City of Woonsocket, Rhode Island
Regional Wastewater Treatment Facility

FACILITY PLAN AMENDMENT

May 2013

CH2MHILL

and

WRIGHT-PIERCE
Engineering a Better Environment

CITY OF WOONSOCKET, RHODE ISLAND
REGIONAL WASTEWATER TREATMENT FACILITY

FACILITY PLAN AMENDMENT

FINAL

May 2013

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This Facility Plan was originally prepared by Wright-Pierce, March 2011, and portions have been superseded by CH2M HILL, May 2013. CH2M HILL endorses this Facility Plan as amended with the stipulation that CH2M HILL did not have access to the original electronic files (the March 2011 version) and cannot verify all analysis and supporting graphics as provided by others.

CITY OF WOONSOCKET, RHODE ISLAND
WOONSOCKET REGIONAL WASTEWATER TREATMENT FACILITY
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EXECUTIVE SUMMARY

ES.1 INTRODUCTION

The City of Woonsocket, Rhode Island is responsible for an extensive wastewater collection system and regional treatment facility that also accepts flow from the neighboring communities of Bellingham and Blackstone, Massachusetts, and North Smithfield, Rhode Island. These three client communities discharge to the Woonsocket Regional Wastewater Treatment Facility (WWTF) under intermunicipal agreements with the City of Woonsocket. The City of Woonsocket is the holder of the discharge permit issued under the Rhode Island Pollution Discharge Elimination System (RIPDES). The treated effluent from the WWTF is discharged into the Blackstone River.

In June 2012, the City contracted with CH2M HILL to enter into an Operate-Design-Build-Operate (O-DBO) contract to operate, maintain, design and build the nutrient removal upgrades to the Regional Wastewater Treatment Facility. Under the new contract, CH2M HILL took over operations of the wastewater treatment facility on October 1, 2012. CH2M HILL is responsible for permit compliance with the facility's RIPDES permit.

The last significant upgrades to the facility were constructed between 2001 and 2004 in accordance with an approved Rhode Island Department of Environmental Management (RIDEM) Facility Plan Amendment completed in February 2000. These upgrades and supplemental improvement projects were completed to achieve the nitrogen and phosphorus removal requirements of the 2000 RIPDES discharge permit.

The RIDEM recently issued a revised Rhode Island Pollutant Discharge Elimination System permit (No. RI 0100111, issued October 1, 2008) which requires more stringent water quality discharge requirements for the facility, specifically lower limits for both the facility's seasonal effluent total nitrogen and total phosphorus concentrations. Under this new permit the facility is now required to meet a seasonal total effluent nitrogen concentration of 3.0 mg/L between May 1 and October 31, and a total effluent phosphorus concentration of 0.1 mg/ L between April 1 and October 31 along with a 1.0 mg/L effluent limit from November 1 to March 31. The facility's permit has interim seasonal limits of 10 mg/L and 1.0 mg/L for nitrogen and phosphorus respectively. RIPDES permit modifications are discussed in detail in Section 1 – Introduction.

In order to address these new permit modifications, the City entered into a Consent Agreement (RIA-368) with the RIDEM (finalized and signed February 2011) agreeing to submit a Facilities Plan Amendment to develop a proposed solution and implementation schedule for necessary improvements to achieve compliance with the new effluent total nitrogen and total phosphorus limits. The Consent Agreement established a compliance schedule for completing the Facility Plan Amendment along with the design and implementation schedule for construction of the necessary improvements. A letter, dated March 23, 2012, from RIDEM to the City revised some of the dates in the consent decree and provided guidance on the construction schedule. Although the WWTF operates very well and satisfies the design objectives and regulated water quality requirements imposed at the time of the previous upgrades, the 2000 facility improvements were not designed to meet the new low level seasonal effluent limits for total nitrogen and total phosphorus.

ES.2 PURPOSE

This document is prepared to serve as an Amendment to the City of Woonsocket's 2000 RIDEM approved Facility Plan Amendment. This Amendment summarize the various system improvements that were investigated and outlines the recommended improvements that should be implemented to ensure compliance with the facility's new RIPDES permit. This Facility Plan Amendment supplements and updates the City's existing Facility Plan Amendment, which was submitted by US Filter (Veolia) and the Maguire Group, Inc. in February 2000.

The overall purpose of this Facility Plan Amendment includes the following:

- Document the current conditions of the WWTF's equipment systems, and facilities;
- Develop projected future flows and loads for the 20-year planning period (Design Year 2030);
- Evaluate alternatives and develop recommendations to address the immediate nutrient removal requirements for effluent nitrogen and phosphorus for the 20-year planning period;

- Evaluate other WWTF equipment, systems and facility upgrade needs required for the 20-year planning period;
- Provide an opinion of probable capital and O&M costs for recommended treatment system improvements and long-term upgrades, and;
- Develop an implementation plan and schedule for the required facility upgrades to achieve compliance with the new RIPDES requirements for enhanced nutrient removal.

ES.3 REPORT FORMAT

The Facility Plan Amendment is presented in ten report sections and five appendices. The ten sections present the existing and future WWTF conditions, analyze current secondary treatment system operation based on process modeling, evaluate alternatives to achieve the immediate enhanced nutrient removal needs, and recommend various improvements to allow for the efficient reliable operation of the facility for the next 20 years. The appendices include WWTF design data, the current RIPDES Permit, a summary of the City's Inflow and Infiltration Removal Program, the Public Participation Workshop Presentation, the RIDEM Facility Plan Checklist, and Intergovernmental Review correspondence.

