Area	Aerobic or Anoxic?	D/T	Exhaust Rate (per Zone)	Area (SF)	Qty	OER/10^6
NSL Influent Channel	Anoxic	1550	80	400	1	0.12
NSL Zones 1C, 1B	Anoxic	1550	500	2500	2	1.55
NSL Zones 1D, 1A	Anoxic	335	500	2500	2	0.34
NSL Effluent Channel	Anoxic	335	66	330	1	0.02
Basin Influent Channel	Aerobic	1500	97	1925	1	0.14
Zones 1E, 3A and 4A	Aerobic	470	3,053	2500	3	4.30
Zones 2A, 2D and 3D	Anoxic	753.33	500	2500	3	1.13
Zones 2B, 2C and 3C	Anoxic	490	500	2500	3	0.74
	Aerobic	383.33	1,771	2500	3	2.04
Zones 1F, 3B and 4B	Aerobic	383.33	2,337	2500	3	2.69
Mixed Liquor Effluent Channel (Zones 5A and 5B)	Aerobic	210	3,334	8132	1	0.70

Exhaust rates and OERs were calculated for Zones 2B, 2C and 3C under both anoxic and aerobic scenarios. The contribution of each source (based on OER) is shown in the final column of Table 5 above. Note that Zones 2B, 2C and 3C have a greater contribution to odors in the atmosphere when operated in an aerobic mode in comparison with operation under an anoxic mode. The normal mode of operation is expected to be in anoxic conditions for these cells. In order of priority based on OER, the first aerobic cells (1E, 3A, 4A) are the largest contributors at approximately 35% of the total OER, followed by the second aerobic Cells (1F, 3B, 4B) at approximately 22% of the total OER, followed by the NSL zones at 15% of the total OER.

In addition to looking at each individual zone, totalized samples for the aeration basins were also considered. Each possible scenario was considered in terms of zones to cover and odor control. These thirty scenarios are summarized in **Table 6** below for both 2020 and 2030 airflow rates.

Table 6 – Total Aeration Basin Scenarios

Cases Modeled	2020 Odor Emission Rate (o.u. x cfm x 10^6)	2030 Odor Emission Rate (o.u. x cfm x 10^6)
"A" Cases - Two zones anoxic		
"A" Total All Zones	9.85	11.73
Case 1A - 1: NSL Cells Covered	7.82	9.70
Case 1A - 2: NSL Cells and Basin Influent Channel Covered	7.71	9.56
Case 2A: NSL Cells and 1st Aerobic Cells Covered	4.44	5.25
Case 3A: NSL, 1st Aerobic and 1st Anoxic Cells Covered	3.31	4.12
Case 4A: NSL, 1st Aerobic and all Anoxic Cells Covered	2.58	3.39
Case 5A: NSL, 1st and 2nd Aerobic and all Anoxic Cells Covered	0.53	0.70
Case 6A: NSL, 1st and 2nd Aerobic Cells Covered	2.40	2.57
Case 7A: NSL, 1st Aerobic and 2nd Anoxic Cells Covered	3.82	4.52
Case 8A: NSL, 1st and 2nd Aerobic and 2nd Anoxic Cells Covered	1.77	1.83
Case 9A: 1st and 2nd Aerobic Cells Covered	4.28	4.45
Case 10A: 1st Aerobic Cell Covered	6.33	7.14
Case 11A: 1st Aerobic and 1st Anoxic Cells Covered	5.20	6.01
Case 12A: 1st Aerobic and all Anoxic Cells Covered	4.46	5.27
Case 13A: 1st and 2nd Aerobic and all Anoxic Cells Covered	2.42	2.59
Case 14A: 1st Aerobic and 2nd Anoxic Cells Covered	5.70	6.40
Case 15A: 1st and 2nd Aerobic and 2nd Anoxic Cells Covered	3.66	3.72
"B" Cases - One zone anoxic		
"B" Total All Zones	9.09	10.96
Case 1B - 1: NSL Cells Covered	7.06	8.93
Case 1B - 2: NSL Cells and Basin Influent Channel Covered	6.94	8.78
Case 2B: NSL Cells and 1st Aerobic Cells Covered	4.38	5.41
Case 3B: NSL, 1st Aerobic and 1st Anoxic Cells Covered	3.25	4.28
Case 4B: NSL, 1st and 2nd Aerobic and 1st Anoxic Cells Covered	1.70	2.24
Case 5B: NSL, 1st and 2nd Aerobic and all Anoxic Cells Covered	0.41	0.54
Case 6B: NSL, 1st and 3rd Aerobic Cells Covered	3.09	3.71
Case 7B: NSL, 1st and 2nd Aerobic Cells Covered	2.83	3.37
Case 8B: NSL, 1st, 2nd and 3rd Aerobic Cells Covered	1.54	1.67
Case 9B: 1st and 3rd Aerobic Cells Covered	4.98	5.59
Case 10B: 1st Aerobic Cell Covered	6.27	7.29
Case 11B: 1st Aerobic and 1st Anoxic Cells Covered	5.14	6.16
Case 12B: 1st Aerobic and all Anoxic Cells Covered	3.59	4.13
Case 13B: 1st, 2nd and 3rd Aerobic and 1st Anoxic Cells Covered	2.30	2.43
Case 14B: 1st and 2nd Aerobic Cells Covered	4.72	5.26
Case 15B: 1st, 2nd and 3rd Aerobic Cells Covered	3.43	3.56

Scenarios “A” and “B” were evaluated for each case. Under the “A” scenarios, two aeration basin zones were anoxic, while under the “B” scenarios, only one zone was anoxic. All calculations were based on the new step feed process arrangement with the third train also in service.

4.0 DISPERSION MODELING

The impact of each zone of the aeration basins and the combined scenarios listed above were evaluated using dispersion modeling. The modeling was performed using the EPA Screen 3 model, Version 5.0. This model allows for the determination of radial transport distances from the odor source at selected meteorological conditions. Model results are reported as radial distances from the modeled unit process, therefore, any point within that distance from the odor source could witness the odor. The Screen 3 model does not use site-specific meteorological data. Regarding theoretical transport distances, wind speed and air stability are key factors. For modeling purposes, the minimum wind speed is considered to be 1 meter/second. At speeds below 1 m/s, the air is considered "calm". A six-stage air stability classification system is typically used, where Class 1 or A is the least stable (most well mixed) and Class 6 or F is the most stable (least mixing). Since odors tend to migrate the farthest during periods of high stability (vertical mixing component) and during periods of low wind speeds (horizontal mixing component), modeling was conducted using a worst case scenario – wind speed of 1 m/s and Class F air stability. During a typical day, these periods typically occur during the evening, nighttime and early mornings, although they can also occur during periods of low barometric pressure.

Input to the model consists of the OERs shown in Tables 4, 5 and 6 above, along with the height and dimensions of each zone. For the revised modeling, only "average" conditions were evaluated. In addition, the OERs from the "A" and "B" cases listed in Table 4 above were compared and only the worst case scenario was evaluated for each case. For instance, Case 1A has a higher OER than Case 1B; therefore, only Case 1A was modeled.

The model results and transport distances were analyzed and an average design model endpoint of a 5 D/T was assumed at the property line. Therefore, all scenarios that transmit an odor greater than 5 D/T beyond the property line were identified for further treatment. **Table 7** summarizes the dispersion modeling results for each individual zone and **Table 8** summarizes the dispersion modeling results for each case considered.

Table 7 – Dispersion Modeling Results for Individual Aeration Basin Zones

	2020 Airflows		2030 Airflows	
Source	Max Conc	Distance to 5 D/T (ft)	Max Conc	Distance to 5 D/T (ft)
Zones 1E, 3A and 4A - A	7.122	951	9.368	1230
Zones 2B, 2C and 3C - B	6.620	869	8.709	1148
Zones 1E, 3A and 4A - B	5.578	673	7.338	968
Zones 1F, 3B and 4B - A	4.447	-	5.850	738
NSL Zones 1C, 1B	3.570	-	3.570	-
Zones 1F, 3B and 4B - B	2.809	-	3.696	-
Zones 2A, 2D and 3D	2.459	-	2.459	-
Zones 2B, 2C and 3C - A	1.600	-	1.600	-
Mixed Liquor Effluent Channel (Zones 5A and 5B) - B	1.774	-	2.333	-
NSL Zones 1D, 1A	0.772	-	0.772	-
NSL Influent Channel	0.523	-	0.523	-
Basin Influent Channel - B	0.283	-	0.372	-
NSL Effluent Channel	0.081	-	0.081	-

Table 8 – Dispersion Modeling Results for Aeration Basin Cases

	2020 Airflows		2030 Airflows	
Case	Max Conc	Distance to 5 D/T (ft)	Max Conc	Distance to 5 D/T (ft)
"A" Total All Zones	13.120	2182	15.620	2477
Case 1A: NSL Cells Covered	10.930	1821	13.540	2133
Case 10A: 1st Aerobic Cell Covered	9.729	1591	-	-
Case 10B: 1st Aerobic Cell Covered	-	-	11.220	1788
Case 14A: 1st Aerobic and 2nd Anoxic Cells Covered	8.696	1444	9.767	1608
Case 11A: 1st Aerobic and 1st Anoxic Cells Covered	9.451	1411	-	-
Case 11B: 1st Aerobic and 1st Anoxic Cells Covered	-	-	11.210	1624
Case 9B: 1st and 3rd Aerobic Cells Covered	9.052	1362	10.180	1509
Case 2B: NSL Cells and 1st Aerobic Cells Covered	-	-	9.214	1427
Case 12A: 1st Aerobic and all Anoxic Cells Covered	9.736	1312	11.540	1476
Case 2A: NSL Cells and 1st Aerobic Cells Covered	7.566	1198	-	-
Case 7A: NSL, 1st Aerobic and 2nd Anoxic Cells Covered	5.926	935	7.017	1148
Case 2B: NSL Cells and 1st Aerobic Cells Covered	-	-	9.165	1247
Case 15A: 1st and 2nd Aerobic and 2nd Anoxic Cells Covered	7.560	1083	7.680	1099
Case 3A: NSL, 1st Aerobic and 1st Anoxic Cells Covered	6.753	968	-	-
Case 6B: NSL, 1st and 3rd Aerobic Cells Covered	4.801	-	5.761	886
Case 4A: NSL, 1st Aerobic and all Anoxic Cells Covered	7.166	853	9.426	1083
Case 13A: 1st and 2nd Aerobic and all Anoxic Cells Covered	5.810	738	6.413	804
Case 8A: NSL, 1st and 2nd Aerobic and 2nd Anoxic Cells Covered	4.800	-	4.956	-
Case 5A: NSL, 1st and 2nd Aerobic and all Anoxic Cells Covered	2.286	-	2.995	-

Where a "—" is shown for the average transport distance, the maximum odor concentration from that scenario was less than 5 D/T; where a "—" is shown for the maximum concentration, that

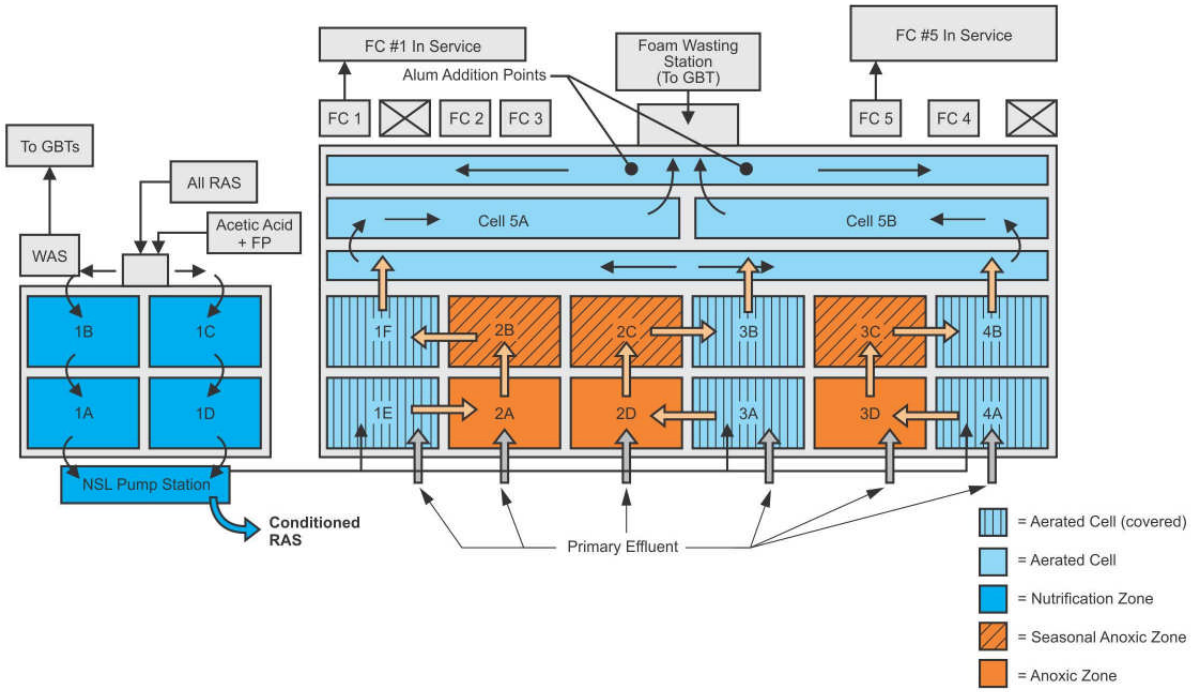
scenario was not modeled. The results of the model can be used to rank the various potential sources based on transport distance.

The NSL basins are located approximately 160 feet from the nearest property line, while the center of the other aeration basins is located approximately 325 feet from the property line. Based on OWASA's goal of meeting a 5 D/T at the property line, any odor sources that have transport distances to 5 D/T greater than the distance to the property line would not comply with this requirement. This goal is challenging due to the close proximity of these odor emission sources to the plant property lines.

5.0 RECOMMENDATIONS

The modeling results indicate that the NSL basins (4 total), select aerobic cells of the aeration basins (6 total) and the aeration influent channel should be covered and odor controlled to meet the goal of 5 D/T at the property line, as indicated in Figure 2. While on-site observations would seem to indicate that the NSL basins may be the greatest source of odors in this general area of the plant, from a dispersion perspective the aeration basins contribute more to detectable odors at the property boundary than the NSL basins. This is due to the aeration mass transfer effect that occurs in the aerobic cells. While the first aerobic cells constitute the greatest source location, the second aerobic cells are the second largest contributor. The step feed process shifts a portion of the primary effluent and NSL effluent to the anoxic cells and the first opportunity for aerobic treatment of this portion of the wastewater is in the second aerobic cells (1F, 3B and 4B) necessitating odor control in these cells as well.

Figure 2 – Recommended Odor Improvements



As wastewater flows approach the plant design capacity, OWASA will have to operate the third cell of each treatment train in an aerobic mode rather than in an anoxic mode seasonally to maintain healthy nitrification during the cold winter months. When using this mode, the odors emitted from the third cells of each train would increase significantly from current levels and these cells would have to be covered and scrubbed to maintain the 5 D/T criteria. It is recommended that covers for this area not be added until flows begin to require aeration of the third cell of each train, likely around 2030. It is recommended, however, that the odor scrubber system and ductwork be sized to accommodate the future air flows.

Hazen and Sawyer has reviewed the odor control technology proposed in the Brown and Caldwell design and concurs that a carbon adsorption scrubber system is the appropriate technology to removal of the types of odors expected in this application in a cost effective manner.

Table 9 summarizes the recommended capture rates for each of the NSL basin and aeration basin zones based on use of the step feed process configuration.

Table 9 – Recommended Capture Rates

Location	Area (ft^2)	Head Space (ft)	Design Criteria	Process Air Flow (ft^3/min)	Leakage Rate (CFM/ft^2)	Capture Rate (ft^3/min)
NSL Influent Channel	400	1	Leakage	0	1	400
NSL Cells	10000	6.16	6 ac/hr	0	0.33	6,160
NSL Pump Station	330	6.16	6 ac/hr	0	0.5	203
Aeration Basins Influent Channel	1925	1.61	Air + Leakage	97	0.66	1,367
Aeration Basin Cells 1E, 3A and 4A	7500	3.08	Air + Leakage	9,158	0.33	11,633
Aeration Basin Cells 2B, 2C and 3C	7500	3.31	Air + Leakage	5,312	0.33	7,787
Aeration Basin Cells 1F, 3B and 4B	7500	3.61	Air + Leakage	7,011	0.33	9,486
Total NSL Cells						**6,763**
Total Aeration Basins (not including NSL Cells)						**30,273**

The recommended capture rates that differ from Brown and Caldwell's original design are highlighted in blue in Table 9 above. A total air flow of 30,000 cfm is recommended for exhausting the covered areas in the aeration basins and 6,700 cfm for the NSL cells. At the aeration basins, the third cells will not require covering until approximately 2030, and these covers can be added in the future when needed in these cells. Sizing the carbon adsorption odor scrubber system in the initial design to address this future condition would however be more cost effective than sizing the initial system smaller and adding another scrubber in the future. OWASA will benefit from operating the larger scrubber with additional carbon as the service life of the carbon media will be greatly extended.

Recommended design criteria for the carbon adsorption system are summarized in **Table 10** below.

Table 10 – Recommended Design Criteria

Parameter	Nutrified Cells Odor Control System	Aerated Cells Odor Control Systems
Type of Tank Covers	Interlocking Aluminum with Trusses	Interlocking Aluminum with Trusses
Type of Carbon Adsorbers	Deep Bed with Hi-Capacity Carbon	Radial Units with Hi-Capacity Carbon
Quantity	1	2
Average H_2S Concentration	5 ppm	5 ppm
Sustained Peak H_2S Concentration	25 ppm	25 ppm
Removal of H_2S (min)	99 percent	99 percent
Removal of non-sulfide VOCs	95 percent	95 percent
Air Flow	6,700 cfm	15,000 cfm
Avg. Face Velocity in Bed	60 fpm	55 fpm
Minimum bed contact time	3.0 seconds	3.0 seconds
Dimensions of Vessels	12 ft diameter, 10 ft high	11 ft diameter, 15 ft high
Depth of Carbon	3 ft	2.5 ft
Minimum H_2S Capacity	0.2 gms/cc	0.2 gms/cc
Carbon Quantity	10,000 lbs	15,750 lbs
Fan Motors	15 HP	40 HP*

* Consider two-speed motor or variable frequency drive to reduce energy costs in the period prior to providing covers and odor control for the third aerated cells.