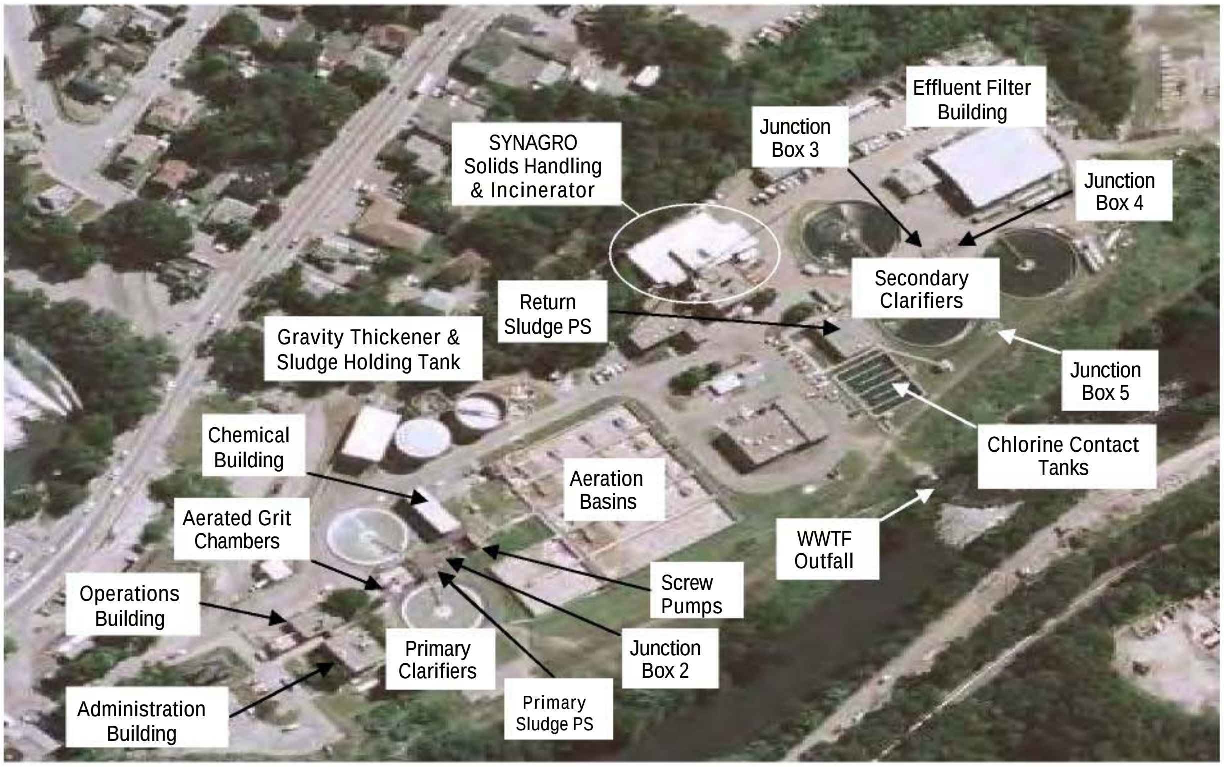
ES.4 SECTION 1 – INTRODUCTION

This section presents background information, objectives of the facility planning process, history and description of the Woonsocket Regional WWTF, description of need for improvements, and presentation of the modified RIPDES discharge permit limits. An outline of the Facility Plan Amendment and its contents is also provided.

ES.4.1 Historical Overview of the Facility

The original treatment facility was constructed in 1897 and has been modified numerous times throughout the years. The first modern era upgrade was completed in 1930. The current WWTF layout and treatment processes bear little resemblance to the plant that existed in the 1930's. In 1962, a major upgrade to the WWTF was completed and included construction of the Operations Building, where the main influent wetwell is located. The next major upgrade to the WWTF was completed in 1977. Based on recommendations in the last Facility Plan Update in February 2000, a series of design-build facility upgrades were implemented in the early 2000's to provide the capacity to meet a seasonal total nitrogen limit of 10 mg/L and a total phosphorus limit of 1 mg/L. Figure ES-1 shows an aerial view of the Woonsocket WWTF with the major buildings and processes highlighted. A summary of basic design data for the existing facility is included as Appendix A.

FIGURE ES-1
WWTF AERIAL VIEW



ES.5 SECTION 2 – EXISTING FLOWS AND LOADS

This section presents a summary of the existing flows and loads in the planning area based on historic operating data from 2007 to 2010, supplemented with additional wastewater characterization data from 2012 (see Appendix G, Current Plant Date) with special emphasis on the characteristics of the raw influent, recycle, primary clarifier influent, and primary effluent flows and loads, specifically BOD, TSS, nitrogen, phosphorus, temperature, and alkalinity. Additionally, loads were allocated for industrial facilities to develop and support the Local Limits Analysis. The primary effluent flows and loads have a direct impact on the evaluation and design of the secondary system process improvements that are needed to meet the revised nitrogen and phosphorus limits.

While developing this Facility Plan Amendment, CH2M HILL and Wright-Pierce worked with City representatives to review historic operations at the WWTF. Current flow and loading conditions were developed based on analysis of historic WWTF operating data over a three year period. The data was used to benchmark current conditions at the WWTF relative to the facility's design capacity of 16 mgd (maximum month): 24 mgd (maximum day) and 32 mgd (peak hour). Table ES-1 summarizes the existing raw influent flows and loads from May 2007 through September 2010, excluding the March 30, 2010 flooding events (see Section 2 for detailed discussion). Table ES-2 provides data for primary effluent flows and loads.

TABLE ES-1
RAW INFLUENT FLOWS AND LOADS
MAY 2007 – SEPTEMBER 2010

Constituent	Average Day	Maximum Month	Maximum Day
Raw Influent Flow, mgd	7.8	13[1]	22.2[1]
TSS (lbs/day)	11,600	18,800	39,200
BOD (lbs/day)	14,200	27,100	51,300
TKN (lbs/day)	1,800	--	3,629
Ammonia-N (lbs/day)	1,146	--	1,997
Total Phosphorus (lbs/day)	237	--	368

Note:

[1]The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data for the raw influent indicates a maximum monthly flow of 15.8 mgd, a peak daily flow of 29.1 mgd, and a peak hourly flow of approximately 38 mgd

TABLE ES-2
PRIMARY EFFLUENT FLOWS AND LOADS
MAY 2007 – SEPTEMBER 2010

Constituent	Average Day	Maximum Month	Maximum Day
Primary Effluent Flow, mgd	11.7	18.1[1]	26.2[1]
TSS (lbs/day)	7,400	12,700	28,500
BOD (lbs/day)	13,900	28,900	48,200
TKN (lbs/day)	2,415	5,188	6,925
Ammonia-N (lbs/day)	1,231	2,091	2,874

Constituent	Average Day	Maximum Month	Maximum Day
Total Phosphorus (lbs/day)	677	1,363	1,958

Note:

[1]The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data for the primary effluent indicates a maximum monthly flow of 22.3 mgd, a peak daily flow of 34.6 mgd, and a peak hourly flow of approximately 42 mgd.

ES.6 SECTION 3 – FUTURE DESIGN FLOWS AND LOADS

This section presents a summary of the projected future design flow and loading conditions, which are the basis for evaluating alternatives for the nutrient removal systems.

Based on consultation and input from City representatives and review of available U.S. Census, State Guide Plans (SGP), and Community Comprehensive Plans (CCPs) it is anticipated there will be minimal population growth within the planning area over the next twenty (20) year planning period. Specifically, published U.S. Census information predicts 2030 population changes of approximately:

TABLE ES-3
POPULATION ESTIMATES AND PROJECTIONS

Community	U.S. Census 2000 Population[a]	U.S. Census 2010 Population[b]	% Increase or Decrease from 2000 to 2010[c]	R.I./MA./U.S. Census 2030 Population Projection[a]
Woonsocket	43,224	41,186	-4.7%	40,772
North Smithfield	10,618	11,967	+12.7%	11,207
Bellingham	15,314	16,332	+6.6%	16,642
Blackstone	8,804	9,026	+2.5%	9,852
Totals	77,960[d]	78,511[d]	+0.7 %	78,473[d]

Notes:

[a] U.S. Census 2000; Rhode Island State Planning Program Technical Paper 154, August 2004; and Metropolitan Area Planning Council, January 2006.

[b] U.S. Census Population Estimate2010 – Population Finder website recently released.

[c] % Population change from 2000 to 2010

[d] Population figures are not for the WWTF service area but rather for the entire communities.

Based on the above, it is anticipated there will be relatively small population growth changes over the 20 year planning period. The future increase in residential and commercial flows and loads will likely be expected at similar levels.

It is recognized that the City of Woonsocket also needs to maintain a certain level of reserve capacity for future industrial flows and loads. Accordingly, a modest reserve capacity allotment for future industrial flows and loads is included as an allowance, which is consistent with the City's goals for economic development.

Table ES-4 summarizes the average, maximum monthly, peak daily, and peak hourly flows through the WWTF from May 2007 through September 2010, excluding the March 30, 2010 flooding events along with the projected 2030 design flows.

TABLE ES-4
SUMMARY OF CURRENT AND PROJECTED YEAR 2030 FLOWS

Item	Existing Design 2000	Current 2010[a]	Updated Design 2030
Raw Influent Flows (MGD):			
Average Annual	16	7.8	9.0
Maximum Monthly	16	13.0[b]	16.0
Peak Daily	24	22.2[b]	24
Peak Hourly	32	~32[b]	32
Recycle Flows (MGD):			
Average Annual	2.6	3.9	3.9
Maximum Monthly		5.6	5.1
Peak Daily		8.5	5.1
Peak Hourly		-	5.1
Primary Inf./Eff. Flows (MGD):			
Average Annual	18.6	11.7	12.9
Maximum Monthly	18.6	18.1[c]	21.1
Peak Daily	26.6	26.2[c]	29.1
Peak Hourly	34.6	35[c]	37.1

Notes:
[a] Based on data from May 2007 through September 2010.
[b] The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data for the raw influent indicates a maximum monthly flow of 15.8 mgd, a peak daily flow of 29.1 mgd, and a peak hourly flow of approximately 38 mgd.

[c] The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data for the primary influent indicates a maximum monthly flow of 22.3 mgd, a peak daily flow of 34.6 mgd, and a peak hourly flow of approximately 42 mgd.

Section 3 concludes with a description of design criteria and the basis for evaluating candidate nutrient removal treatment systems for future conditions in the planning area.

ES.7 SECTION 4 and 4.A – PERFORMANCE OF SECONDARY TREATMENT FACILITIES AND PROCESS MODEL DEVELOPMENT

In this section, the WWTF's secondary treatment facilities were evaluated to identify their conditions and need for potential improvements or upgrades. Sections 4 and 4.A describe the findings of these evaluations based on historic plant operating data. These sections also presents a summary of the Bio Win™ and Pro2D process models and include an operational review of the WWTF's existing MLE activated sludge process. A description of the existing MLE process is presented along with a summary of existing operating data and options for optimizing nitrification and denitrification. Additionally, the process modeling and basis of design for the two-stage activated sludge AB process are presented.

ES.8 SECTION 5 – SCREENING OF NUTRIENT REMOVAL ALTERNATIVES

This section presents a preliminary screening of nutrient removal alternatives. Wastewater characterization is presented along with a discussion of the WWTF's ability to achieve the revised nutrient limits. Nutrient removal technologies are evaluated for both advanced nitrogen and phosphorus removal including budgetary opinions of probable costs for each alternative.

Given the nature of the facility's new RIPDES treatment limits and the size and complexity of the WWTF, only those processes with a proven track record (i.e., other full scale similar installations) were considered. To achieve the newly regulated limits, multiple barriers will be required and unit treatment processes in series will be required.

Preliminary opinions of probable cost estimates were prepared for each explored alternative to determine the general order of capital costs for the different alternative. Of the five process alternatives evaluated, four processes were recommended for further analysis as follows:

- 4-Stage Bardenpho with Integrated Fixed Film Activated Sludge (IFAS) (*for nitrogen removal*) followed by a ballasted flocculation process (*for phosphorus and solids removal*).

- MLE Process with tertiary Biological Anoxic Filter (BAF) (*for nitrogen removal*) followed by a ballasted flocculation process.
- MLE Process with tertiary Moving Bed Biological Reactor (MBBR) (*for nitrogen removal*) followed by a ballasted flocculation process.
- Two–Stage Activated Sludge AB Process followed by Traveling Bridge Sand Filters

The first three of the evaluated nitrogen removal systems will need to upgrade the WWTF's screening facilities to ensure the new tertiary treatment systems are capable of reliably meeting the facility's new permit limits. However, the fourth nitrogen removal system will not require a fine screening upgrade, but will include screening upgrades to increase reliability since the process does not include a new tertiary treatment process. Similarly, there is also need to upgrade various ancillary electrical services and equipment systems which are integral to all of the treatment alternatives.

ES.9 SECTION 6 – DETAILED EVALUATION OF NUTRIENT REMOVAL ALTERNATIVES

This section presents a more detailed evaluation of the short-listed nutrient removal alternatives outlined in Section 5. For each evaluated alternative, more detailed capital and operation and maintenance (O&M) cost, along with the 20-year life cycle costs comparison is presented. This section evaluates and ranks the selected alternatives.