The proposed odor control system improvements would consist of one 6,700 cfm carbon adsorption unit drawing air from the NSL basins and two 15,000 cfm carbon adsorption units drawing air from the aeration basins. It is anticipated that these improvements would be constructed in conjunction with aeration and mixing improvements. The estimated cost of the odor control system is approximately $2.7 million including engineering, which is approximately $500,000 higher than the amount estimated by Brown and Caldwell in their 95% Design cost

estimate dated 12/21/2009. A total of ten treatment cells (4 NSL and 6 aerated) and the aeration influent channel will be covered in lieu of eight cells (4 NSL and 4 aerated) as proposed in the original Brown and Caldwell design. The aeration tanks were designed with considerable operational flexibility. Because the aerobic cells will be covered and odor scrubbed to meet the odor control objectives for this facility, there will be a reduction in operational flexibility as compared to the existing facilities without odor control. These design changes have been reviewed with Brown and Caldwell and it is our understanding that they are in concurrence with the proposed design modifications to the Phase 3 odor control improvements. It is also of note that the 2007 odor study by Black and Veatch also recommended to cover the NSL basins and the first aerated zones of the aeration basins.

PREPARED FOR: Orange Water and Sewer Authority

FROM: Hazen and Sawyer

PREPARED BY: Sarah Lothman, Joe Rohrbacher, Ron Taylor

DATE: November 8, 2010
Revised November 19, 2010

SUBJECT: Mason Farm Wastewater Treatment Plant
Hydraulic and Treatment Capacity Study
Technical Memorandum – Nutrient Analyzers

The installation of nutrient analyzers at the Mason Farm WWTP was recommended as part of the Hydraulic and Treatment Capacity Study. Nutrient analyzers are a reliable and relatively affordable way to provide a continuous, online analysis of nutrient concentrations in wastewater. At the Mason Farm WWTP, where strict nutrient limits are forthcoming, uninterrupted monitoring of ammonia, pH, phosphorus, and nitrogen can provide advance warning of process issues that can be addressed before there is a permit violation and provide for precise process control within the biological treatment process. A continuous stream of data from the analyzers would allow operators to adjust air flows, chemical feeds, and seasonal variation in the process to optimize the treatment process. Figure 1 shows the proposed locations for nutrient analyzers. The individual analyzers and locations are briefly described below:

- A nitrate analyzer should be provided in the aeration basin effluent channel 2 as shown on Figure 1 and will determine nitrate loading at this location.
- An orthophosphate analyzer is recommended at the end of Cells 5A and 5B in the basin. One analyzer can be installed at this location with the ability to collect samples from each cell. Installation of an analyzer at this location will help plant staff to check for process upsets and provide confirmation that phosphorus uptake is occurring. Phosphate concentration at this point in the process can be used to automate the dosing of alum to the mixed liquor just downstream of the probe.
- An ammonium probe is recommended at the end of the final aeration zone. Measurements recorded by this instrument will provide indication of the nitrification performance of the process; it will also provide advance warning of process issues upstream of the plant effluent monitoring

locations allowing operators to make airflow and/or swing cell configuration adjustments to maximum denitrification while maintaining a healthy nitrification process.

- One pH probe is recommended at the aeration basin effluent channel 2. Measurements recorded by this instrument can be used to control caustic feed to the process to provide supplemental alkalinity for nitrification.

There are two basic styles of nutrient monitoring instruments. One style uses a centralized analyzer to which samples are pumped, and is primarily the product of one manufacturer (Chemscan). The other style uses individual analyzers that may be either distributed probes placed directly in the process or wet chemistry analyzers mounted out of the process flow to which short sample lines are routed. The Hach Company and Endress + Hauser manufacture lines of these units. The Mason Farm WWTP has experience successfully operating the Hach Phosphax wet chemistry analyzer. Hazen and Sawyer has good experience operating the Hach Nitratax 1 mm analyzer in aerated, high solids environments. Each style of instrument has advantages and disadvantages as shown in Table 1 below. It is recognized that the Mason Farm WWTP will be implementing odor control strategies within the aeration basins in the future. For all instruments, minimal provisions will be required in the design of odor control hoods in order to provide access to the submerged instruments. Manufacturer information and cut sheets are included in the Appendix.

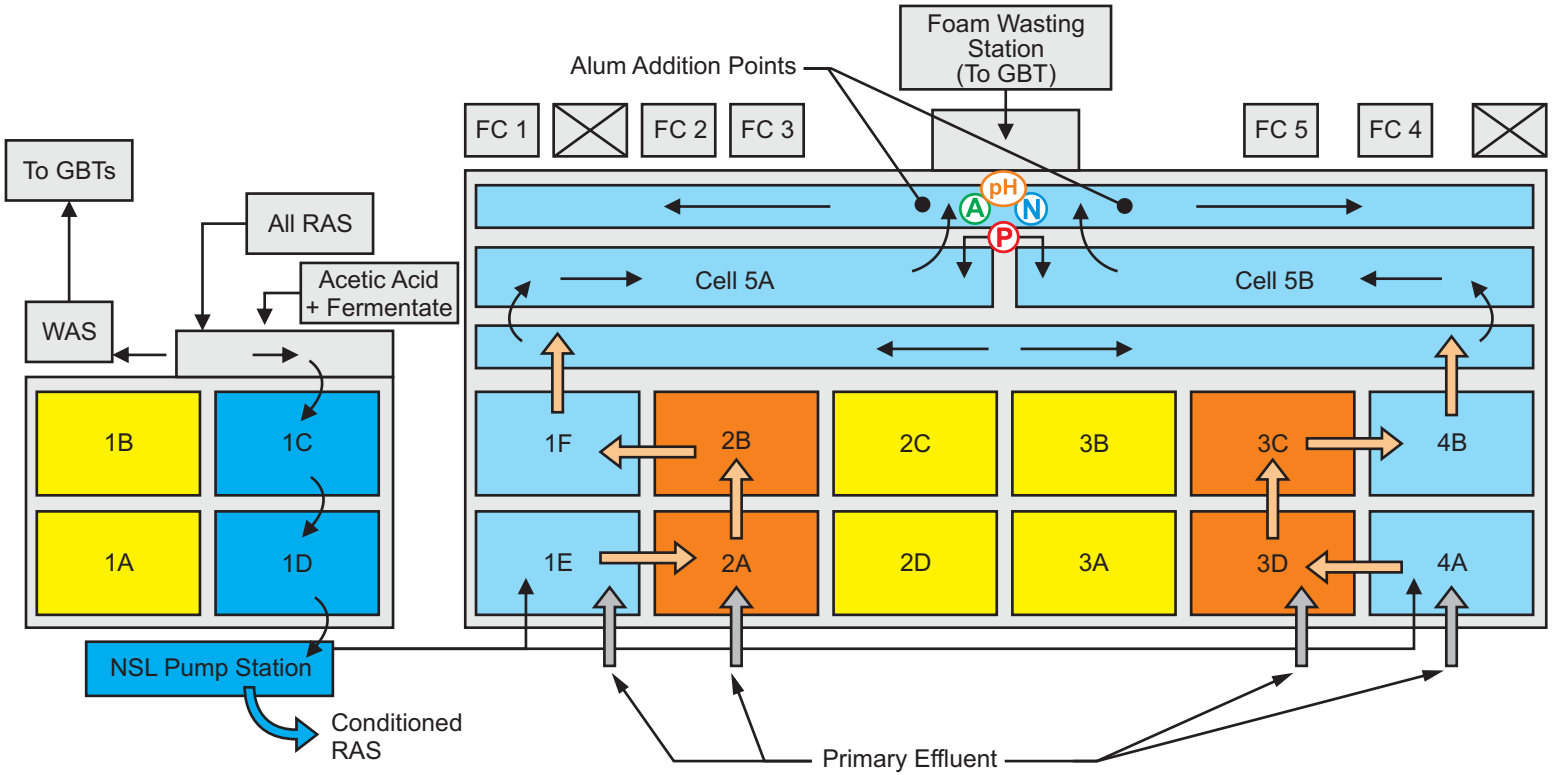
Table 1

Advantages and Disadvantages of Nutrient Analyzer Styles

	Centralized System	Distributed System
O&M Requirements	Fewer instruments to operate and maintain	Many instruments to operate and maintain
Detection Methods Used	Wet chemical systems are accurate and reliable, but require periodic replenishment of reagents.	Ammonia and orthophosphate analyzers use wet chemical systems while nitrite, nitrate, and ammonium use optical probes
Sampling Complexity	Complicated sample delivery systems consisting of pumps, pipes, and valves.	Simple or non-existent sample delivery systems.
Relative Cost	More expensive on a per unit basis, however, numerous sampling locations and parameters can bring considerable economies of scale.	Less expensive on a per unit basis, but installing many units can become expensive.
Fault Tolerance	Failure of central unit eliminates all monitoring.	Failure of a single unit is not as critical.
Response Time	Response time can be long due to sample delivery time plus analysis time.	Response time is not dependent on long sample runs. In-situ probes response is nearly instantaneous.
Calibration Requirements	Wet chemical devices can employ periodic standards testing to ensure testing.	In-situ probes require individual, albeit simple, calibration.
Weather Protection	Allows consolidation of equipment into a single structure protected from the elements.	Requires individualized weather protection, although this is simplified for in-situ probes. All instruments can be mounted in a central structure but this may become cost prohibitive.

The total capital cost of the proposed instruments is based on installation of probes and local wet chemistry instruments and is estimated at $59,600. After the proposed cold weather and biological phosphorus removal optimization is completed, OWASA should evaluate whether additional on-line nutrient analyzers could be used to improve process control and performance for the biological phosphorus removal process. It is important to note that these costs do not include installation or integration costs; installation and integration could cost up to approximately 30% of the capital cost, or $17,900. This results in an estimated total installed cost of $77,500 for the recommended instrumentation.

Figure 1



Legend

- = Aerated Cell
- = Nutrification Zone
- = Anoxic Zone
- = Out of Service
- (P) = Ortho-Phosphate Analyzer
- (A) = Ammonium Analyzer
- (N) = Nitrate Analyzer
- (pH) = pH Analyzer

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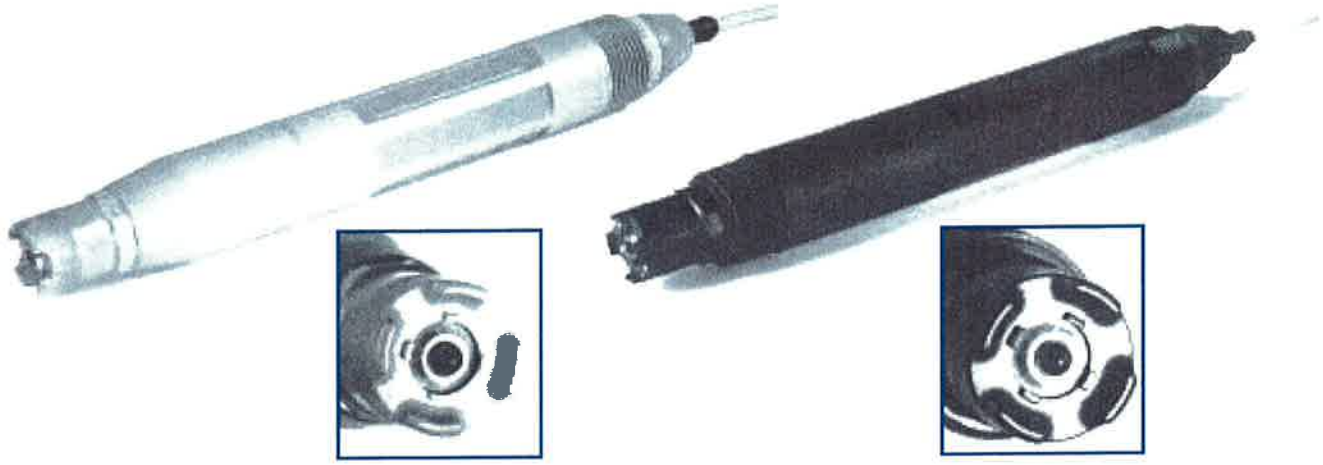
HAZEN AND SAWYER
Environmental Engineers & Scientists

MASON FARM WWTP

NUTRIENT ANALYZER LOCATIONS

APPENDIX

Differential pH and ORP Sensors



Hach Digital pHD sc sensors are available in convertible (PEEK® or Ryton®), insertion, and sanitary body styles. Three electrodes are used in these sensors to increase measurement accuracy and eliminate sensor ground loops.

Features and Benefits

Differential Electrode Measurement Technique

This field-proven technique uses three electrodes instead of the two normally used in conventional pH sensors. Process and reference electrodes measure the pH differentially with respect to a third ground electrode. The end result is unsurpassed measurement accuracy, reduced reference junction potential, and elimination of sensor ground loops. These sensors provide greater reliability, resulting in less downtime and maintenance.

Patented Technology

The former GLI, now a Hach Company brand, invented the Differential Electrode Technique for pH measurement in 1970. The pHD™ sensor series (U.S. Patent Number 6395158B1, dated May 28, 2002) takes this field-proven technology to a new level.

Replaceable Salt Bridge/Protector

The unique, replaceable salt bridge holds an extraordinary volume of buffer to extend the working life of the sensor by protecting the reference electrode from harsh process conditions. The salt bridge simply threads onto the end of the sensor if replacement is needed.

Built-in Encapsulated Preamp

Encapsulated construction protects the sensor's built-in preamp from moisture and humidity, ensuring reliable sensor operation. The preamp in the pHD analog sensor produces a strong signal, enabling the sensor to be located up to 1000 m (3280 ft.) from the analyzer.

Durable Body Materials

Both the digital and analog pH and ORP differential sensors feature a durable PEEK® body for chemical compatibility with most process solutions. For less aggressive solutions, Hach offers a Ryton® sensor in a convertible style for pH and ORP measurement. A sensor with a stainless steel body is available for immersion applications.

Digital Electronics Modules

Sensors are available with integral digital electronics or with a gateway module for high temperature (above 70°C) applications.

Versatile Mounting Styles

Sensors are available in four mounting styles—convertible, insertion, immersion, and sanitary. Please turn to page 5 for more information.

Full Featured "Plug and Play" sc100 Digital Controller

There's no complicated wiring or set up procedures with the Hach sc100 controller. Just plug in any Hach digital sensor and it's ready to use—it's "plug and play."

One or two sensors—Use the sc100 Digital Controller to receive data from up to two Hach digital sensors in any combination.

Communications—Multiple alarm/control schemes are available using three relays and two PID control outputs. Communications use analog 4-20 mA and digital MODBUS®/RS485, MODBUS®/RS232 protocols. (Other digital protocols are available. Contact your Hach representative for details.) Every sc100 controller is equipped with wireless communication through an infrared port.

Data logger—A built-in data logger collects measurement data, calibration, verification points, and alarm history for up to 6 months.

DW WW PW IW

Specifications*

pH Sensors

Most pH applications fall in the 2.5 to12.5 pH range. A Hach pHD sc Differential pH sensor with the wide-range glass process electrode performs exceptionally well in this range. Some industrial applications require accurate measurement and control below 2 or above 12 pH. In these special cases, please contact Hach Technical Support for further details.

Measuring Range
-2 to 14 pH

Sensitivity
± 0.01 pH

Stability
0.03 pH per 24 hours, non-cumulative

Operating Temperature
Digital Sensor: -5 to 70°C (23 to 158°F)
Analog Sensor with Digital Gateway: -5 to 105°C (23 to 221°F)
Immersion Sensor: 0 to 50°C (32 to 122°F)

Flow Rate
3 m (10 ft.) per second, maximum

Sensor Pressure/Temperature Limits
Digital: 6.9 bar at 70°C (100 psi at 158°F)
Analog: 6.9 bar at 105°C (100 psi at 221°F)

Built-in Temperature Element
NTC 300 ohm thermistor for automatic temperature compensation and analyzer temperature readout

Transmission Distance
100 m (328 ft.), maximum
1000 m (3280 ft.), maximum when used with a termination box

Sensor Cable (integral)
4 conductor cable with one shield and polyurethane jacket; rated to 105°C (221°F); 10 m (33 ft.) standard length

Wetted Materials
PEEK® or Ryton® (PVDF), salt bridge of matching material with Kynar® junction, glass process electrode, titanium ground electrode, and Viton® O-ring seals

(pH sensor with optional HF-resistant glass process electrode has 316 stainless steel ground electrode, and perfluoroelastomer wetted O-rings; consult factory for other available wetted O-ring materials)

ORP (Redox) Sensors

For best ORP measuring results in solutions containing zinc, cyanide, cadmium or nickel, Hach recommends using the pHD sc ORP sensor equipped with an optional gold electrode.

Measuring Range
-1500 to +1500 mV

Sensitivity
± 0.5 mV

Stability
2 mV per 24 hours, non-cumulative

Operating Temperature
Digital Sensor: -5 to 70°C (23 to 158°F)
Analog Sensor with Digital Gateway: -5 to 105°C (23 to 221°F)
Immersion Sensor: 0 to 50°C (32 to 122°F)

Flow Rate
3 m (10 ft.) per second, maximum

Sensor Pressure/Temperature Limits
Digital: 6.9 bar at 70°C (100 psi at 158°F)
Analog: 6.9 bar at 105°C (100 psi at 221°F)

Built-in Temperature Element
NTC 300 ohm thermistor for analyzer temperature readout only—no automatic temperature compensation necessary for ORP measurement

Transmission Distance
100 m (328 ft.), maximum
1000 m (3280 ft.), maximum when used with a termination box

Sensor Cable (integral)
4 conductor cable with one shield and polyurethane jacket; rated to 105°C (221°F); 10 m (33 ft.) standard length

Wetted Materials
PEEK® or Ryton® (PVDF), salt bridge of matching material with Kynar® junction, glass and platinum (or plastic and gold) process electrode, titanium ground electrode, and Viton® O-ring seals

**Specifications subject to change without notice.*

PEEK® is a registered trademark of ICI Americas, Inc.; Ryton® is a registered trademark of Phillips 66 Co.;
Kynar® is a registered trademark of Pennwalt Corp.; Viton® is a registered trademark of E.I. DuPont de Nemours + Co.