Conceptual layouts of the four short-listed treatment alternatives are presented along with a summary of the basis-of-design information, advantages/disadvantages, site modification requirements, economic analysis and associated process recommendations based on the recommended design flow and loading information previously presented in Section 3.

Based on the treatment technology evaluations and associated life-cycle costs for each alternative to reliably achieve the effluent nitrogen and phosphorus limits, the Two-Stage Activated Sludge AB Process followed by the existing Traveling Bridge Filters is the recommended alternative for nitrogen and phosphorus removal. It is recommended that the City of Woonsocket upgrade the WWTF to incorporate use of a two-stage activated sludge AB Process for the following reasons:

1. Two-stage process, thereby providing a robust multi-barrier type treatment approach to low level nitrogen removal

2. Nitrification is seeded in the first stage and protected from washout by the second stage

3. Simple operation (no complicated control sequence)

4. Nitrification and denitrification are incorporated into both stages of the AB biological treatment process. This gives greater operational flexibility and the optimization of the process size allows both stages to be located in the same area of the plant.

5. This alternative has a competitive capital and annual differential cost (annual debt retirement plus power and maintenance cost) of the four process alternatives evaluated.

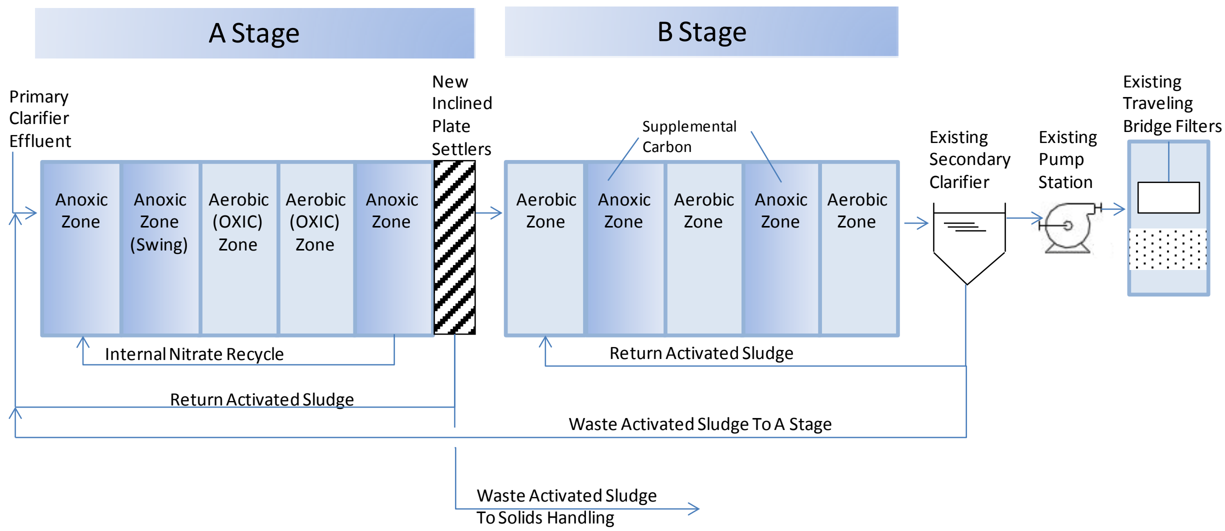
6. The facility's existing secondary effluent pumping station can be re-utilized with this process, thereby minimizing overall project costs for compliance.

7. The alternative will require minimum new aeration basin tankage to be built and will fit within the available land area of the existing facility's fenced property line.

8. Of the four short-listed process alternatives, the two-stage activated sludge AB process will be the simplest to operate and maintain and is endorsed by the plant's current operating staff.

This recommended nutrient removal treatment system combines nitrogen removal and phosphorus removal processes and is graphically depicted in Figure ES-2. The AB process would consist of three (3) “A” process trains followed by two (2) “B” process trains and then flow would be conveyed by gravity to the secondary clarifiers and then pumped to the existing traveling bridge sand filters (4 filters).

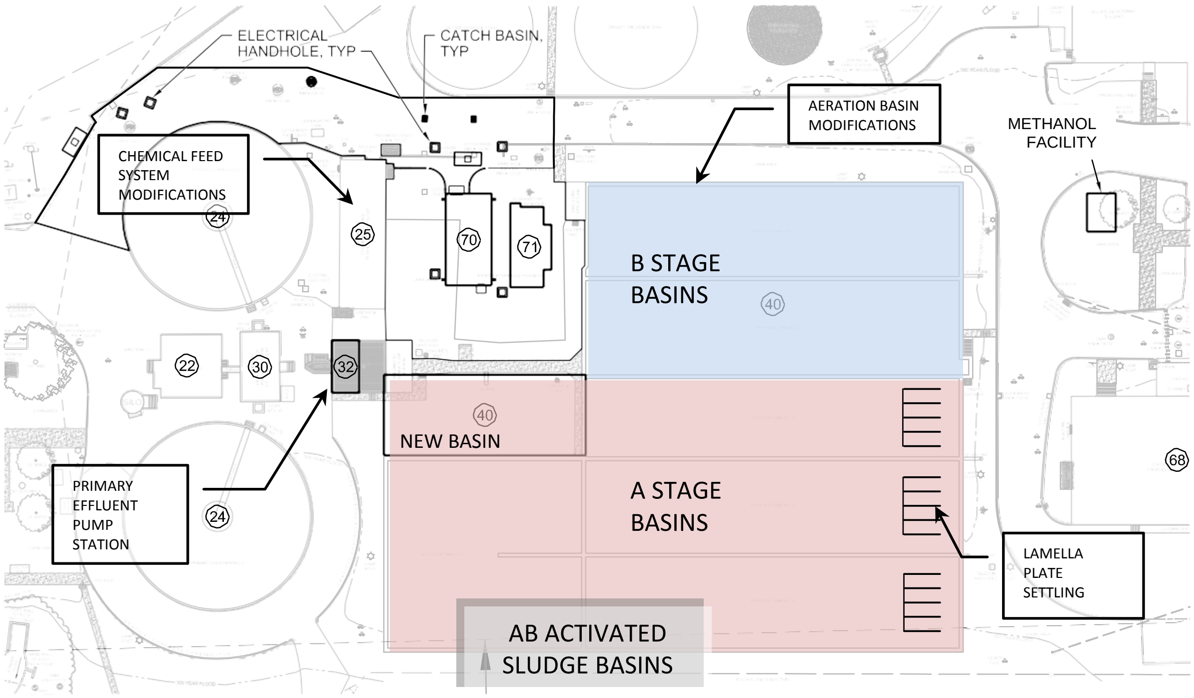
FIGURE ES-2
TWO-STAGE ACTIVATED SLUDGE AB PROCESS FOLLOWED BY EXISTING TRAVELING BRIDGE SAND FILTERS
PROCESS FLOW DIAGRAM



With this recommended nitrogen and phosphorus removal alternative it is proposed that the existing aeration tanks be modified into a two-stage activated sludge AB process. The facility currently has six (6) 0.95 million gallon (mgal) aeration tanks; thereby providing a total volume of approximately 5.7 mgal. Under this nitrogen removal alternative it is proposed that a new 0.5 mgal aeration tank be constructed and the existing aeration tanks be reconfigured into two-stages with multiple anoxic and aerobic zones. One of the significant advantages of the AB process is that the traveling bridge sand filters are reused and the filter building retained.

Another advantage of the AB process is that no new process tanks are constructed to the west or within the vicinity of the current filter building complex. Figure ES-3 depicts the Two-stage Activated Sludge AB Process conceptual layout.

FIGURE ES-3
TWO-STAGE ACTIVATED SLUDGE AB PROCESS CONCEPTUAL LAYOUT



ES.10 SECTION 7 – DEVELOPMENT AND EVALUATION OF ANCILLARY WASTEWATER TREATMENT IMPROVEMENTS

This section presents an evaluation of ancillary WWTF process improvements needed to support the new nitrogen and phosphorus removal processes. As a result of the evaluations in Sections 5 and 6, certain WWTF systems and unit processes were identified as requiring upgrades and/or improvements. Evaluations of alternatives are provided for individual unit processes.

The recommended upgrades in Section 7 are separated into two categories; those required as part of the immediate nutrient removal treatment system upgrades, and those upgrades that may be addressed under the WWTF's long-term capital improvements plan.

The use of renewable energy at the WWTF and energy efficiency of the equipment and system will be integrated within the WWTF capital improvements. The proposed electrical system connection with the Synagro co-generation facility will provide renewable energy for plan operation. Energy efficient features that can be implemented as part of the capital improvements include the use of efficient high speed turbo blower for process aeration and the use of an enhanced nutrient removal process to optimize oxygen requirements. The remainder of the recommended improvements are expected to be implemented through a series of smaller capital projects in subsequent years or through repair and replacement funds in the operations contract.

Although not directly required for the immediate nitrogen and phosphorus system upgrades that are required to ensure compliance with the new RIPDES discharge permit, a number of other WWTF systems and unit processes were identified as requiring upgrades and/or improvements over the 20-year planning period. It is envisioned that the majority of these ancillary system improvements will be completed in a prioritized phased approach over the 20-year planning period to replace and/or upgrade aging infrastructure and treatment systems.

Similarly, the City of Woonsocket and its sewage collection system operator, Veolia Water (VWNA) have been working on a comprehensive underground asset management (UGAM) program throughout the community. This program has resulted in recent and planned upgrades to the collection system, which will result in additional benefit to the treatment facility.

Working with Veolia Water (VWNA), Wright-Pierce conducted several site visits to the WWTF to assess the condition of the existing facilities. The information gathered from visual observations during these site visits along with review of facility record drawings, reports and discussions with WWTF staff was used to develop the various recommendations for equipment replacement and facility upgrades in this section. Those systems that are recommended to be replaced as a part of the planned capital improvements are included in CH2M HILL's proposal for the design-build upgrades to the treatment plant.

By the time the new RIPDES permit limits officially take effect on May 1, 2017, many of the buildings, structures and equipment systems at the facility will have been in service for approximately 40-50 years. Generally speaking, structures and buildings at wastewater facilities are considered to have a useful design life of 50 years, while individual unit processes and equipment systems are designed for an approximate 20 to 25 year useful life expectancy. Typically, this is considered an appropriate design life such that all of the existing process equipment systems would not be expected to last through the next 20 year planning period.

Significant portions of the older systems and equipment at the WWTF should be considered for replacement or upgrade over the 20-year planning period due to deteriorating physical conditions and the diminishing availability of spare parts. Major facility systems that were considered for future replacement and their respective issues and concerns identified are summarized in Table ES-5.

TABLE ES-5
WWTF SYSTEMS AND EQUIPMENT TO BE CONSIDERED FOR REPLACEMENT AND/OR UPGRADE OVER 20-YEAR PLANNING PERIOD

Unit Process/Equipment	Issues/Concerns
Headworks – Influent Screening Facility	• 1962 and 1974 vintage equipment systems
Headworks – Raw Influent and Recycle Pumping Facilities	• 1962 and 1974 vintage equipment systems Deterioration/loss of wall thickness on discharge piping • Insufficient pumping system redundancy Isolation sluice gates inoperable • Area sump pumps • Need for Plant Water for seal water
Aerated Grit Facilities	• 1962 vintage equipment systems • Corrosion/deterioration • Aerated Grit Blower unit service reliability
Primary Clarifiers	• 1962 vintage equipment systems (sludge collection mechanisms) • Corrosion/deterioration
Primary Sludge Pumping Station	• 1962 vintage equipment systems • Corrosion/ deterioration • Primary Sludge Pump (1)
Primary Effluent Pumping Facilities	• 1974 vintage screw pumps/ systems • Insufficient pumping system redundancy
Aeration Tanks – Optimize BNR Process	• 1974 vintage aeration blower units (2) • Inefficient air flow control systems • Poor dissolved oxygen control • RAS piping system modifications • Recycle Pumps and mixer systems
Secondary Clarifiers	• 1974 vintage equipment systems • Corrosion/deterioration
Return Sludge Pumping Station	• 1974 vintage Pumping units • Deteriorated discharge piping systems (pipe and valves) • RAS, WAS and Secondary Scum pumping units

ES.11 SECTION 8 – EVALUATION OF BUILDING SUPPORT SYSTEMS

The assessment of the existing building and support systems at the Woonsocket WWTF is summarized in this Section including architectural/structural, heating, ventilation and air conditioning (HVAC), instrumentation and controls, and electrical.