Engineering Specifications

PEEK® Sensor

1. The pH or ORP sensor shall be of Differential Electrode Technique design using two measuring electrodes to compare the process value to a stable internal reference standard buffer solution. The standard electrode shall have non-flowing and fouling-resistant characteristics.
2. The sensor shall have a hex-shaped body to facilitate mounting, and shall be constructed of PEEK® material for exceptional chemical resistance and mechanical strength. This material shall enable the sensor to be installed in metal fittings without leakage usually caused by heating and cooling cycles when dissimilar materials are threaded together.
3. The sensor shall have a:
   a) Convertible body style featuring 1-inch NPT threads on both ends to mount into a standard 1-inch pipe tee, into a Hach adapter pipe for union mounting with a standard 1-1/2 inch tee, or onto the end of a pipe for immersion into a vessel.
   b) Insertion body style featuring 1-inch NPT threads only on the cable end to mount into a Hach ball valve hardware assembly, enabling the sensor to be inserted into or retracted from the process without stopping the process flow.
   c) Sanitary body style featuring an integral 2-inch flange to mount into a Hach 2-inch sanitary tee. The sanitary body style sensor shall include a special cap and EDPM compound gasket for use with the Hach sanitary hardware.
4. The built-in electronics of the sensor shall be completely encapsulated for protection from moisture and humidity.
5. The sensor shall have a built-in preamplifier to enable the signal to be transmitted up to 100 m (328 ft.) with standard cabling and up to 1000 m (3280 ft.) with a termination box.
6. The sensor signal shall have an integral temperature sensor. The pH sensor shall automatically compensate measured values for changes in process temperature.
7. The ORP sensor shall include a titanium ground electrode (standard) to eliminate ground loop currents in the measuring electrodes.
8. The sensor shall be Hach Company Model pHD sc or pHD for pH or ORP measurement.

Ryton® Sensor

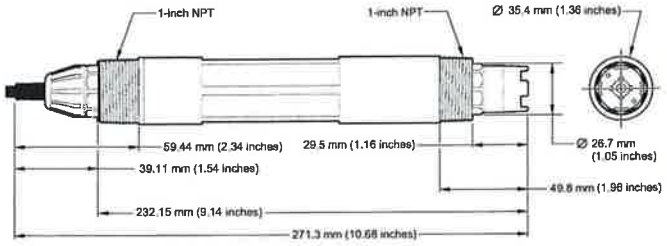
1. The pH or ORP sensor shall be of Differential Electrode Technique design using two measuring electrodes to compare the process value to a stable internal reference standard buffer solution. The standard electrode shall have non-flowing and fouling-resistant characteristics.
2. The sensor shall have a hex-shaped body to facilitate mounting, and shall be constructed of Ryton® material for exceptional chemical resistance and mechanical strength. This material shall enable the sensor to be installed in metal fittings without leakage usually caused by heating and cooling cycles when dissimilar materials are threaded together.
3. The sensor shall have a convertible body style featuring 1-inch NPT threads on both ends to mount into a standard 1-inch pipe tee, into a Hach adapter pipe for union mounting with a standard 1-1/2 inch tee, or onto the end of a pipe for immersion into a vessel.
4. The built-in electronics of the sensor shall be completely encapsulated for protection from moisture and humidity.
5. The sensor shall have a built-in preamplifier to enable the signal to be transmitted up to 100 m (328 ft.) with standard cabling and up to 1000 m (3280 ft.) with a termination box.
6. The sensor signal shall have an integral temperature sensor. The pH sensor shall automatically compensate measured values for changes in process temperature.
7. The ORP sensor shall include a titanium ground electrode (standard) to eliminate ground loop currents in the measuring electrodes.
8. The sensor shall be Hach Company Model pHD sc or pHD for pH or ORP measurement.

Stainless Steel Sensor

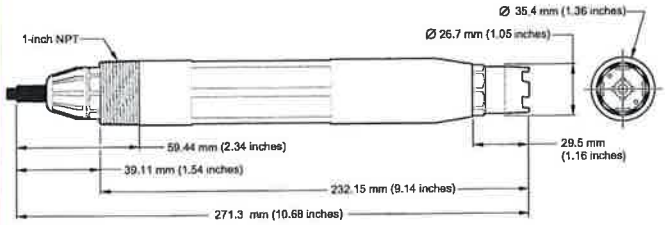
1. The pH or ORP sensor shall be of differential electrode technique design using two measuring electrodes to compare the process value to a stable internal reference standard buffer solution. The standard electrode shall have non-flowing and fouling-resistant characteristics.
2. The sensor shall be capable of chain mounting for immersion applications, and shall be constructed of 316 stainless steel.
3. The built-in electronics of the sensor shall be completely encapsulated for protection from moisture and humidity.
4. The sensor shall have a built-in preamplifier to enable the signal to be transmitted up to 100 m (328 ft.) with standard cabling and up to 1000 m (3280 ft.) with a termination box.
5. The sensor signal shall have an integral temperature sensor to automatically compensate measured values for changes in process temperature.
6. The sensor shall include a titanium ground electrode (standard) to eliminate ground loop currents in the measuring electrodes.
7. The sensor shall be Hach Company Model pHD sc or pHD for pH or ORP measurement.

Dimensions

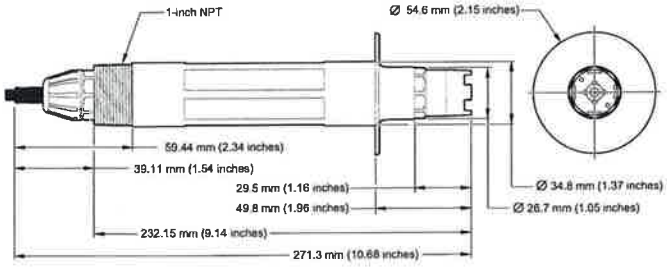
Convertible Style



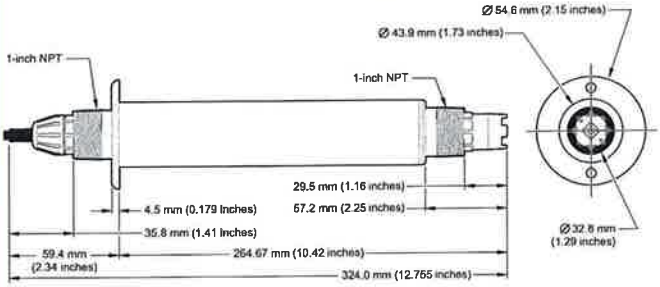
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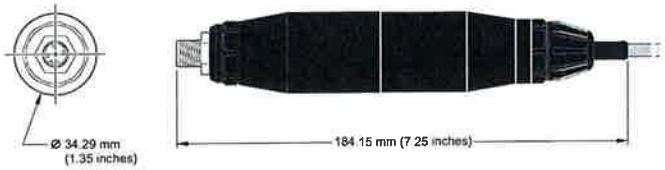
Sanitary Style



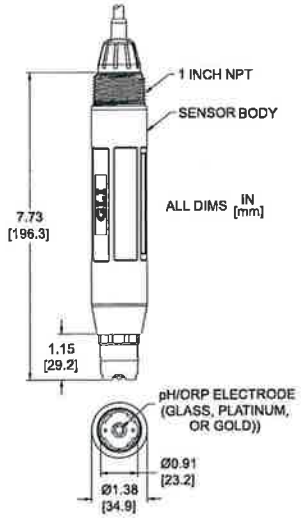
Immersion Style



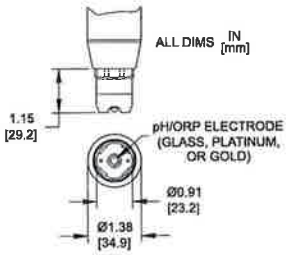
Digital Gateway



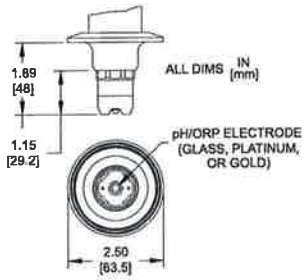
Analog Convertible Style



Analog Insertion Style



Analog Sanitary Style



Ordering Information

pHD sc Digital Differential pH/ORP Sensors

All digital sensors include built-in digital electronics and integral 10 m (33 ft.) cable terminated with connector for the sc100 digital controller. Body styles:

- *Convertible – 1-inch NPT threads at both ends, designed for tee-mounting or other flow through mountings, and pipe mounting for immersion*
- *Insertion – no threads on the electrode end, designed for use with insertion valve assembly*
- *Sanitary – 2-inch flange for a tri-clover style fitting*
- *Immersion – used with chain mounting or pipe mounting*

pH Sensors

Product Number	Body Material	Body Style	Electrode Material	Max Temp
DPD1P1	PEEK[1]	Convertible	Glass, General Purpose	70°C (158°F)
DPD1P3	PEEK[1]	Convertible	Glass, HF-resistant	70°C (158°F)
DPD2P1	PEEK[1]	Insertion	Glass, General Purpose	70°C (158°F)
DPD3P1	PEEK[1]	Sanitary	Glass, General Purpose	70°C (158°F)
DPD1R1	Ryton[2]	Convertible	Glass, General Purpose	70°C (158°F)
DPD1R3	Ryton[2]	Convertible	Glass, HF-resistant	70°C (158°F)
DPS1	Stainless Steel	Immersion	Glass, General Purpose	50°C (122°F)

[1]Polyetheretherketone [2]Polyphenelene Sulfide

ORP Sensors

Product Number	Body Material	Body Style	Electrode Material	Max Temp
DRD1P5	PEEK[1]	Convertible	Platinum	70°C (158°F)
DRD1P6	PEEK[1]	Convertible	Gold	70°C (158°F)
DRD2P5	PEEK[1]	Insertion	Platinum	70°C (158°F)
DRD1R5	Ryton[2]	Convertible	Platinum	70°C (158°F)
DRD1R6	Ryton[2]	Convertible	Gold	70°C (158°F)
DRS5	Stainless Steel	Immersion	Platinum	50°C (122°F)

[1]Polyetheretherketone [2]Polyphenelene Sulfide

Digital Gateway

6120500 Digital Gateway, convert pHD analog sensors to digital output for connecting to sc100 digital controller

pHD Analog Sensors

All analog sensors include built-in preamplifier and integral 4.5 m (15 ft.) cable terminated with stripped and tinned wires.

Definitions of body styles:

- *Convertible – 1-inch NPT threads at both ends, designed for tee-mounting or other flow through mountings, and pipe mounting for immersion*
- *Insertion – has no threads on the electrode end, designed for use with insertion valve assembly*
- *Sanitary – has a 2-inch flange for a Tri-Clover style fitting*

pH Sensors

Product Number	Body Material	Body Style	Electrode Material	Max Temp
PD1P1	PEEK[1]	Convertible	Glass, General Purpose	95°C (203°F)
PD1P3	PEEK[1]	Convertible	Glass, HF-resistant	95°C (203°F)
PD2P1	PEEK[1]	Insertion	Glass, General Purpose	95°C (203°F)
PD3P1	PEEK[1]	Sanitary	Glass, General Purpose	95°C (203°F)
PD1R1	Ryton[2]	Convertible	Glass, General Purpose	95°C (203°F)
PD1R3	Ryton[2]	Convertible	Glass, HF-resistant	95°C (203°F)

[1]Polyetheretherketone [2]Polyphenelene Sulfide

ORP Sensors

Product Number	Body Material	Body Style	Electrode Material	Max Temp
RD1P5	PEEK[1]	Convertible	Platinum	95°C (203°F)
RD1P6	PEEK[1]	Convertible	Gold	95°C (203°F)
RD2P5	PEEK[1]	Insertion	Platinum	95°C (203°F)
RD1R5	Ryton[2]	Convertible	Platinum	95°C (203°F)
RD1R6	Ryton[2]	Convertible	Gold	95°C (203°F)

[1]Polyetheretherketone [2]Polyphenelene Sulfide

Ordering Information continued

pHD sc Digital and pHD Analog Sensor Accessories

Cables

Extension cables are used only with digital sensors or digital gateways when connecting to the sc100 Digital Controller.

6122400 Digital Extension Cable, 1 m (3.2 ft.)

5796000 Digital Extension Cable, 7.7 m (25 ft.)

5796100 Digital Extension Cable, 15 m (50 ft.)

5796200 Digital Extension Cable, 31 m (100 ft.)

Interconnect cables are used only with analog sensors, junction box, and controller.

1W1100 Analog Interconnect Cable, order per foot

Digital Termination Box

Required when the length of cable between the digital sensor/digital gateway and sc100 Digital Controller is between 100 m (328 ft.) and 1000 m (3280 ft.)

5867000 Digital Termination Box

Analog Junction Box

Required when the length of cable between the analog sensor and analog controller is greater than standard length of sensor cable. Each junction box includes terminal strip and gasket.

60A2053 Junction Box, Surface-mount, aluminum (includes mounting hardware)

60A9944 Junction Box, Pipe-mount, PVC (for 1/2-inch diameter pipe, includes mounting hardware)

60G2052 Junction Box, Pipe-mount, PVC (for 1-inch diameter pipe, includes mounting hardware)

76A4010-001 Junction Box, NEMA 4X (no mounting hardware included)

Protector for Convertible style sensor

1000F3374-002 PEEK protector

1000F3374-003 Ryton protector

Salt Bridges

The double junction salt bridge on the standard cell of all Hach pHD sensors is field-replaceable. Each salt bridge has a ceramic inner junction, Viton® O-ring, and contains binary, equi-transferrant fill solution. Salt bridges are shipped in a salt solution.

Product Number	*pHD sc and pHD Sensor Body Material*	*Salt Bridge Materials*	
		Body	*Outer Junction*
SB-P1SV	PEEK	PEEK	Kynar (PVDF)
SB-P2SV	PEEK	PEEK	Ceramic
SB-P1SP[1]	PEEK	PEEK	Kynar (PVDF)
SB-R1SV	Ryton	Ryton	Kynar (PVDF)

[1]*Special perfluoroelastomer O-ring in place of the Viton® O-ring*

Cleaning Systems for pHD sc and pHD Sensors

Self-Contained Air Blast Cleaning System
Includes Kynar® (PVDF) washer head with 7.6 m (25 ft.) tubing for air delivery, a quick-disconnect tube fitting, and a compressor housed in a NEMA 4X enclosure.

1000A3335-005 For 115 Vac operation
1000A3335-006 For 230 Vac operation

Air/Water Blast Cleaning Washer Head
Intended only for immersion applications with a user-supplied air or water wash system.

1000A3335-004 Kynar (PVDF) washer head includes 1/4-inch barb fitting

pHD sc Digital and pHD Analog Sensor Reagents and Standards

25M1A1025-115 Standard Cell Solution, to replenish standard cell chamber in Hach pHD sensors while replacing salt bridge, 500 mL

25M8A1002-101 Gel Powder, for high temperature applications, 2 g

pH Buffers

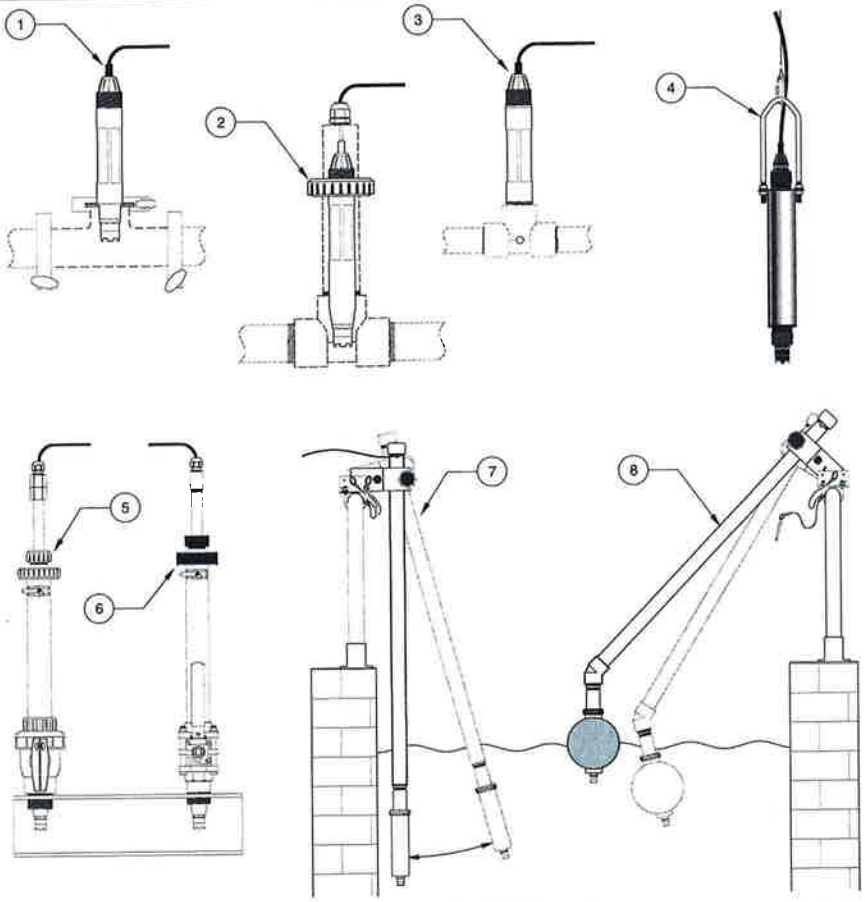
Product Number	*Description*	*Volume*
2283549	pH 7	500 mL (1 pint)
2283449	pH 4	500 mL (1 pint)
2283649	pH 10	500 mL (1 pint)

ORP Reference Solutions (in resealable plastic bottles)

Product Number	*Description*	*Volume*
25M2A1001-115	200 mV	500 mL (1 pint)
25M2A1002-115	600 mV	500 mL (1 pint)

Ordering Information *continued*

Mounting Hardware for pHD sc Differential Sensors



1. Sanitary mount
2. Union mount
3. Flow-through mount
4. Hanging stainless steel sensor with the bail
5. PVC Insertion mount
6. Stainless steel insertion mount
7. Immersion mount
8. Immersion mount, ball float

Sanitary Mount

MH018S8SZ 316 SS
Includes 2-inch sanitary tee and heavy-duty clamp. Special cap and EPDM compound gasket are supplied with sensor but can be separately ordered.

Union Mount

6131300 CPVC
6131400 316 SS
Includes standard 1-1/2 inch tee, special union pipe with adapter, sealing hub, and lock ring in respective material, and Viton® O-ring.

Flow-through Mount

MH334N4NZ CPVC
MH314N4MZ 316 SS
Includes a standard 1-inch tee in respective material.

Insertion Mount

Digital		*Analog*	
6136700	CPVC	**MH736M4MZ**	CPVC
6136800	316 SS	**MH716M4MZ**	316 SS

Includes a 1-1/2 inch ball valve in respective material, 1-1/2 inch NPT close nipple, sensor adapter with two Viton® O-rings and wiper, extension pipe, pipe adapter, back tube, and lock ring.

Immersion Mount

Standard Hardware

Digital		*Analog*	
6136400	CPVC	**MH434A00B**	CPVC
6136500	316 SS	**MH414A00B**	316 SS

Includes 1-inch diameter by 4 ft. long pipe and 1-inch x 1-inch NPT coupling in respective material. (Pipe-mount junction box with terminal strip included in analog hardware.)