The assessments presented in Section 8 are based on information gathered from site visits, review of facility record drawings, previous studies/reports, and discussions with WWTF operations staff. The assessment includes both improvements that are required by the immediate project to achieve enhanced nitrogen and phosphorus removal, and other additional long-term capital improvements which are recommended over the 20-year planning period.

The Woonsocket WWTF has numerous systems that have served the City well and exceeded their intended design life expectancy. Generally speaking, the overall condition of the facility and its building and support services is acceptable, but significant capital improvement needs were identified which should be included in the City's WWTF long-term capital improvement plan. The facility's electrical systems and equipment are the source of the greatest upgrade needs that must be addressed as part of the immediate improvements project. The recent electrical mapping work, performed in 2012 by CH2M HILL, provided observations and recommendations for upgrades to the electrical system. The need to address standby/emergency power is an immediate priority and the facility's instrumentation and controls will also require a major upgrade.

The most significant elements of the facility's building and support service upgrade needs include:

- New 13.8kV switchgear
- Incoming power from National Grid
- The interconnection between the WWTF and Synagro
- Power from the new 2500 KW standby generator
- Distribution feeders to WWTF site loads
- A new 2500 kw standby generator (back-up power for the treatment plant);
- New Main Distribution Switchboard in an existing building

The improvements to the electrical service equipment are graphically shown in Figure 8-10.

ES.12 SECTION 9 – PLAN SELECTION and SECTION 10 – PLAN IMPLEMENTATION

These sections present the recommended upgrade plan and schedule to ensure compliance with the revised water quality criteria contained in the new permit. Institutional responsibilities, public work shop awareness, and a schedule of improvements are addressed.

Section 9 includes the recommendations for the nutrient removal upgrade requirements outlined in Sections 5 and 6, along with the other long-term maintenance program capital improvements that will need to be completed in a prioritized manner during the 20-year planning period. Section 9 also presents the design-build construction costs for the WWTF's immediate nutrient removal upgrade and other planned capital improvements. The recommended upgrades and are as described below.

ES.12.1 Nutrient Removal Recommended Improvements

As a result of the Facility Planning efforts, a number of major WWTF systems and equipment are recommended to be upgraded to ensure reliable compliance with the new nitrogen and phosphorus treatment standards. These upgrade recommendations are summarized in Table ES-6. These upgrades are also described in more detail in Sections 5, 6, 7 and 8, and are shown on Figure ES-5.

TABLE ES-6
RECOMMENDED IMPROVEMENTS FOR NUTRIENT REMOVAL

Unit Process/Equipment	Recommended Improvements
Headworks – Influent Screening Facility	Requirement for the nitrogen and phosphorus upgrades: • 1st Stage Screening System (replace existing coarse screen system with new 3/8" mechanical screening system)
Headworks – Raw Influent and Recycle Pumping Facilities	Requirement for peak design flow unit process redundancy: • New pumping unit for system redundancy

TABLE ES-6
RECOMMENDED IMPROVEMENTS FOR NUTRIENT REMOVAL

Unit Process/Equipment	Recommended Improvements
Influent Flow Metering	Requirement for the nitrogen and phosphorus upgrades: • Magnetic type flow meters on influent pumping station discharge lines • Magnetictype flow meter on recycle line/ plant drain
Primary Effluent Pumping Facility	Requirement for peak design flow unit process redundancy: • New pumping unit(s) for system redundancy
Aeration Basins	Requirement for the nitrogen and phosphorus upgrades: • 1st and 2nd Stage Modifications to Aerations Basins • New Plate Settlers • New Fine Bubble Diffusers • New 1st Stage RAS/WAS Pumping
Nutrient Removal Modifications and Chemical Systems	Nitrogen Removal Treatment Process: • Supplemental Carbon System • Modifications to optimize BNR in Aeration Tanks • Chemical Feed System modifications Phosphorus Removal Systems: • Chemical Feed System modifications
Electrical and Standby Power Systems	Requirement of the nitrogen and phosphorus upgrades: • New double ended Motor Control Center to replace MCC-1 in Operations Building • Relocated primary power feed to MCC-2 in Operations Building • Remove existing generator in Operations Building • Replace Main Distribution Switchboard. • New 2500 kW generator within outdoor sound-proof enclosure for backup power to Main Distribution Switch Gear

TABLE ES-6
RECOMMENDED IMPROVEMENTS FOR NUTRIENT REMOVAL

Unit Process/Equipment	Recommended Improvements
SCADA System Modifications	Requirement of the nitrogen and phosphorus upgrades: • Integrate new nutrient removal systems into existing SCADA • Replace multiple PLC with more-open architecture (Allen-Bradley based with Ethernet communication protocol) • Selective upgrades to communication wiring • Expand SCADA to include data from Synagro's Solids Handling Facility SCADA system

ES.12.2 Opinion of Probable Construction Costs

The WWTF design build capital improvements cost is $36,899,314, based on the contract agreement between the City and CH2M HILL for construction and engineering services for the treatment facility modifications to meet the nitrogen and phosphorus removal requirements and to provide the facility modifications including the following:

- New influent pump
- New influent screening facility with odor control system
- New influent flow measurement
- Modified primary effluent pump station
- Activated sludge modifications to include first and second stage activated sludge basins, lamella plate first stage settling, and new first stage return and waste sludge pumping
- Electrical power system modifications and new standby generator
- HVAC modifications to selected facilities
- I&C modifications

ES.13 IMPLEMENTATION SCHEDULE FOR RECOMMENDED TREATMENT SYSTEM IMPROVEMENTS

The recommended implementation schedule for the required nutrient removal upgrades and ancillary system improvements required to reliably meet the new nitrogen and phosphorus standards (identified in Section 10 – Plan Implementation) has been developed. This schedule which is based upon deadlines agreed upon through the Consent Agreement and the revised schedules are presented in Figure ES-4.

ES.14 RECOMMENDED PLAN TO MEET RIPDES PERMIT LIMITS

There are many variables to consider when evaluating different nutrient removal technologies and determining the most appropriate technology. Since RIPDES Permit compliance was of primary concern, the selection of a treatment technology that achieves permit requirements was essential. Based on the technology evaluations and understanding of the individual effectiveness for each evaluated alternative to reliably meet the new permitted effluent nitrogen and phosphorus limits a new two-stage activated sludge AB process will be implemented for nitrogen and phosphorus removal.