Handrail Hardware

MH236B00Z CPVC
Includes 1-1/2 inch diameter by 7.5 ft. long CPVC pipe, and a unique swivel/pivot/ pipe clamp assembly.

Chain Mount Hardware

2881900 316 ss
Includes stainless steel bail, nuts, and washers. Does not include chain. To be used with stainless steel immersion sensor only.

NOTE
Contact Hach Technical Support or your Hach representative for information about retro fit hardware for existing installations.

To complete your pH and ORP measurement system, choose the sc100 or the sc1000 controller...

Model sc100 Controller

(see Lit. #2463)



LXV401.52.00002 sc100 Controller Standard
LXV401.52.01002 sc100 Controller with RS-232 MODBUS®
LXV401.52.02002 sc100 Controller with RS-485 MODBUS®

Model sc1000 Controller

(see Lit. #2403)



LXV402.99.00002 sc1000 Display Module
LXV400.99.1R572 sc1000 Probe Module, 4 sensors, 4 mA Out, 4 mA In, 4 Relays, 110-230V
LXV400.99.1B572 sc1000 Probe Module, 4 sensors, 4 mA Out, 4 mA In, 4 Relays, RS-485 (MODBUS), 110-230V
LXV400.99.1F572 sc1000 Probe Module, 4 sensors, 4 mA Out, 4 mA In, 4 Relays, PROFIBUS DP, 110-230V
LXV400.99.1R582 sc1000 Probe Module, 6 sensors, 4 mA Out, 4 mA In, 4 Relays, 110-230V

At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

For current price information, technical support, and ordering assistance, contact the Hach office or distributor serving your area.

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www.hach.com

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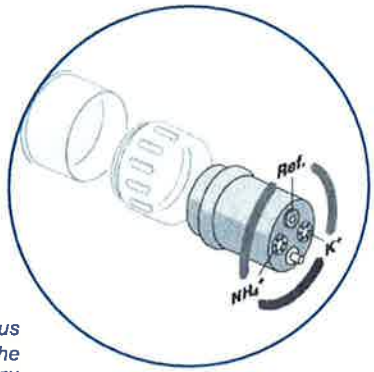
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In the interest of improving and updating its equipment, Hach Company reserves the right to alter specifications to equipment at any time.

HACH®

Be Right™

NH4D sc Ammonium Sensor with CARTRICAL™ Technology





The Hach NH4D sc Ammonium Sensor provides continuous trending of ammonium levels in the aeration basin to control the nitrification process. Exclusively from Hach, CARTRICAL™ technology reduces interferences providing greater accuracy.

Features and Benefits

A New Approach to ISE Technology—CARTRICAL™

The Hach NH4D sc Ammonium Sensor uses an ion-selective electrode (ISE) to detect ammonium ions (NH_4^+) directly in the aeration basin as ammonium nitrogen (NH_4-N). A pHD (differential pH) electrode is used as the reference electrode for superior stability. The most significant potential interference is from potassium ions (K^+) which is compensated through the use of an integrated potassium ISE to correct the ammonium value. Potential interferences are further reduced using CARTRICAL technology. CARTRICAL calibrates each electrode individually and calibrates all three electrodes to each other. A temperature sensor is also included to improve accuracy.

Insitu—No Sample Preparation

The NH4D sc sensor is designed for immersion directly in the aeration basin eliminating the need for sample preparation. It installs quickly outdoors without special housing. The sensor can easily be relocated to various locations for greater knowledge about the process.

Compact Cartridge Simplifies Maintenance

The NH4D sc sensor uses a calibrated sensor cartridge that should be replaced approximately twice a year (in typical operating conditions). No electrolyte or reagents to change. Cartridge replacement only takes minutes—just remove the old cartridge, plug in the new, and enter the sensor code. Maintenance is also minimized because sample pumps, valves, tubing, or filtration devices are not needed.

Economical Continuous Measurement

Real time measurements of ammonium in the aeration basin can be used to more effectively control the nitrification process. The low purchase price makes the NH4D sc the perfect tool for providing more visibility into daily trends to enable optimization of nutrient removal.

Optional Automatic Cleaning System

A specially-designed air cleaning unit is available for the NH4D sc sensor. Air is directed at each of the three individual electrodes for optimal cleaning. A compact compressor is available to provide pressurized air for the cleaning unit with weather-proof housing suitable for tank-side installation without additional structural requirements.

Plug and Play with Hach sc Digital Controllers

The Hach NH4D sc Ammonium Sensor is "plug and play" ready to use with the Hach sc100 or sc1000 Controllers. Complicated wiring or set-up procedures are not necessary. The sc100 controller can accept up to two sensors while the sc1000 controller can accept up to eight sensors.

Applications

- *Municipal wastewater treatment plants with less than 30% industrial waste*
- *Aeration basin (profile multiple trains)*
- *Control feeding of centrate/filtrate back into the process*

Specifications*

Measurement Method
Ion-selective electrodes for ammonium and potassium with pHD reference electrode and temperature sensor

Range
0.2 to 1000 mg/L NH_4-N

Accuracy
5% of measured value ±0.2 mg/L (with standard solution)

Detection Limit
0.2 mg/L

Response Time
Less than 2 minutes (T90)

Operating Temperature
-20 to 45°C (-4 to 113°F)

Sample Temperature
0 to 40°C (32 to 104°F)

Sample pH
5 to 9

Sensor Immersion Depth
0.3 to 3.0 m (1 to 10 ft.) maximum

Sample Pressure
0.3 bar (4.4 psi) maximum

Storage Temperature
Sensor: -20 to 60°C (-4 to 140°F)

Sensor Cartridge: 5 to 40°C (41 to 104°F)

Calibration
Sensor cartridge, calibrated (sensor code contains factory calibration in code form)

Entry of sensor code calibrates the sensor

No standard solutions needed

Optional: 1- or 2-point inline matrix correction adapts sensor to the wastewater matrix

Certifications
CE approved

Sensor Construction
316 stainless steel with Ryton® ends

Dimensions
48 x 361 mm (1.9 x 14.2 in.)

Cable Length
Standard: 10 m (33 ft.)

Optional Extensions: 7.6 m (25 ft.), 15.2 m (50 ft), or 30.5 m (100 ft.)

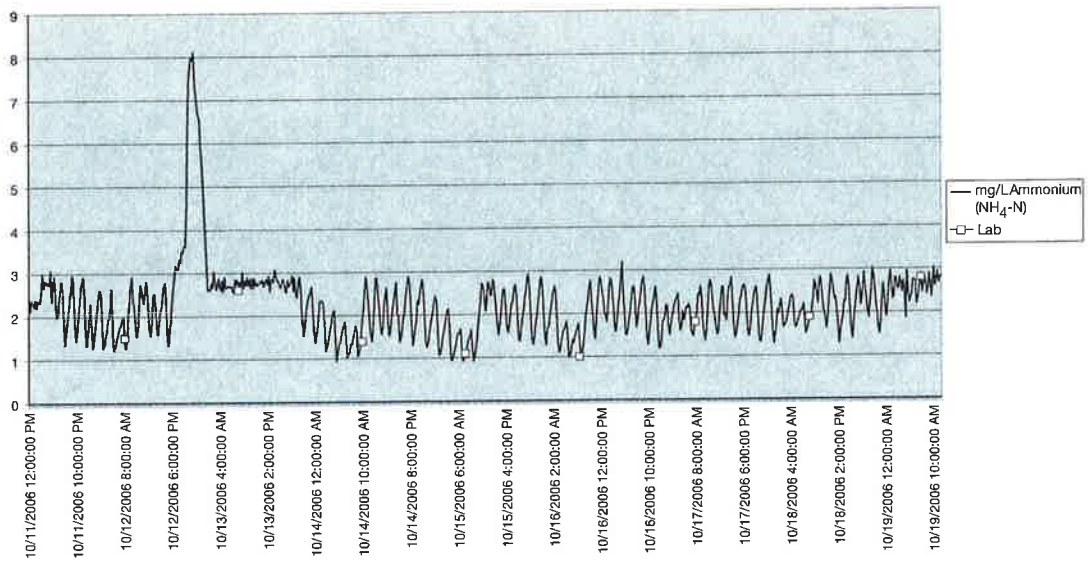
Maximum Total Length: 100 m (328 ft.)

Weight
1.3 kg (2.9 lbs.)

Warranty
2 years

Ryton® is a trademark of Phillips Petroleum Company.

*Specifications subject to change without notice.

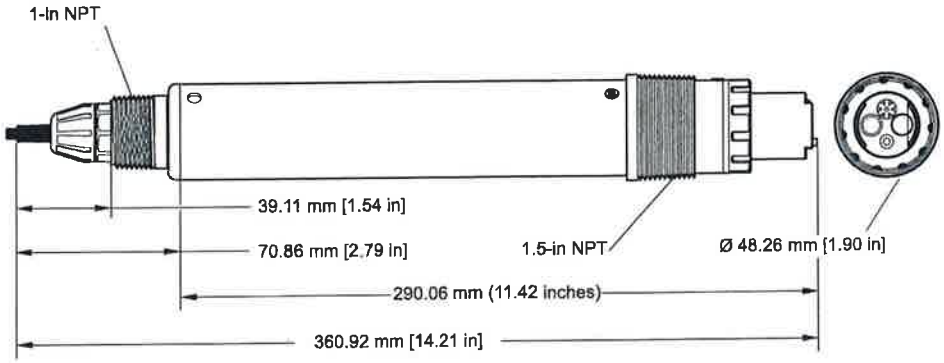


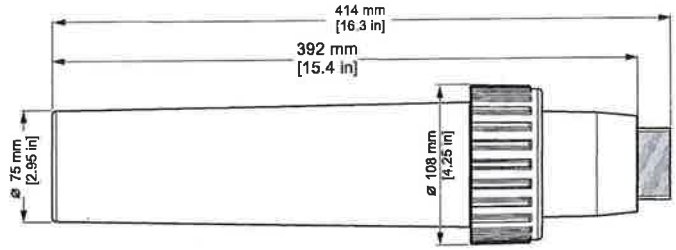
Engineering Specifications

1. The ammonium sensor shall consist of a digital sensor designed to connect to a universal controller.
2. The ammonium sensor shall be a continuous-reading sensor that utilizes Ion-Selective Electrode (ISE) technology for ammonium and potassium with pHD reference electrode and temperature sensor.
3. The sensor cartridge shall be factory calibrated with all electrodes calibrated to each other and integrated in a one-piece design.
4. The sensor body shall be 316 stainless steel with ends made of Ryton®. All parts of the sensor shall be corrosion resistant and fully-immersible.
5. The measurement range shall be 0.2 to 1000 mg/L NH_4-N.
6. The sensor shall compensate for the interference effects of potassium and temperature.
7. The sensor shall operate without the requirement of sample conditioning.
8. The sensor shall be offered with optional cleaning device that encloses the entire sensor.
9. The optional cleaning device can either take compressed air from the plant supply or from an onsite compressor supplied by the manufacturer.
10. The sensor shall be furnished with a choice of rail or chain mount kit.
11. The sensor shall be capable of performing a one or two-point inline matrix correction to adapt the sensor to the wastewater matrix.
12. The sensor shall be the NH4D sc Ammonium Sensor, manufactured by Hach Company.

Dimensions

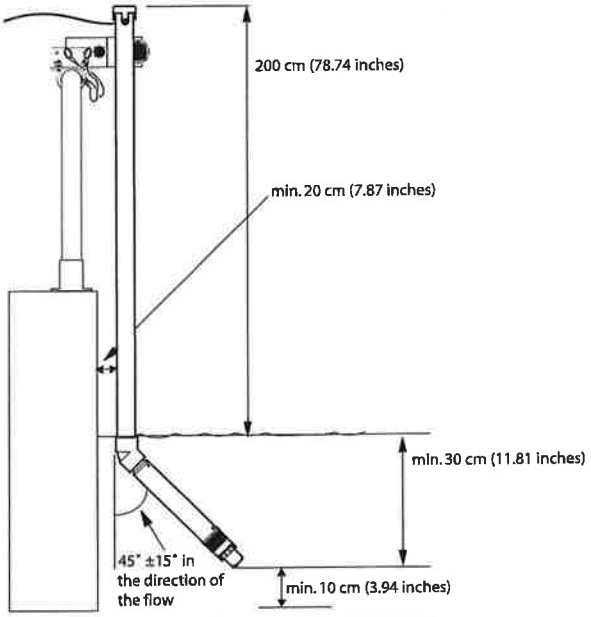
The Hach NH4D sc Ammonium Sensor can be mounted using the Rail Mount or Chain Mount Kit (see ordering information), with or without the optional cleaning unit.





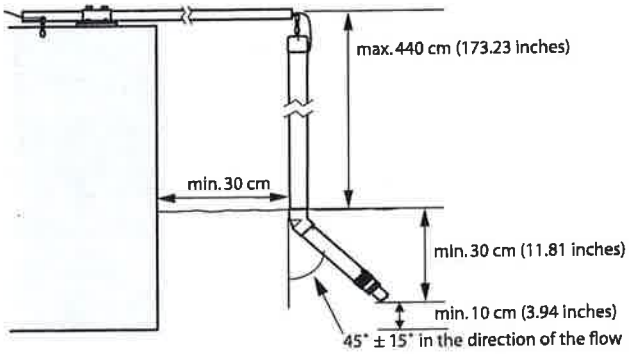
Cleaning Unit

Installation Examples



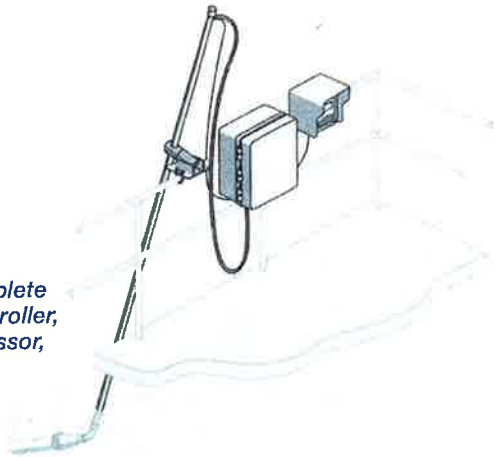
Note: Ensure the sensor cannot make contact with the tank wall

Rail Mount



Note: Ensure the sensor cannot make contact with the tank wall

Chain Mount



NH4D sc sensor as complete system with Hach sc controller, sensor, optional compressor, and cleaning unit.

Ordering Information

LXV437.99.00002	NH4D sc Ammonium Sensor, includes 10 m (32.8 ft.) integral cable and a calibrated sensor cartridge

Mounting Kits

6184900	Rail Mount Kit
LZX914.99.12400	Chain Mount Kit

Accessories

LZY331	Cleaning Unit
6174100	High Output Air Blast Cleaning Compressor, 115 Vac
6174200	High Output Air Blast Cleaning Compressor, 230 Vac

Replacement Parts

6188400	Calibrated Sensor Cartridge

To complete your ammonium measurement system, choose from these Hach controllers...

Model sc100 Controller

(see Lit. #2463)

The Model sc100 Controller receives data from one or two sensors. Its "plug and play", mix-and-match operation lets it fit into any facility or workflow. Digital communication with any Hach digital sensor or probe is simple and reliable.



Model sc1000 Controller

(see Lit. #2403)

Get the same great features as the sc100 Controller above—"plug and play", all digital operation and communication—but with the Hach sc1000 Controller, up to eight Hach sensors can be used with one controller in any combination. The sc1000 Controller is also expandable and upgradeable to easily adapt to your needs.



At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

For current price information, technical support, and ordering assistance, contact the Hach office or distributor serving your area.

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NITRATAX™ sc UV Nitrate Sensors



Sophisticated, yet simple, technology means the Hach NITRATAX sc UV Nitrate Sensors require little maintenance and offers remarkably low cost of operation.

Features and Benefits

UV Analysis—Eliminates Reagents, Sampling, and Sample Conditioning

Using advanced ultraviolet (UV) absorption technology, Hach NITRATAX sc UV Nitrate Sensors offer unprecedented simplicity, accuracy, and economy in nitrate analysis. By continuously measuring the UV light absorbed by nitrates, NITRATAX sc sensors eliminate the need for reagents, sample conditioning, and frequent calibrations.

UV Absorption Method—Proven, Continuous, Precise

NITRATAX sc sensors rely on the principle that molecular bonds absorb ultraviolet (UV) light—in this case, nitrate (NO_3) and nitrite (NO_2) absorb UV light. As the concentration of nitrate or nitrite increases, UV absorption also increases. A built-in photometer measures the primary beam, while a second beam of UV light provides a reference standard and corrects for interference caused by turbidity and organic matter. Results are reported on the controller unit's alphanumeric display in mg/L of nitrate (NO_3) or nitrate-nitrogen (NO_3-N).

Choice of Three Models

NITRATAX plus sc—Process sensor for continuous measurement in drinking water, wastewater, or activated sludge. Turbidity compensation using reference measurement.

NITRATAX eco sc—Low cost sensor for measurement, especially for sewage treatment plants with intermittent aeration technology. Turbidity compensation using reference measurement.

NITRATAX clear sc—Process sensor for continuous measurement in clean water such as drinking water or wastewater effluent.

Self-Cleaning Sensor

With the sensor submerged in the sample stream, the detector windows are automatically cleaned by a built-in wiper that eliminate surface films or particles that can diminish accuracy.

Optional Bypass Panel

An optional Bypass Panel—or flow-through sample cell—is available for clean water applications when direct immersion in a sample stream is impractical. The Bypass Panel uses the same probes as the immersion sensors for greater flexibility.

Applications

Hach NITRATAX sc sensors are the ideal choice to ensure consistent water quality, keep discharge waters clean, comply with regulatory requirements, and safeguard the environment.

Wastewater—Monitor influent, effluent, and aeration basin water. Control methanol feed and/or mixed liquor recycle denitrification process to minimize costs.

Drinking Water—Monitor nitrate levels in both influent and effluent streams before and after the disinfection process.

Agriculture—Measure and control nitrate discharges into rivers, lakes, wetlands, and other natural bodies of water.

Full-Featured "Plug and Play" Hach sc Digital Controllers

There are no complicated wiring or set up procedures with any Hach sc controller. Just plug in any combination of Hach digital sensors and it's ready to use—it's "plug and play."

One or multiple sensors—The sc controller family allows you to receive data from up to eight Hach digital sensors in any combination using a single controller.

Communications—Multiple alarm/control schemes are available using the relays and PID control outputs. Available communications include analog 4-20 mA, digital MODBUS (RS485 and RS232) or Profibus DP protocols. (Other digital protocols are available. Contact your Hach representative for details.)

Data logger—A built-in data logger collects measurement data, calibration, verification points, and alarm history.