It is recommended that the City of Woonsocket construct a two-stage activated sludge AB process followed by reuse of the existing traveling bridge sand filters to meet the new nitrogen and phosphorus permit limits. All of the proposed construction activities will be within the existing city-owned property lines for the WWTF, thereby minimizing disturbance to the environment including the Blackstone River. Figure ES-5 shows a layout of a portion of the Woonsocket WWTF highlighting the proposed location for the recommended new tertiary treatment facilities that are required to ensure compliance with the new RIPDES permit limits.

FIGURE ES-4
IMPLEMENTATION SCHEDULE

Phase and Tasks	Dates
Respond to RIDEM Comments and Submit Upda	December 17, 2012
RIDEM Review	Dec 17 - Mar 28, 2013
Respond to RIDEM Comments and Submit Final	Mar 29 - May 3, 2013
RIDEM Approval of Final Facilities Plan	May 17, 2013
Early Tasks	
Site visit - electrical and I&C investigation	Oct 15 - 19, 2012
Survey and develop mapping	Oct 15 - 26, 2012
Design	
30% - Schematic Design	Jun 14 - Jul 30, 2013
30% - Schematic Design QC	Jul 31 - Aug 13, 2013
60% - Design Development	Aug 14 - Oct 16, 2013
60% - Design Development QC	Oct 17 - 30, 2013
90% - Construction Documents	Oct 30 - Dec 27, 2013
90% - Construction Documents QC	Jan 2 - 15, 2014
100% - Design	Jan 16 - Mar 1, 2014
Construction	
Construction	Mar 1, 2014 - Jan 1, 2017
Substantial Completion	October 1, 2016
1st Performance Testing	Dec 1 - 30, 2016
2nd Performance Testing	May 1 - 30, 2017
Revised Permit	May 1, 2017

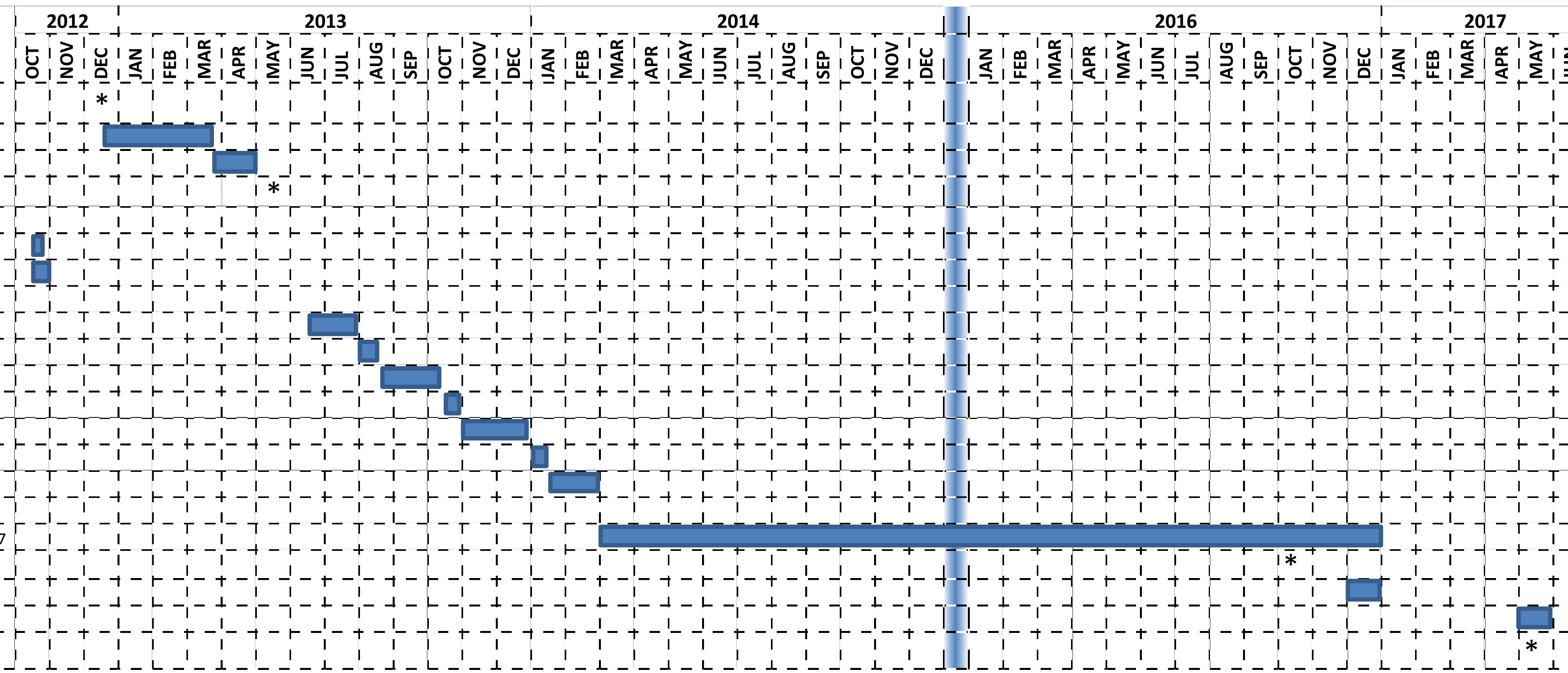
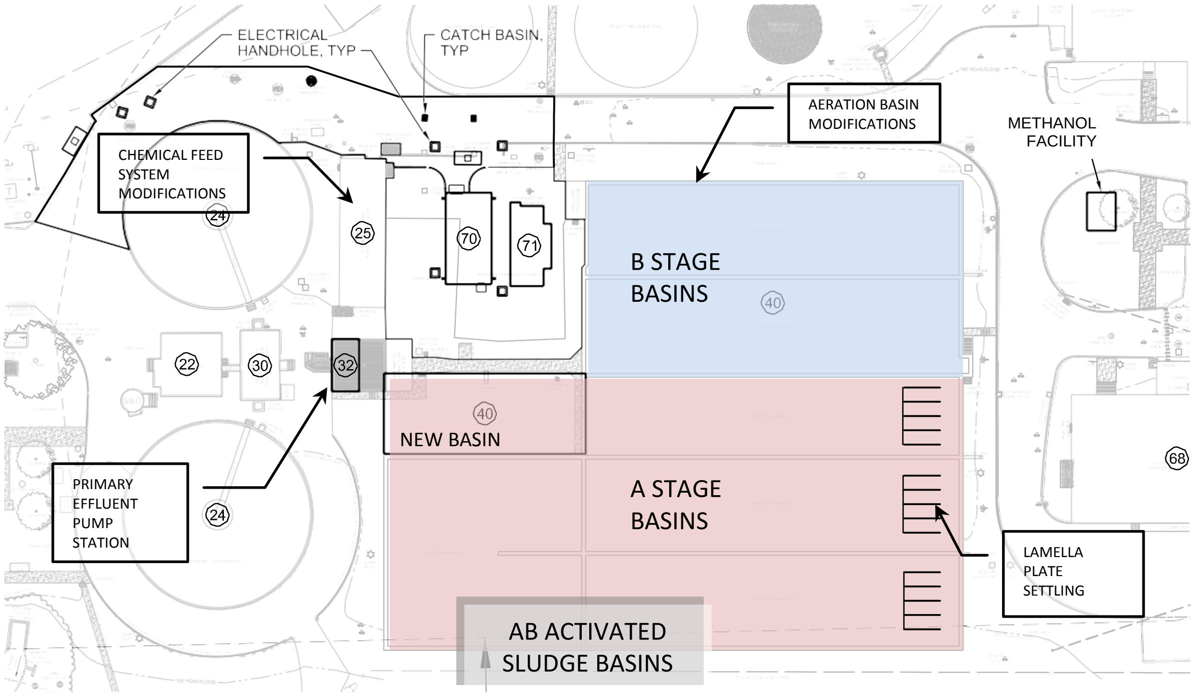