DW
WW
IW
E

DW = drinking water WW = wastewater municipal PW = pure water / power IW = industrial water E = environmental C = collections FB = food and beverage

HACH®

Be Right™

Specifications*

	NITRATAX plus sc	NITRATAX eco sc	NITRATAX clear sc
Measuring Principal	Reagent-free UV absorption with patented 2-beam technique		
Measuring Gap/Path Length	1, 2, and 5 mm	1 mm	5 mm
Measuring Range	0.1 to 100.0 mg/L NO_{2+3}-N (1 mm) 0.1 to 50.0 mg/L NO_{2+3}-N (2 mm) 0.1 to 25.0 mg/L NO_{2+3}-N (5 mm)	1.0 to 20.0 mg/L NO_{2+3}-N (1 mm)	0.5 to 20.0 mg/L NO_{2+3}-N (5 mm)
Detection Limits	0.1 to 100 mg/L NO_3-N	1.0 to 20 mg/L NO_3-N	0.5 to 20 mg/L NO_3-N
Accuracy	±3% of reading or ±0.5 mg/L , whichever is greater	±5% of reading or ±1.0 mg/L , whichever is greater	±5% of reading or ±0.5 mg/L , whichever is greater
Resolution	0.1 mg/L	0.5 mg/L	0.1 mg/L
Sludge Compensation	Yes	Yes	—
Measurement Interval	1 minute	5 minutes	5 minutes
Response Time (T100)	1 minute	15 minutes	5 minutes
Available with Bypass	Yes	No	Yes
Sensor Construction			
Enclosure	Stainless Steel 1.4571		Stainless Steel 1.4581
Wiper Axle	Stainless Steel 1.4104	Stainless Steel 1.4571	
Wiper	Silicon		
Measuring Window	Quartz Glass		
Functional Verification	Using standard solutions		
Service Intervals	6 months or as experience dictates		
Maintenance Required (typical)	1 hour/month	2 hours/month	1 hour/month
Operating Temperature	2 to 40°C (36 to 104°F)		
Operating Pressure	0.5 bar (7.2 psi), maximum		
Cable Length	10 m (32.8 ft.)		
Dimensions (approximate)	33.3 x 7.0 cm (13.1 x 2.8 in.)	32.3 x 7.5 cm (12.7 x 3.0 in.)	32.3 x 7.5 cm (12.7 x 3.0 in.)

Specifications subject to change without notice.

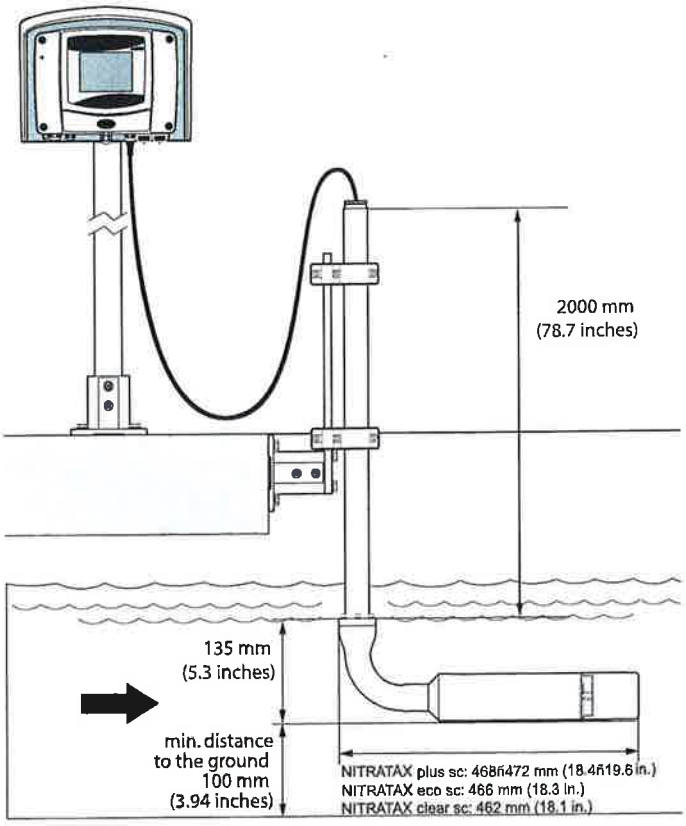
Hach NITRATAX sc UV Nitrate Sensors use UV light absorption to assess the sum of nitrates and nitrites in aqueous sample streams.

A stainless-steel sensor immersed directly in the sample stream, provide continuous readings at exceptionally low operating cost.

- *Two UV light wavelengths provide precise readings, compensating for solids and turbidity.*
- *With the sensor submerged in the sample stream, the detector windows are automatically cleaned by a built-in wiper, eliminating surface films or particles that can diminish accuracy.*

Installation Dimensions

Installation for mounting NITRATAX sc using Fixed Point Installation Kit (LZX414.00.10000).



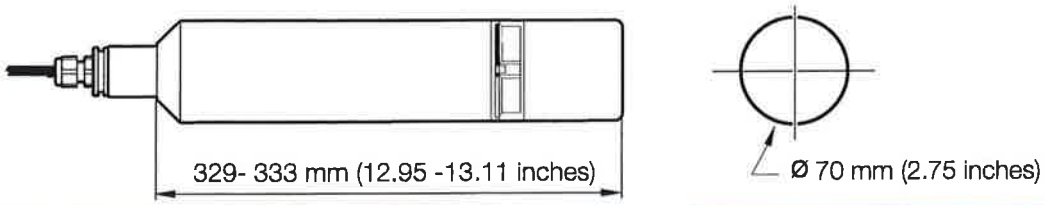
Engineering Specifications

1. The nitrate sensor shall be a continuous-reading sensor that utilizes a 2-beam ultra-violet absorption technology with a 1, 2, or 5 mm path length.
2. The measurement range shall be 0.1 to 100 mg/L NO_2-N + NO_3-N, depending on model.
3. The measurement interval shall be user-selectable from 1 to 30 minutes (unit dependent) with the ability to average up to 12 signals depending upon unit selected.
4. The sensor shall compensate for the interference effects of turbidity and organic contamination.
5. The sensor shall provide reagent-free operation without the requirements of sample conditioning.
6. The sensor shall be self-cleaning via a wiper and retain a life-long factory calibration.
7. The sensor shall be warranted for one full year against defects in material and workmanship.
8. The sensor shall be the NITRATAX plus sc, NITRATAX eco sc, or NITRATAX clear sc UV Nitrate Sensor for nitrate measurement, manufactured by Hach Company.

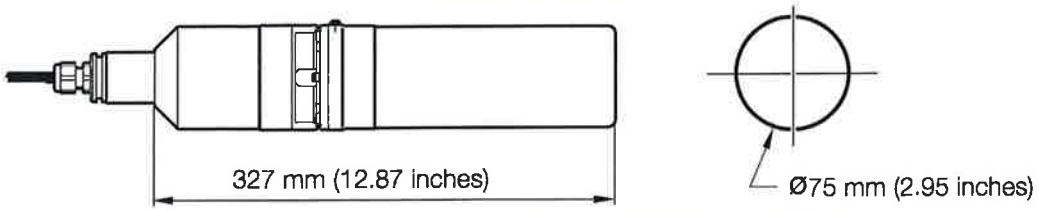
Sensor and Bypass Panel Dimensions

Hach NITRATAX sc UV Nitrate Sensors can be installed using a fixed-point installation kit. With the cable supplied, the sensor can be used in a sample stream within 10 meters (32.8 feet) of the controller.

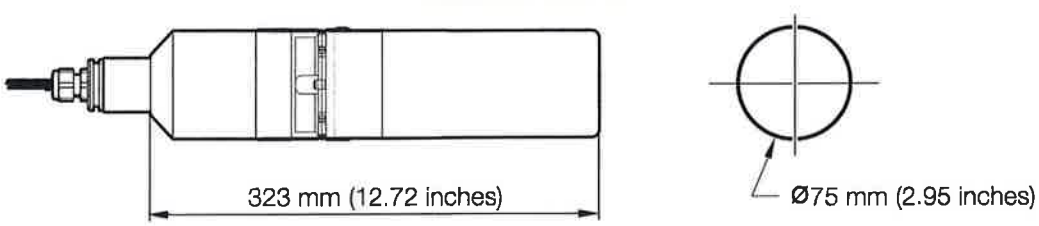
NITRATAX plus sc



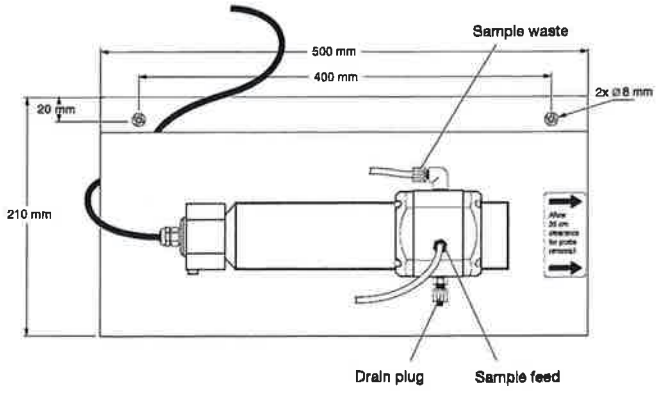
NITRATAX eco sc



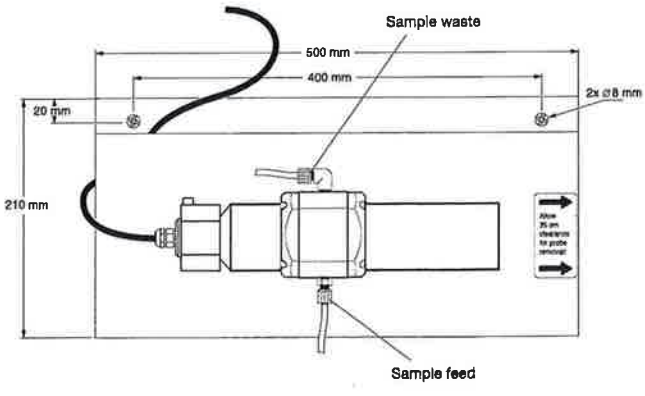
NITRATAX clear sc



NITRATAX plus sc Bypass Panel



NITRATAX clear sc Bypass Panel



Ordering Information

Sensor Selection

	NITRATAX plus sc	NITRATAX eco sc	NITRATAX clear sc
Wastewater Application			
Anoxic Zone	X	-	-
Aeration Tank	X	●	-
Plant Effluent	X	-	X
Sequencing Bath Reactor (SBR)	X	X	-
Cascade	X	●	-
Drinking Water Application			
Influent Monitoring	X	-	X
Water Blending	X	-	X
Denitrification Process	X	-	X
Outlet/Quality Control	X	-	X
Bypass/Flowthrough Cell	X	-	X

X = suitable ● = of limited suitability

NITRATAX sc UV Nitrate Analyzer Systems

6139200	NITRATAX plus sc system includes sensor with 2 mm path length, mounting hardware, and sc100 controller
6139500	NITRATAX plus sc system includes flow-through cell sensor with 2 mm path length, mounting hardware, and sc100 controller.
6139400	NITRATAX eco sc system includes sensor with 1 mm path length, mounting hardware, and sc100 controller
6139300	NITRATAX clear sc system includes sensor with 5 mm path length, mounting hardware, and sc100 controller
6139600	NITRATAX clear sc system includes flow through cell sensor with 5 mm path length, mounting hardware, and sc100 controller

Individual NITRATAX sc UV Nitrate Sensors

All sensors are equipped with 10 m (32.8 ft.) cable.

LXV417.99.10002
NITRATAX plus sc sensor, 1 mm path length

LXV417.99.20002
NITRATAX plus sc sensor, 2 mm path length

LXV417.99.50002
NITRATAX plus sc sensor, 5 mm path length

LXV420.99.50002
NITRATAX clear sc sensor, 5 mm path length

LXV415.99.10002
NITRATAX eco sc sensor, 1 mm path length

Accessories

LZX414.00.10000
Mounting hardware for sensor

LZX869	Flow-through cell for NITRATAX plus sc-sensors, 2 mm path length
LZX867	Flow-through cell for NITRATAX plus sc-sensors, 5 mm path length
LZX866	Flow-through cell for NITRATAX clear sc-sensors, 5 mm path length
LCW828	Calibration standard 25 mg/L NO_3 (5.56 mg/L NO_3-N)
LCW825	Calibration standard 50 mg/LNO_3 (11.3 mg/L NO_3-N)
LCW826	Calibration standard 100 mg/L NO_3 (22.6 mg/L NO_3-N)
LCW827	Calibration standard 200 mg/L NO_3 (45.2 mg/L NO_3-N)
LCW863	Calibration standard 400 mg/L NO_3 (90.4 mg/L NO_3-N)
LZX148	Spare wiper blades for 1 mm Nitratax, pk/5
LZX012	Spare wiper blades for 2 mm Nitratax, pk/5
LZX117	Spare wiper blades for 5 mm Nitratax, pk/5

Cable Accessories

5867000	Junction box for extension cables
5796000	Extension cable, 7.6 m (25 ft.)
5796100	Extension cable, 15.2 m (50 ft.)
5796200	Extension cable, 30.5 m (100 ft.)

This sensor requires a Hach sc digital controller.
Ask your Hach representative for more information.

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In the interest of improving and updating its equipment, Hach Company reserves the right to alter specifications to equipment at any time.

At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

For current price information, technical support, and ordering assistance, contact the Hach office or distributor serving your area.

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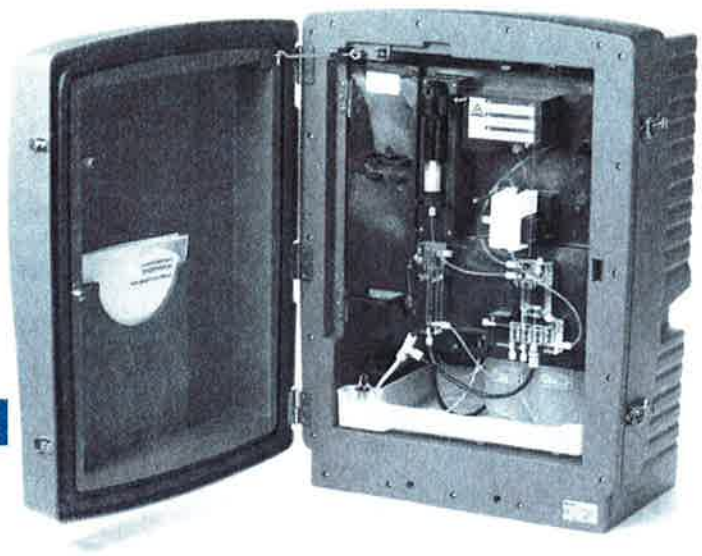
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www.hach-lange.com

HACH®

Be Right™

PHOSPHAX™ sc Phosphate Analyzer



The Hach PHOSPHAX sc Phosphate Analyzer measures PO_4-P concentrations as low as 0.05 mg/L and as high as 50 mg/L. And operation costs are low because it uses 50% less chemicals. This is no "black box" system—you get transparent, high-quality measurements for reliable values.

Features and Benefits

Wide Measurement Range

Hach PHOSPHAX sc Phosphate Analyzers are available in two measurement ranges. Each range consists of an optimized dosing vessel and photometer to ensure the highest accuracy at each range.

Low Range—0.05 to 15 mg/L PO_4-P
High Range—1 to 50 mg/L PO_4-P

Wide Detection Limit for a Variety of Wastewater Applications

With detection limits as low as 0.05 mg/L and as high as 50 mg/L, the PHOSPHAX sc Phosphate Analyzer can be used anywhere in the wastewater treatment process, from the influent or start of the phosphorous removal process where phosphate levels may be high to the effluent where phosphate levels are at their lowest.

Low Cost of Operation with Proven Analysis Method

The PHOSPHAX sc analyzer determines phosphate concentration using the vanado-molybdate yellow colorimetric method. This method requires very little reagent and therefore reduces operating cost considerably.

Fast Response Time for Control of Nutrient Removal Process

Nutrient removal is a critical and sensitive process in wastewater treatment. Quick response is vital for optimal operation. The fast response time of only 5 minutes, including sample preparation and the ability to choose the measurement interval, make the PHOSPHAX sc Phosphate Analyzer the fastest responding phosphate analyzer on the market.

Easy Installation at the Measurement Point

Hach's PHOSPHAX sc Phosphate Analyzer is designed to be installed at the measurement point—including outdoor installations. The housing is weatherproof and lockable so that the PHOSPHAX sc analyzer can be installed right at the basin, even in the toughest climates.

- *Easily mounts to a wall. Rail- and stand-mounting options are available*
- *The unit comes complete and assembled—no separate housing is required*

Low Maintenance

Several features make the PHOSPHAX sc Phosphate Analyzer easy to use and maintain:

- *The Filterprobe sc is equipped with a self-cleaning filter*
- *With at least a 4-month supply of reagents, the PHOSPHAX sc analyzer offers the most economical consumption*
- *Automatic cleaning at customized intervals and automatic zero-calibration at each measuring cycle*
- *Comprehensive self diagnosis*
- *Easy access to reagents and wear parts*

Full Featured "Plug and Play" Digital Controller

There's no complicated wiring or set up procedures with the Hach sc1000™ Multi-parameter Universal Controller. Just plug in any Hach digital sensor or analyzer and it's ready to use—it's "plug and play." Please see Hach Lit. #2403 for more information.

Optional Filterprobe sc Sample Preparation System

The Hach PHOSPHAX sc Phosphate Analyzer is available with a low-maintenance filtration probe that removes particles larger than 0.15 µm. Filtrate is sent to the analyzer while air bubbles constantly clean the filtration membrane. When cleaning is required, a simple wipe with a cloth or sponge is all that is needed. The PHOSPHAX sc analyzer may also be used with existing filtration systems that deliver a continuous sample stream.

Specifications*

PHOSPHAX sc Phosphate Analyzer

Measurement Method
Photometric method using vanado-molydan

	Low Range	High Range
Measurement Range (PO_4-P)	0.05 to 15 mg/L	1 to 50 mg/L
Lower Detection Limit	0.05 mg/L	1 mg/L
Accuracy	2% ±0.05 mg/L	2% ±1 mg/L
Reproducibility	2% ±0.05 mg/L	2% ±1 mg/L

Response time (T90)
Less than 5 minutes, including sample preparation

Measurement Interval
5 to 120 minutes, adjustable

Sample Conditions
Temperature: 4 to 40°C (39 to 104°F)
pH: 5 to 9

Operating Conditions
Temperature: -20 to 45°C (-4 to 113°F)
Humidity: 95% relative humidity, non-condensing

Storage Conditions (excluding reagents)
Temperature: -20 to 50°C (-4 to 122°F)
Humidity: 95% relative humidity, non-condensing

Power Supply
230 Vac/50Hz or optional 115 Vac/ 50-60Hz; connected to sc1000 Multi-parameter Universal Controller (see Hach Lit. #2403)

Data Transmission
With data cable on sc1000 Multi-parameter Universal Controller (see Hach Lit. #2403)

Outputs
Relay, current outputs, and bus interface via sc1000 Multi-parameter Universal Controller (see Hach Lit. No. 2403)

Special Features
ASA UV-resistant, lockable housing, rated to IP55
Automatic cleaning and calibration
Extensive self-diagnostics
Optional 2-channel version for continuous sample preparation

Sample Preparation
Filterprobe sc (see specifications below) or continuous sample preparation (approximately 500 to 1000 mL/min) with FILTRAX, ultrafiltration, etc.

Dimensions
540 x 720 x 390 mm
(21.3 x 28.3 x 15.4 in.)

Cable Length
2 m (6.6 ft.) from housing

Weight
35 kg (77 lbs.) including reagents

Filterprobe sc

Operation
In-situ membrane filtration

Filter modules are exchangeable

Continuous self-cleaning with air bubbles

Particles larger than 0.15 µm are separated from sample stream

Power Supply
Via analysis instrument 100 VA (brief peak power consumption: 1000 VA), with 5 or 10 m heated filter probe hose. Electrical fuse protection.

Immersion Depth
3 m (9.8 ft.), maximum

Sample Flow Rate
3 m/s, maximum

Filtrate Flow Rate
5 mL/minute, minimum,
4 out of 5 minutes

Maintenance
0.5 hours per month, typical

Dimensions
315 x 250 x 120 mm (12.4 x 9.8 x 4.7 in.)

Supply Hose Length
5 or 10 m (16.4 or 32.8 ft.)

Weight
Filter housing: 9.5 kg (30 lbs.)
Hose: 8 kg (18 lbs.)

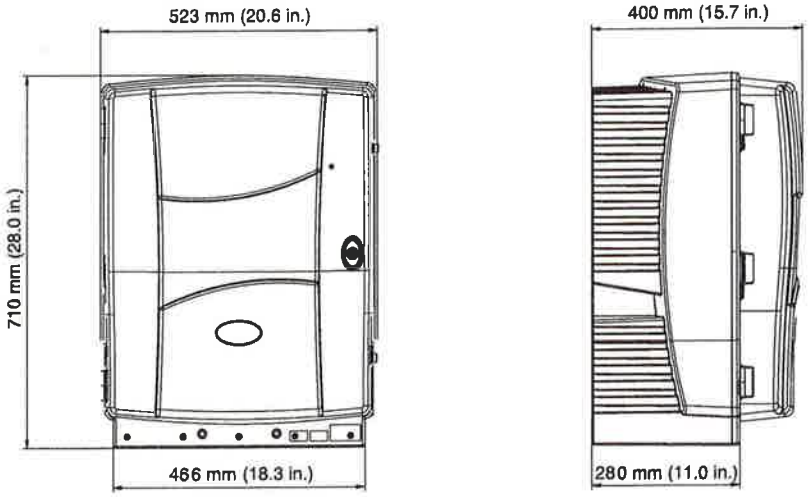
**Specifications subject to change without notice.*

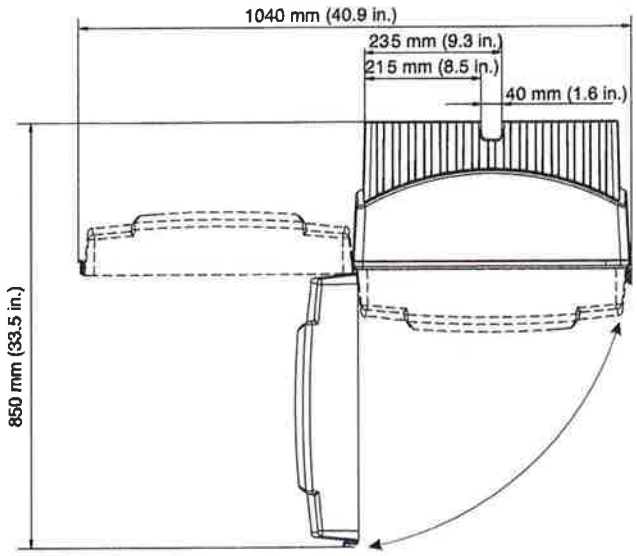
Engineering Specifications

1. The analyzer shall utilize the vanado-molybdate yellow colorimetric method to measure ortho-phosphate (PO_4-P).
2. The analyzer shall be housed in an ASA UV-resistant, IP55-rated, lockable housing to be installed at the measurement point.
3. The analyzer measurement range shall be from 0.5 to 50 mg/L PO_4-P in two intervals.
4. The lower detection limit of measurements shall be 0.05 mg/L PO_4-P.
5. The response time, including sample preparation, shall be 5 minutes.
6. The analyzer's operating temperature range shall be from -20 to 45°C.
7. The power requirements shall be 115 Vac, 50-60Hz or 230 Vac, 50 Hz, via the controller.
8. The analyzer shall zero-calibrate and clean automatically.
9. The analyzer shall be equipped with self-diagnostics.
10. There shall be available an optional filtration system that conditions the sample for the analyzer.
11. The analyzer shall be Hach Company PHOSPHAX sc Phosphate Analyzer.

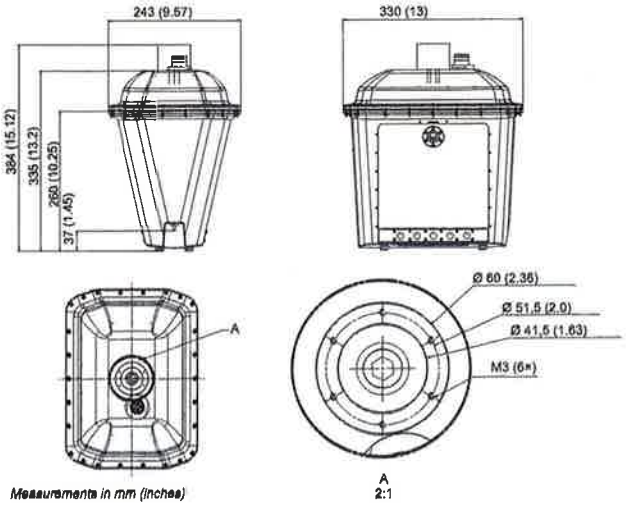
Dimensions

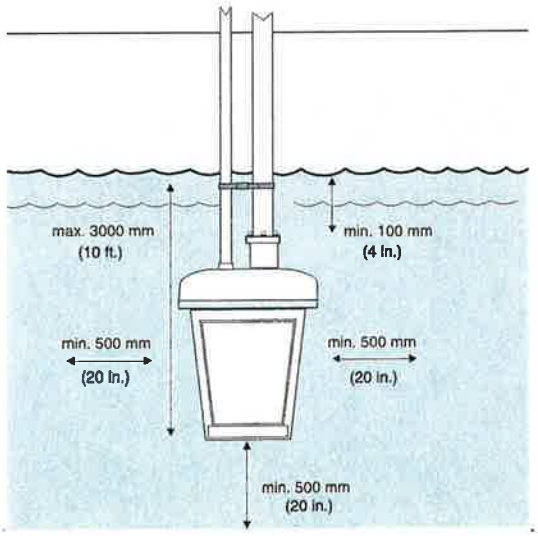
The PHOSPHAX sc Phosphate Analyzer is designed for wall mounting, outdoors or indoors. Rail- and stand-mounting options are available. The enclosure is rated to IP55, is weatherproof, and is lockable.





The Filterprobe sc should be installed at least 20 inches from the bottom and walls of the tank and must have at least four inches, but no more than 10 feet, of water above it.





Ordering Information

PHOSPHAX sc Phosphate Analyzer with Filterprobe sc

61590-00 PHOSPHAX sc Phosphate Analyzer, 0.05 to 15 mg/L, 115 Vac; includes filtration probe with 5 m heated hose

61591-00 PHOSPHAX sc Phosphate Analyzer, 0.05 to 15 mg/L, 115 Vac; includes filtration probe with 10 m heated hose

61594-00 PHOSPHAX sc Phosphate Analyzer, 1 to 50 mg/L, 115 Vac; includes filtration probe with 5 m heated hose

61595-00 PHOSPHAX sc Phosphate Analyzer, 1 to 50 mg/L, 115 Vac; includes filtration probe with 10 m heated hose

PHOSPHAX sc Phosphate Analyzer Only

61596-00 PHOSPHAX sc Phosphate Analyzer, 0.05 to 15 mg/L, one channel with continuous sample, 115 Vac

61598-00 PHOSPHAX sc Phosphate Analyzer, 1 to 50 mg/L, one channel with continuous sample, 115 Vac

For two-channel version of the PHOSPHAX sc Phosphate Analyzer, please contact your Hach representative or call 1-800-227-4224.

Sample Filtration

LXV429.99.01000 Filterprobe sc, 5 m heated hose, 230 Vac/50 Hz

LXV429.99.02000 Filterprobe sc, 10 m heated hose, 230 Vac/50 Hz

LXV429.99.01100 Filterprobe sc, 5 m heated hose, 115 Vac/50 to 60 Hz

LXV429.99.02100 Filterprobe sc, 10 m heated hose, 115 Vac/50to 60 Hz

57389-00 Filtrax system, 2 m unheated hose, 115 Vac

57390-00 Filtrax system, 10 m heated hose, 115 Vac

57391-00 Filtrax system, 20 m heated hose, 115 Vac

57389-01 Filtrax system, 2 m unheated hose, 230 Vac

57390-01 Filtrax system, 10 m heated hose, 230 Vac

57391-01 Filtrax system, 20 m heated hose, 230 Vac

Controller

To order the sc1000 Multi-Parameter Universal Controller, please contact your Hach representative or call 1-800-227-4224.

Mounting Accessories

LZY285 Rail Mounting Kit for PHOSPHAX sc analyzer with sc1000 controller

LZY316 Rail Mounting Kit for PHOSPHAX sc analyzer without sc1000 controller

LZY286 Stand Mounting Kit for PHOSPHAX sc analyzer with sc1000 controller

LZY287 Stand Mounting Kit for PHOSPHAX sc analyzer without sc1000 controller

LZX414.00.50000 Rim Mounting Kit for Filterprobe sc

LZX414.00.60000 Rail Mounting Kit for Filterprobe sc

Reagents

28252-54 PHOSPHAX Reagent, 2000 mL

28253-52 PHOSPHAX Cleaning Solution, 1000 mL

Replacement Parts

LZY181 Pump Head for PHOSPHAX sc analyzer air pump

LZY140 Filter Module for Filterprobe sc

At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

For current price information, technical support, and ordering assistance, contact the Hach office or distributor serving your area.

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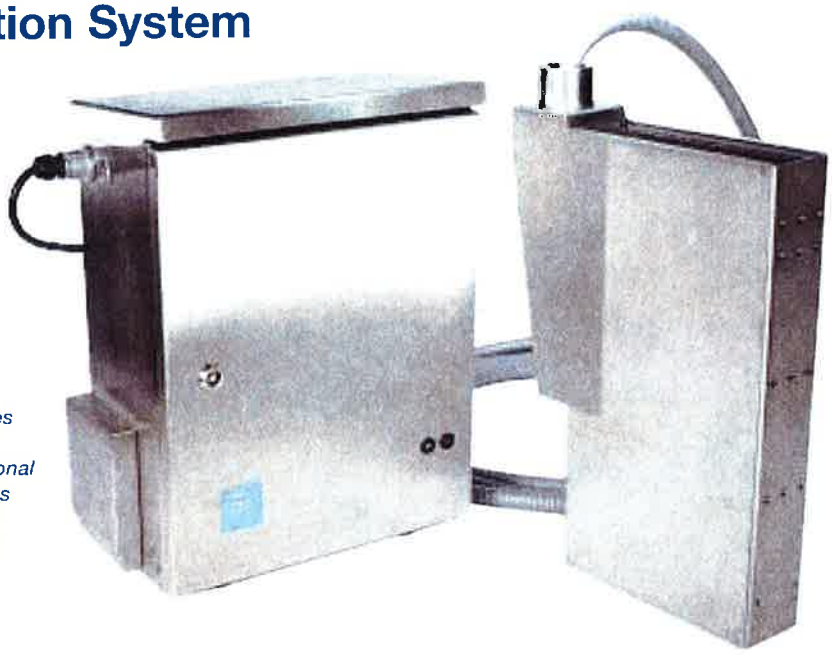
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HACH®

Be Right™

Filtrax™ Sample Filtration System



The Hach Filtrax Sample Filtration System provides high permeate quality with low operating costs. It is unnecessary to purchase and maintain additional pumps. Operation is automatic and self-cleaning is continuous. Depend on the Filtrax system to work reliably—even with a high sludge volume index or floating sludge.

Features and Benefits

Established Technology *In Situ* and Down the Line

The Hach Filtrax Sample Filtration System is specifically designed for efficient wastewater analysis. Samples are extracted directly from the activated sludge aeration basin or final settling tank and solids are removed *in situ*. Separate, high-maintenance submersible pumps become unnecessary. Sample delivery lines and analyzers stay clean. Sample extraction automatically alternates between two continuously cleaned filter elements to allow optimum cleaning of the idle filter. Sample pre-treatment and delivery is continuous and efficient—even with a high sludge volume index or floating sludge.

Simple to Use and Maintain

A built-in air purge system in the Filtrax system prevents adhesion of solids to the filter membranes and greatly reduces maintenance cost and time. All tubing is completely accessible and easy to replace. Moving parts never come into contact with the sample. Self-diagnostics provide operators with three levels of error messages if components require attention. Operators control all Filtrax system functions with menu-driven software. The instrument display provides continuous readout of current flow rate through each filter module as well as heater status. Two programmable alarm relays are provided—one relay can be used to alert operators to inspect the Filtrax system when flow decreases and a second programmable relay can shut down the unit if flow decreases further.

Operating Principle

All control components are heated and contained in an IP55-rated enclosure, suitable for outdoor installation. The Filtrax system is made of three components.

- **Control unit:** *The control unit is internally heated for outdoor installation. Inside are sample pumps, an air compressor for filter membrane cleaning, and the electronic controller. Filters are constantly cleaned by a vigorous stream of air bubbles against the sides of the module. Two small peristaltic pumps alternately draw wastewater sample through the two filter membranes for optimum cleaning of the idle filter.*
- **Filtration module holder:** *The filtration module holder is made of stainless steel and is designed for submerged installation. It contains two filter modules each equipped with an ultra-filtration (0.15 µm) filter membrane.*
- **Sample delivery hoses:** *Samples are pulled to the control unit from the filtration module holder with a 15-ft. (5-m) heated suction hose. This hose also contains the air compressor tubing. Samples are then delivered directly to the on-line measuring instrument.*

Hach's Family of Nutrient Analyzers

The Hach Filtrax Sample Filtration System is designed to optimize performance of Hach's Amtax™ sc Ammonia Analyzer or Phosphax™ sc Phosphate Analyzer. It also complements Hach's NITRATAX plus sc or NITRATAX clear sc Nitrate Analyzer by supplying solids-free sample to its flow through panel. Rely on the Filtrax system to provide properly conditioned sample to Hach's APA 6000™ Alkalinity, Ammonia, Monochloramine, and Phosphate Analyzers too.

Specifications*

Sample Flow
Approx. 900 mL/h for up to three process instruments

Sample Temperature
41 to 104°F (5 to 40°C)

Ambient Temperature
-4 to 104°F (-20 to 40°C)

Suction Head Space
9 ft. (3 m) maximum (filter module holder to controller)

Maximum Sample Delivery Head Space
21 ft. (7 m) maximum (control unit to analyzer)

Cable Length
Suction Hose: 5 m (heated)

Sample Delivery Hose: 2 m (heated or unheated), 10 m (heated), 20 m (heated), or 30 m (heated)

Outputs
Programmable Fault Alarm Contact:
Potential free contact (230 V, 3A maximum)

Programmable Warning Contact:
Potential free contact (230 V, 3A maximum)

Service Interface: RS232

Power Supply
115 or 230 Vac ±10% AC, 50/60 Hz

Inspection Requirement
Approximately 1 hour/month

Enclosure Class
IP55 (outdoor installation)

Certification
CE, UL, CSA

Weight (approximate)
Control Unit: 49 lb. (kg)

Module Holder with 5-m Suction Tube: 20 lb. (9 kg)

Sample Tube, 33-ft. (5-m): 11 lb. (5 kg)

Sample Tube, 66-ft. (20-m): 22 lb. (10 kg)

Mounting pipe, 6.6-ft. (2-m): 11 lb. (5 kg)

Dimensions (approximate)
Control Unit: 17 x 21 x 9 in. (430 x 530 x 220 mm)

Module Holder: 4 x 20 x 13 in. (92 x 500 x 340 mm)

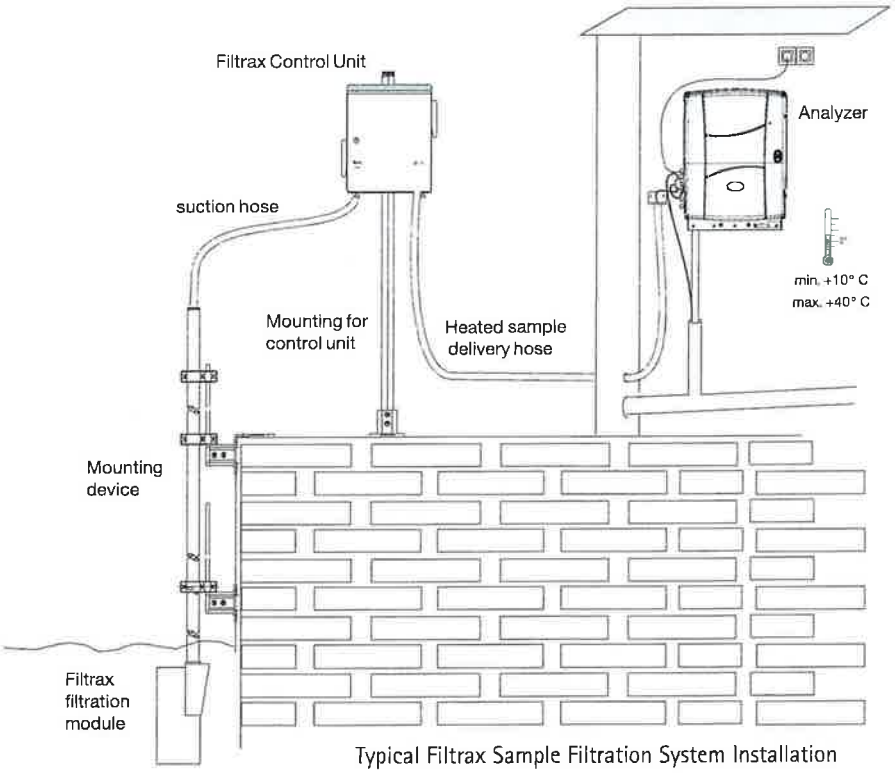
**Specifications subject to change without notice.*

Engineering Specifications

1. The sample extraction and filtration system shall consist of three components: filter module holder, control unit, and sample delivery hose.
2. The filter module holder shall be immersed at the sampling point.
3. The module holder shall contain two filters that are each spanned by a 0.15 µm membrane.
4. An air-purge system underneath the two immersed filter modules shall provide a continuous air supply to automatically clean the filter elements by preventing adhesion of solids to the filter membranes.
5. The sample shall be drawn through the immersed membranes via two tube-metering pumps which are housed within the control unit.
6. The control unit must be an IP55 rated, heated enclosure, designed for outdoor installation.
7. Operating with no wetted parts, the metering pumps shall draw the sample alternately from the two filter modules into the control unit via a 5-m (15-ft.) heated suction hose.
8. The sample shall be pumped up to 30-m (90-ft.) from the control unit to the process-measuring unit.
9. The system shall deliver a sample at a rate of up to 900 mL/h for up to three process instruments.
10. The sample filtration system shall provide two programmable, potential free contacts (115 or 230 VAC, max. 3A) to monitor the flow rate across the filter media.
11. The system power requirements shall be 100 to230 VAC, 50/60 Hz.
12. The system's ambient temperature range shall be from -20 to 40 °C (-4 to 104 °F).
13. The entire filtration system shall be designed to meet UL, CSA, and CE safety standards.

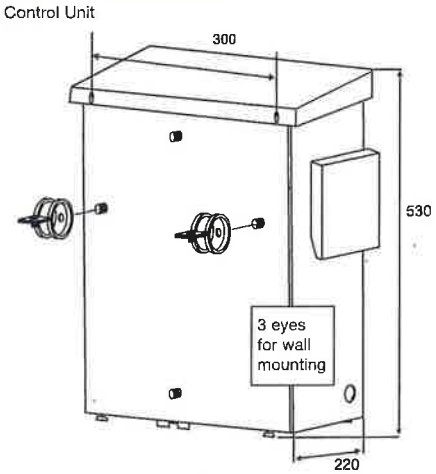
Installation Options

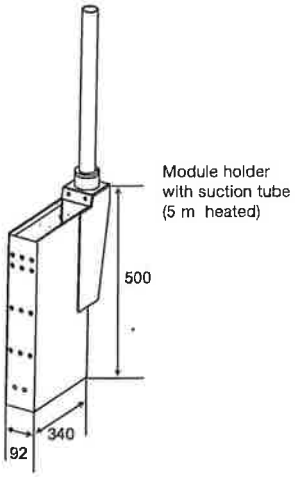
The control unit of the Filtrax system can be mounted on a wall using supplied hardware. Where no wall is available, an optional, free-standing mounting kit for the control unit can be used. An optional basin wall mount kit available for module holder mounting (submerged) in an aeration basin or effluent channel is also available. The control unit and sample suction hose are equipped with standard heating; also available unheated in several lengths.



Typical Filtrax Sample Filtration System Installation

The Hach Filtrax Sample Filtration System module holder should be installed at least three inches from the bottom of the tank and must have between three and 18 inches of water above it. Mount the control unit on a wall with three #10 mounting screws (supplied). Four pipe bracket locations for vertical or horizontal guardrail mounting are provided. (All measurements at the right are in mm.)





Ordering Information

The Hach Filtrax Sample Filtration System includes: module holder, control unit, sample delivery hose, two filter modules, wall mount installation kit for control unit, maintenance schedule, six pump hose sets, six air filter elements, and manual.

5738900	Filtrax System with 2-m Unheated Sample Delivery Hose, 115 Vac
5739000	Filtrax System with 10-m Heated Sample Delivery Hose, 115 Vac
5739100	Filtrax System with 20-m Heated Sample Delivery Hose, 115 Vac
5738901	Filtrax System with 2-m Unheated Sample Delivery Hose, 230 Vac
5739001	Filtrax System with-10 m Heated Sample Delivery Hose, 230 Vac
5739101	Filtrax System with-20 m Heated Sample Delivery Hose, 230 Vac

Accessories

4696400	Power Cord, 115 Vac, 6 ft.
4743900	Power Cord, 230 Vac, 6 ft.
LZX414.00.40000	Basin Wall-mounting Kit, for module holder
LZX676	Mounting Kit, for control unit

Replacement Items

LZX667	Maintenance Kit
LZX677	Filter Module

At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

For current price information, technical support, and ordering assistance, contact the Hach office or distributor serving your area.

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www.hach.com

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In the interest of improving and updating its equipment, Hach Company reserves the right to alter specifications to equipment at any time.

HACH®
Be Right™

sc1000™ Multi-parameter Universal Controller



The Hach Model sc1000 Multi-parameter Universal Controller is a state-of-the-art modular controller system. Use it directly with 8 sensors or network several together to accommodate many more sensors and parameters. It is completely compatible with Hach's full range of digital sensors.

Features and Benefits

Modular System

The Hach sc1000 Multi-parameter Universal Controller is a fully modular system consisting of a Display Module and one or more Probe Modules.

Display Module—The sc1000 Display Module is intuitive, with an easy to use interface and large color touch-screen display that can be used for any number of parameters. One Display Module controls one or several Probe Modules connected by a digital network. The Display Module is fully portable, and can be disconnected and moved anywhere within the network. The Display Module is also available with GSM capability.

Probe Module—Each sc1000 Probe Module provides power to the system and can accept up to 8 digital sensors/expansion boards. Probe Modules can be networked together to accommodate up to 32 digital sensors/expansion boards attached to the same network.

"Plug and Play" Operation

There's no complicated wiring or set up procedures with the sc1000 controller. Just plug the sensor into a Probe Module and it's ready for use. No special ordering or software configuration is needed.

Mix and Match Digital Sensors

Any Hach digital sensor can be plugged into the sc1000 Probe Module in any combination. Available parameters include—but are not limited to—dissolved oxygen, pH, ORP, conductivity, turbidity, suspended solids, nitrate, chlorine, ammonia, phosphate, etc.

Calculations Capabilities

The user can define many functions with the sc1000 controller that is necessary to complete useful calculations. This value is displayed as a probe reading and can be output and/or used to control processes. Each calculated value takes up one 4-20 mA signal.

Flexible Input/Output Options

Communication and relay options for the sc1000 controller can be configured to suit any situation. Available configurations for a single Probe Module include:

- *up to 4 relay contacts for alarm and control functions,*
- *up to 12 analog outputs for measured values,*
- *up to 12 analog or digital inputs from non-digital sensors (i.e., flow or pressure sensors),*
- *up to 8 digital inputs from digital sensors,*
- *select from multiple network card options such as MODBUS®, PROFIBUS DP, and more. (Contact your Hach representative for available network options.)*

Additional relays and analog inputs/outputs can be added by networking additional Probe Modules or optional DIN-rail communication modules.

Advanced Communication Options

Use the sc1000 controller for hassle-free communications. Advanced communication features include:

- *Ethernet port (standard)—attach a computer directly to the sc1000 controller to operate the system from the computer. Download data logs and upload software updates.*
- *GSM wireless modem (optional; SIMM card purchased separately)—integrated into the Display Module. Use it for remote operation of the controller and sensors, including data log transfer and software updates. Text message errors, warnings, and process events can be sent to up to four user-selectable recipients and a Hach Company service number. (FCC and IC approved.)*

Digital Reliability and Integration

Digital signals between the sc1000 controller and attached sensors assure data integrity and immunity from signal interference. Digital outputs from the sc1000 make it easy to integrate the controller into an existing network.

Expandable and Upgradeable

The Hach sc1000 Controller can adapt to your needs. Change probes without changing the controller. With a single Display Module, additional Probe Modules can be added or removed depending on operational needs. Fully upgradeable software ensures that this system will not be obsolete. Hach service plans are available.

Specifications*

Ambient Conditions
Operation: -20 to 55°C (-4 to 131°F);
0 to 95% relative humidity, non-condensing

Storage: -20 to 70°C (-4 to 158°F);
0 to 95% relative humidity, non-condensing

Power Requirements
100 to 230 Vac, 50/60 Hz

Power: 75 W

Optional: 24 Vdc

Display
1/4 VGA graphical backlit TFT color touch screen

Size: 11.4 x 8.6 cm (4.5 x 3.4 in.)

Resolution: 320 x 240 pixels

Relays
Up to four SPDT, user-configurable contacts rated 100 to 230 Vac, 5 Amp resistive maximum, per probe module. Additional relays are available with additional probe modules.

Outputs
Up to 12 analog 0/4-20 mA, maximum impedance 500 Ohms per probe module.

Additional analog outputs with additional probe modules.

Optional digital communications via MODBUS® (RS-485) or PROFIBUS DP.

Inputs
Up to 12 analog 0-20 mA, maximum impedance 500 Ohms per probe module.

Additional inputs are available with additional probe modules.

Control
PID, high/low phasing, setpoint, deadband, overfeed timer, off delay, and on delay

Alarms
Low alarm point, low alarm point deadband, high alarm point, high alarm point deadband, off delay, and on delay

Communication (Optional)
MODBUS® (RS-485): Advanced communications/networking with PLC or SCADA system directly from analyzer.

PROFIBUS DP

GSM cellular module (FCC and IC approved)

Ethernet service port (standard)

Memory Backup
All user settings are retained indefinitely in memory (non-volatile) (EEPROM)

Mounting Configurations
Surface, panel, and pipe (horizontal and vertical)

Optional sun shield

Enclosure
IP65; ABS (display module) and metal (probe module) enclosure with corrosion-resistant finish

Dimensions
Probe module with attached display module:
315 x 250 x 142 mm (12.4 x 9.8 x 5.6 in.)

Weight
Approximately 6.5 kg (14.3 lbs.) depending on configuration

Certifications
North American Certifications:
cTUVus to UL 61010A-1 and CSA C22.2 No. 1010.1
FCC ID QIPMC56 / IC ID 267W-MC56

European Certifications:
CE per 73/23/EEC and 89/336/EEC
TUV-GS to EN 61010-1
EN 61326 Amd's 1 & 2

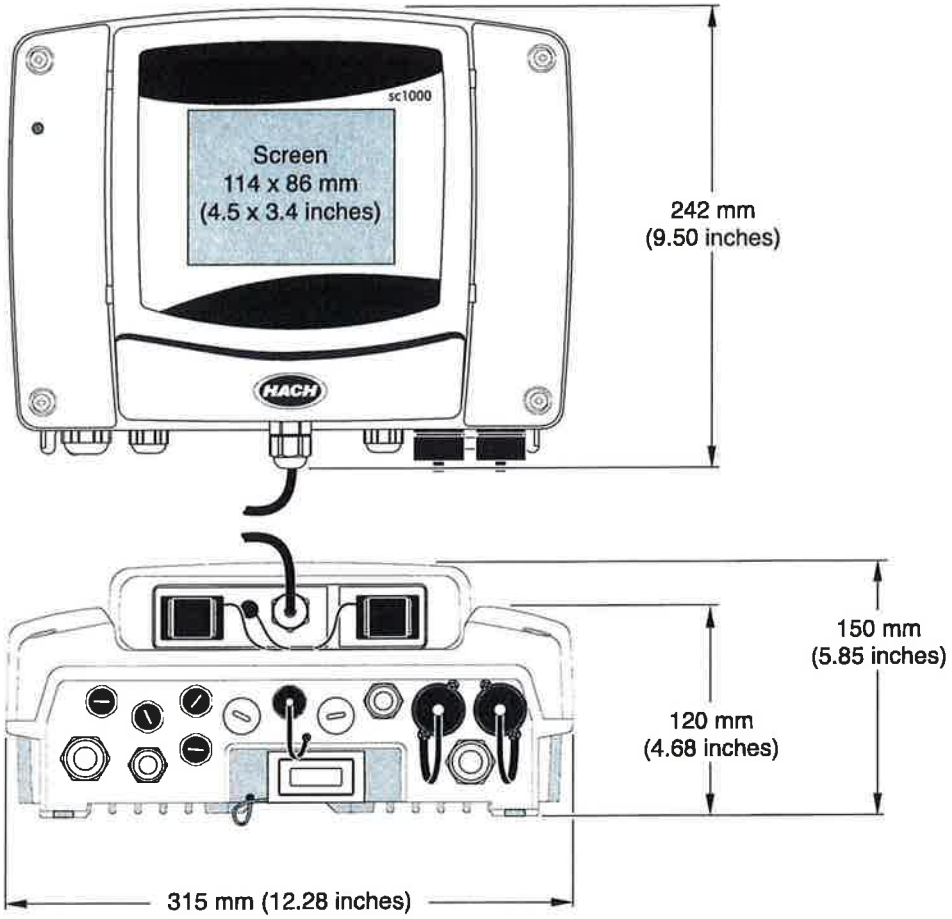
**Specifications subject to change without notice.*

Engineering Specifications

1. The controller shall consist of a portable display module connected to one or more probe modules.
2. The portable display module shall have a color graphical touch-screen display.
3. Connections between the sensors and the controller shall be "plug and play."
4. The controller shall have the option for MODBUS or PROFIBUS DP communication.
5. The interface unit shall allow operators to control sensor and interface functions with menu-driven software.
6. The controller shall have up to four potential free relays, twelve analog outputs and twelve analog inputs per probe module (all expandable via the controller's internal network).
7. The controller shall have an Ethernet service port for direct connection to a personal computer for transfer of data and software updates.
8. The controller shall be housed in an IP65 enclosure. An optional sun shield shall be available.
9. The controller shall be mounted horizontally or vertically on surface or pipe. An optional sun shield shall be available.
10. The AC power supply shall be housed in the interface unit and automatically accept input in the range of 100 to 230 Vac, 50/60 Hz. A 24 Vdc probe module shall be available as an option.
11. All system components shall be certified as follows. North American Certifications: cTUVus to UL 61010A-1 and CSA C22.2 No. 1010.1; FCC ID QIPMC56 / IC ID 267W-MC56. European Certifications: CE per 73/23/EEC and 89/336/EEC; TUV-GS to EN 61010-1; EN 61326 Amd's 1 & 2.
12. The controller shall be warranted for 12 months against defects in material and workmanship.
13. The controller shall be Hach Company Model sc1000 Multiparameter Universal Controller.

Dimensions

The sc1000 controller unit can be installed on a pole, wall, or a floor stand. No tools are needed to connect the controller unit to any Hach digital sensor.



Ordering Information

LXV402.99.00002 sc1000 Display Module

LXV402.99.01002 sc1000 Display Module with GSM

LXV400.99.1R572 sc1000 Probe Module, 4 Sensors, 4 mA Out, 4 mA In, 4 Relays, 110-230V

LXV400.99.1B572 sc1000 Probe Module, 4 Sensors, 4 mA Out, 4 mA In, 4 Relays, RS-485 (MODBUS), 110-230V

LXV400.99.1F572 sc1000 Probe Module, 4 Sensors, 4 mA Out, 4 mA In, 4 Relays, PROFIBUS DP, 110-230V

LXV400.99.1R582 sc1000 Probe Module, 6 Sensors, 4 mA Out, 4 mA In, 4 Relays, 110-230V

Additional combinations are available. Contact your Hach representative or call 1-800-227-4224 for more information.

Power Cords

5448800 Power Cord with strain relief, 125 Vac, American-style plug

5448900 Power Cord with strain relief, 230 Vac, European-style plug

Accessories

6169900 sc1000 Panel Mount Kit

LZX958 Sun Shield, for sc1000 controller

LZX918 sc1000 internal network connector

LZX988 sc1000 internal network cable, 100 m (328 ft.)

LZX989 sc1000 internal network cable, 500 m (1640 ft.)

To complete your digital measurement system, choose from Hach's family of digital products...



sc100™ Controller

Plug-and-play, mix-and-match operation for one or two sensors
(see Lit. # 2463)

Hach LDO™ Dissolved Oxygen Probe

Break-through luminescent technology for dissolved oxygen
(see Lit. # 2455)

Model 5740 sc Galvanic Membrane Dissolved Oxygen Sensor

Replaceable membrane cartridge for simple maintenance
(see Lit. # 2469)

Differential pH and ORP Sensors

Three electrodes for increased measurement accuracy
(see Lit. # 2467)

3/4-inch Combination pH and ORP Sensor Kits

Designed specifically for immersion or in-line mounting
(see Lit. # 2470)

Inductive Conductivity Sensors

Innovative technology for harsh environments
(see Lit. # 2465)

Contacting Conductivity Sensors

Enhanced performance sensors for a variety of applications
(see Lit. # 2468)

1720E Low Range Turbidimeter

Meets performance criteria established by the USEPA for regulatory reporting
(see Lit. # 2457)

SOLITAX® sc Turbidity and Suspended Solids Sensors

Accurate, color-independent measurement in any application
(see Lit. # 2472)

NITRATAX™ UV Nitrate Sensors

Simple and accurate technology for low cost of operation
(see Lit. # 2464)

At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

For current price information, technical support, and ordering assistance, contact the Hach office or distributor serving your area.

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In the interest of improving and updating its equipment, Hach Company reserves the right to alter specifications to equipment at any time.

HACH®

Be Right™

Hach sc200™ Universal Controller



Choose from up to 29 digital or analog sensors for up to 15 different parameters.

Product Overview

One Controller for the Broadest Range of Sensors

The sc200 Universal Controller is the most versatile controller on the market. The new sc200 controller is the only controller that allows the use of digital and analog sensors, either alone or in combination, to provide compatibility with the broadest range of sensors. It replaces the Hach sc100 digital and GLI53 analog controllers with advanced features for easier operator use.

The sc200 controller platform can be configured to operate either 2 Digital Sensor Inputs, or 1 or 2 Analog Sensor Inputs, or a combination of Digital and Analog Sensor Inputs. Customers may choose their communication options from a variety of offerings ranging from MODBUS RTU to Profibus DPV1.

Features and Benefits

Maximum Versatility

- *Standardized controller eliminates the need for a variety of dedicated controllers*
- *Multi-channel controller operates either 1 or 2 sensors reducing inventory holding costs and providing an inexpensive option to add a second sensor at a later time*
- *"Plug and Play" operation with all Hach digital sensors*
- *True dual sensor controller provides 4-20 mA outputs to transmit primary and secondary measurement values*

Ease of Use and Confidence in Results

- *New display and guided calibration procedures reduce operator error*
- *Password protected SD card reader offers a simple solution for data download and transfer*
- *Visual warning system provides critical alerts*



Communication Options

- *MODBUS RS232/RS485 or Profibus DPV1*

Controller Configuration	Functionality
2 Channel Digital Controller	Maximum versatility and flexibility: • Plug and play with all Hach digital sensors • Mix and match with Hach digital and GLI analog sensors
2 Channel Controller with 1 Analog and 1 Digital Sensor Input	• Plug and play with any one Hach digital sensor • Mix and match with any one GLI analog sensor
1 or 2 Channel Analog Controller	• Mix and match up to two GLI analog sensors

DW PW

DW = drinking water WW = wastewater municipal PW = pure water / power
IW = industrial water E = environmental C = collections FB = food and beverage

HACH®
Be Right™

Controller Comparison







Features	Current sc100™ Controller	GLI53 Controller	NEW! sc200™ Controller	Benefits
Display	64 x 128 pixels 33 x 66 mm (1.3 x 2.6 in.)	64 x 128 pixels 33 x 66 mm (1.3 x 2.6 in.)	160 x 240 pixels 48 x 68 mm (1.89 x 2.67 in.) Transreflective	• Improved user interface—50% bigger • Easier to read in daylight and sunlight
Data Management	irDA Port/PDA Service Cable	N/A	SD Card Service Cable	• Simplifies data transfer • Standardized accessories/ max compatibility
Sensor Inputs	2 Max Direct Digital Analog via External Gateway	2 Max Analog Depending on Parameter	2 Max Digital and/or Analog with Sensor Card	• Simplifies analog sensor connections • Works with GLI and Hach's digital sensors
Analog Inputs	N/A	N/A	1 Analog Input Signal Analog 4-20mA Card	• Enables non-sc analyzer monitoring • Accepts mA signals from other analyzers for local display • Consolidates analog mA signals to a digital output
4-20 mA Outputs	2 Standard	2 Standard	2 Standard Optional 4 Additional	• Total of six (6) 4-20 mA outputs enables up to 3 mA outputs per sensor input
Digital Communication	MODBUS 232/485 Profibus DP V1.0	N/A	MODBUS 232/485 Profibus DP V1.0	• Unprecedented combination of sensor breadth and digital communication options

To complete your measurement system, choose from Hach's portfolio of controller and sensor products...



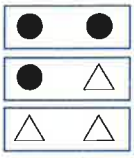
Choose from Hach's Broad Range of Digital and Analog Sensors

Parameter	Sensor	Digital or Analog
Ammonia	AMTAX™ sc, NH4D sc	●
Chlorine	CLF10 sc, CLT10 sc, 9184 sc	●
Chlorine Dioxide	9185 sc	●
Conductivity	3400, 3700	△
Dissolved Oxygen	LDO™, 5740 sc	●
Dissolved Oxygen	5500	△
Flow	U53, F53 Sensors	△
Nitrate	NITRATAX™ sc, NO3D sc	●
Oil in Water	FP360 sc	●
Organics	UVAS sc	●
Ozone	9187 sc	●
pH/ORP	pHD	●
pH/ORP	pHD, pH Combination, LCP	△
Phosphate	PHOSPHAX™ sc	●
Sludge Level	SONATAX™ sc	●
Suspended Solids	SOLITAX™ sc, TSS sc	●
Turbidity	1720E, FT660 sc, SS7 sc, ULTRATURB sc, SOLITAX sc	●

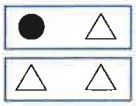
● = Digital △ = Analog

The diagrams below demonstrate the versatility and flexibility for the base controller units. Connect any of the appropriate sensors listed above to meet your measurement needs. Operation of analog sensors require the controller to be equipped with the appropriate sensor card.

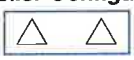
2 Channel Digital Controller Configurations



2 Channel Controller with 1 Analog and 1 Digital Sensor Input Configurations



2 Channel Analog Controller Configurations



Engineering Specifications

The controller shall be a microprocessor based instrument.

2. The enclosure shall be 1/2 DIN format, NEMA4X rated for wall, pole and panel mounting.
3. The controller shall offer both 100–240 Vac 50/60 Hz and 24 Vdc power options.
4. The controller shall offer two analog 0/4-20 mA output signals with independent PID control functions and optional four additional 4-20 mA outputs.
5. The controller shall accept either Digital Sensors or Sensor Modules for pH, Conductivity, DO, Paddle Wheel Flow, and Ultra Sonic Flow sensors.
6. The controller shall have single channel and dual channel options.
7. The controller shall have options for MODBUS RS232, MODBUS RS485 and Profibus DPV1 communication.
8. The display contrast shall be adjustable.
9. The Menu shall be available in at least 19 different languages.
10. The controller shall have 2 Data logs, 128 kb each. The logged data shall be downloadable on a SD card in XML format.
11. The controller shall be Hach Company sc200 Universal Controller.

Specifications*

sc200 General Specifications

Display
Graphic dot matrix LCD with LED backlighting
Transreflective

Display Size
48 x 68 mm (1.89 x 2.67 in.)

Display Resolution
240 x 160 pixels

Height x Width x Depth
144 x 144 x 181 mm (5.7 x 5.7 x 7.1 in.)

Weight
1.70 kg (3.75 lb)

Power Requirements
100 – 240 Vac ±10%, 50/60 Hz
24 Vdc -15% + 20%

Operating Temperature
-20 to 60°C (-4 to 140°F), 0 to 95% RH non-condensing

Storage Temperature
-20 to 70°C (-4 to 158°F), 0 to 95% RH non-condensing

Analog Output Signal
Two 0/4 to 20 mA isolated current outputs, max 500Ω

Operational Mode
Primary or secondary measurement or calculated value (dual channel only)

Functional Mode
Linear, Logarithmic, Bi-linear, PID

Optional 4 additional 4/20 mA isolated current outputs, max 500Ω @ 18-24 Vdc (customer-supplied power source)

Security Levels
Two password protected levels

Enclosure Materials
Polycarbonate, Aluminum (powder coated), Stainless Steel

Mounting Configurations
Wall, pole and panel mounting

Enclosure Rating
NEMA4X / IP66

Conduit Openings
1/2" NPT Conduit

Relays
Four electromechanical SPDT (Form C) contacts, 1200W, 5 A, 250 Vac

Operational Mode
Primary or secondary measurement, calculated value (dual channel only) or timer

Functional Mode
Alarm, Timer, Feeder Control, PWM or FM Control, System Alarm

Digital Communication
MODBUS RS232/RS485, Profibus DPV1 optional

Memory Backup
Flash memory

Electrical Certifications
EMC: Certified CE compliant for conducted and radiated emissions (EN 50081-2) and immunity (EN 61000-6-2)

General Purpose: UL through ETL

Class I, Div. 2 (Groups A, B, C, and D): CSA (pending)

sc200 for Hach Analog pH/ORP Sensors

Measuring Range
-2.0 to 14.0 pH or -2.00 to 14.00 pH
- 2,100 to 2,100 mV

Repeatability
±0.1% of range

Response Time
0.5 s

Temperature Range
PT100/PT1000: -20 to 200°C (-4 to 392°F)
NTC300: -20 to 110°C (-4 to 230°F)
Manual: -25 to 400°C (-13 to 752°F)

Temperature Accuracy
±0.5°C (0.9°F)

Temperature Drift
±0.03% of reading /°C

Temperature Compensation
Automatic from -20 to 110°C (-4 to 230°F) or manual

Temperature Sensors
PT100/PT1000/NTC300

Temperature Compensation Curves
Nernst, for Pure Water: Ammonia, Morpholine, User Defined (linear)

Sensor-to-Controller Distance (maximum)
pHD or LCP sensor: 914 m (3000 ft.)
pH Combination electrode w/ preamplifier: 300 m (958 ft.)
pH Combination electrode w/o preamplifier: 30 m (100 ft.), depending on environment this distance is shorter

Calibration Methods
2-point buffer (pH only)
1-point buffer (pH only)
2-point sample (pH only)
1-point sample (pH or ORP)

Specifications *continued*

sc200 for Hach Analog Contacting Conductivity Sensors

Measuring Range
Conductivity
µS/cm: 0-2.000, 0-20.00, 0-200.0 or 0-2,000
mS/cm: 0-2.000, 0-20.00 or 0-200.0

Resistivity
0-19.99 MΩ•cm or 0-999.9 kΩ•cm

TDS
0-9999 ppm or 0-9999 ppb

Repeatability, Precision (0-20 µS/cm, K=1)
±0.02 mS/cm

Repeatability (20-200,000 µS/cm, K=1)
±0.1% of reading

Response Time
0.5 s

Temperature Range
-20 to 200°C (-4 to 392°F)

Temperature Accuracy
±0.5°C (0.9°F)

Temperature Drift
> 20 µS/cm: ±0.02% of reading / °C
< 20 µS/cm: ±0.004 µS/cm

Temperature Compensation
Automatic from -20 to 200°C (-4 to 392°F) or manual

Temperature Sensor
PT100/PT1000

Temperature Compensation Curves
Linear, Ammonia, Natural water, User Defined, none

Sensor-to-Controller Distance (max)
91m (300 ft.)

Calibration Methods
Zero
GLI DRY-CAL
1-point sample

sc200 for Hach Analog Inductive Conductivity Sensors

Measuring Range
Conductivity
µS/cm: 0-200.0 or 0-2,000
mS/cm: 0-2.000, 0-20.00, 0-200.0 or 0-2,000
S/cm: 0-2.000

% Concentration
0-99.99% or 0-200.0%

TDS
0-9999 ppm repeatability

Repeatability > 500 µS/cm
±0.5% of reading

Repeatability < 500 µS/cm
±2.5 µS/cm

Response Time
1 s

Temperature Range
-20 to 200°C (-4 to 392°F)

Temperature Accuracy
±0.5°C (0.9°F)

Temperature Drift
> 500 µS/cm: ±0.02% of reading / °C
< 500 µS/cm: ±0.1 µS/cm

Temperature Compensation
Automatic from -20 to 200°C (-4 to 392°F) or manual

Temperature Sensors
PT1000

Temperature Compensation Curves
Linear, Natural water, User Defined, none**

Concentration Curves
H_3PO_4: 0-40%; HCl: 0-18%; HCl: 22-36%; NaOH: 0-16%; $CaCl_2$: 0-22%; HNO_3: 0-28%; HNO_3: 36-96%; H_2SO_4: 0-30%; H_2SO_4: 40-80%

Sensor-to-Controller Distance

Full-scale value	*Maximum length*
200 to 2,000 µS/cm	61m (200 ft.)
2,000–2,000,000 µS/cm	91m (300 ft.)

Calibration Methods
1-point Cond (or Concentration or TDS)
Zero

***Available curves depend on the selected type of measurement (Conductivity, Concentration or TDS).*

Continued on next page.

Specifications *continued*

Linearity 3700 Inductive Conductivity Sensors

1.5 mS/cm – 2 S/cm
1% or reading

< 1.5 mS/cm
±15 µS/cm

Linearity 3700 Inductive Conductivity Sensors with Multiple Point Calibration

1.5 mS/cm – 2 S/cm
0.5% or reading

< 1.5 mS/cm
±5 µS/cm

sc200 for Hach Analog Dissolved Oxygen Sensors

Measuring Range
0 to 40 ppm
200% saturation

Repeatability
±0.05% of range

Response Time
0.5 s

Temperature Range
0 to 50°C (32 to 122°F)

Temperature Accuracy
±0.5°C (0.9°F)

Temperature Drift
±0.02% of reading / °C

Temperature Compensation
Automatic from 0 to 40 ppm or manual

Temperature Sensor
NTC30K / Manual

Sensor-to-Controller Distance (max)
305 m (1000 ft.)

Calibration Methods
Sample
Air
Saturation

sc200 for Hach UltraSonic Flow Sensor

Flow Rate
0-9999, 0-999.9, 0-99.99 with selectable flow rate units and multiplier

Volume
0-9,999,999 with selectable volume units

Depth
0-1200.0 inches, 0-100.0 feet, 0-30,000 mm, or 0-30.00 meters

Input Filter
999 sec

Totalizers
8-digit resettable LCD software totalizer

Totalized Flow
Gal., $ft.^3$, acre-ft., lit., m^3

Repeatability
±0.1% of span

Sensor-to-Controller Distance (max)
100 m (328 ft.)

Calibration Methods
Cal Depth 1 point
Cal Depth 2 point

sc200 for Hach Paddle Wheel Flow Sensor

Flow Rate
Function of Structure Type: 0-9999, 0-999.9, 0-99.99 with selectable flow rate units and multiplier

Volume
0-9,999,999 with selectable volume units

Input Filter
999 sec

Totalizers
8-digit resettable LCD software totalizer

Totalized Flow
Gal., $ft.^3$, acre-ft., lit., m^3

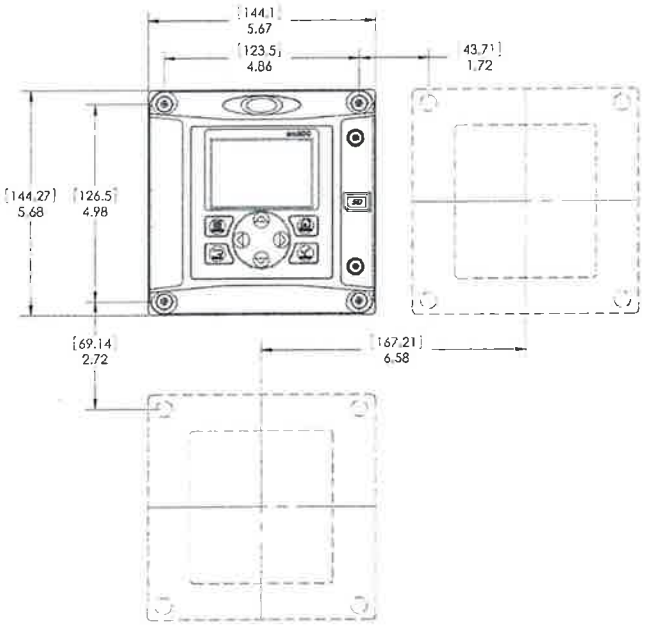
Sensor-to-Controller Distance (max)
GLI impeller Sensors: 610m (2000 ft.)
Non-GLI Sensors: 91m (300 ft.)

**Specifications subject to change without notice.*

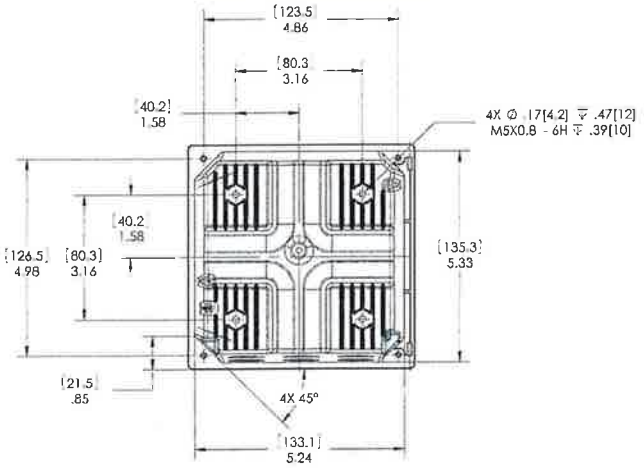
Dimensions

The sc200 controller unit can be installed on a surface, panel, or pipe (horizontally or vertically). No tools are needed to connect the controller unit to any Hach digital sensor. NOTE: Dimensions are in inches [millimeters].

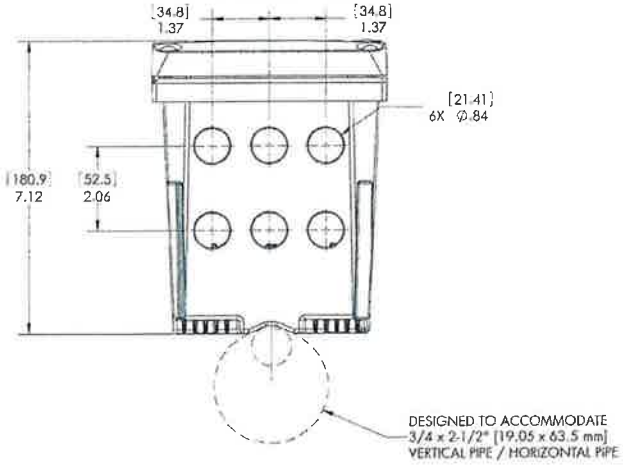
Minimum Spacing Dimensions for Group Mounting



Mounting Dimensions



Bottom View



Ordering Information

sc200 for Hach Digital Sensors

LXV404.99.00552	sc200 controller, 2 channel, digital
LXV404.99.00502	sc200 controller, 1 channel, digital
LXV404.99.00542	sc200 controller, 2 channel, digital & mA input
LXV404.99.00512	sc200 controller, 2 channel, digital & pH/DO
LXV404.99.00522	sc200 controller, 2 channel, digital & Conductivity
LXV404.99.00532	sc200 controller, 2 channel, digital & Flow

sc200 for Hach Analog Sensors

LXV404.99.00102	sc200 controller, 1 channel, pH/DO
LXV404.99.00112	sc200 controller, 2 channel, pH/DO
LXV404.99.00202	sc200 controller, 1 channel, Conductivity
LXV404.99.00222	sc200 controller, 2 channel, Conductivity
LXV404.99.00212	sc200 controller, 2 channel, pH/DO & Conductivity
LXV404.99.00302	sc200 controller, 1 channel, Flow
LXV404.99.00332	sc200 controller, 2 channel, Flow
LXV404.99.00312	sc200 controller, 2 channel, Flow & pH/DO
LXV404.99.00322	sc200 controller, 2 channel, Flow & Conductivity

Note: Other Sensor combinations are available. Please contact Hach Technical Support or your Hach representative.

Note: Communication options (MODBUS and Profibus DPV1) are available. Please contact Hach Technical Support or your Hach representative.

Power Cords

9202900	sc200 power cord with strain relief, 125 Vac
9203000	sc200 power cord with strain relief, 230 Vac, European-style plug

Accessories

9220600	sc200 Weather and Sun Shield with UV Protection Screen
8809200	sc200 UV Protection Screen
1000G3088-001	Weather Protection Cover
9218200	SD card reader (USB) for connection to PC
9218100	4 GB SD card

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At Hach, it's about learning from our customers and providing the right answers. It's more than ensuring the quality of water—it's about ensuring the quality of life. When it comes to the things that touch our lives...

Keep it pure.

Make it simple.

Be right.

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