FIGURE ES-5
LOCATION OF RECOMMENDED TWO-STAGE ACTIVATED SLUDGE AB PROCESS



SECTION 1
INTRODUCTION

1.1. BACKGROUND

The City of Woonsocket, Rhode Island is responsible for an extensive wastewater collection system and regional treatment facility that also accepts flow from the communities of Bellingham and Blackstone, Massachusetts, and North Smithfield, Rhode Island. These three client communities discharge to the Woonsocket Regional Wastewater Treatment Facility (WWTF) under intermunicipal agreements with the City of Woonsocket. The City of Woonsocket is the holder of the discharge permit issued under the Rhode Island Pollutant Discharge Elimination System (RIPDES). The WWTF treats a combination of domestic and industrial wastewater and includes advanced secondary treatment and biological nutrient removal using the Modified Ludzack-Ettinger or MLE activated sludge (AS) process. The treated effluent from the WWTF is discharged into the Blackstone River. The WWTF is operated by CH2M HILL through an operations contract with the City of Woonsocket.

The last significant series of upgrades to the facility were carried out between 2001 and 2004 in accordance with a Facility Plan Amendment completed in February 2000. The subsequent upgrades and supplemental improvement projects were completed to achieve the nitrogen and phosphorus removal requirements of the 2000 RIPDES discharge permit requirements. The improvements included the construction of two additional aeration tanks, effluent sand filters, and a chemical storage building.

The WWTF is currently designed to treat a maximum monthly raw influent flow of 16 mgd, a maximum day flow of 24 mgd, and a peak hourly flow of 32 mgd. The treatment processes at the facility include coarse screening, aerated grit removal, primary clarification, biological nutrient removal, secondary clarification, effluent sand filtration, disinfection using sodium hypochlorite, and dechlorination with sodium bisulfite. The activated sludge nutrient removal process includes six aeration tanks and three secondary clarifiers. Primary and secondary (waste activated) sludge is pumped to a gravity thickener. Synagro Technologies, Inc (Synagro)

operates and maintains the sludge thickening, storage, and dewatering facilities through a long-term agreement with the City of Woonsocket. Synagro owns and operates the incinerator facilities subject to the provisions within their contract. These facilities include dewatering centrifuges, a fluidized bed incinerator, and other related sludge processing equipment.

1.2. PROJECT NEED AND PLANNING AREA

The facility's latest RIPDES permit (No. RI 0100111) went into effect on October 1, 2008 and includes more stringent seasonal discharge limitations for nitrogen and phosphorus. The City entered into a Consent Agreement (RIA-368) with the Rhode Island Department of Environmental Management (RIDEM) to prepare a Facilities Plan Amendment evaluating the necessary improvements to achieve the revised nitrogen, phosphorus, and cadmium limits, and to implement the recommended improvements. The Consent Agreement which was finalized and signed in February 2011 established a compliance schedule for completing the Facility Plan Amendment. A letter, dated March 23, 2012, from RIDEM to the City revised some of the dates in the consent decree and provided guidance on the construction schedule. The permit contains interim permit limits for nitrogen and phosphorus as well as the revised cadmium limit. The issue with total cadmium is addressed in Section 1.6 below.

The Facility Plan Amendment is the first step in the process to modify the plant to comply with the revised RIPDES permit and Consent Agreement. CH2M HILL is responsible for meeting the new permit limits in accordance with the schedule from RIDEM. CH2M HILL is the program manager for the City, and is coordinating all required operations, planning, design and construction-related services through the operate-design build operate contract (O-DBO). The last upgrade to the WWTF provided the capacity to meet an interim seasonal total nitrogen limit of 10 mg/L and a total phosphorus limit of 1 mg/L. The revised effluent limits in the RIPDES permit require the facility to meet a seasonal total nitrogen limit of 3 mg/L and a total phosphorus limit of 0.1 mg/L. The existing facility currently does not have the capacity to achieve the more stringent nutrient limits, and therefore capital improvements are required.

The pending nitrogen and phosphorus limits are among the lowest permitted levels in the United States, especially since the criteria include monthly average limits and not seasonal or annual averages. Proven methods for total nitrogen and phosphorus removal will be required and are evaluated in detail as part of this Facility Plan Amendment. This includes an updated assessment of flows and loads to the WWTF as well as recycle flows and loads including those from the Synagro sludge handling facilities. The intent of the revised Consent Agreement is to complete the capital improvements by January 1, 2017.

The Consent Agreement requires the City of Woonsocket to reduce excessive wet weather flow in the collection system. In response, the City has contracted with Veolia to implement an Underground Asset Management (UGAM) Program that includes infiltration and inflow removal. A summary of the Infiltration and Inflow Program was prepared as part of the Facility Plan Amendment and is attached as Appendix C. It includes a summary of collection system maintenance completed to date by the City and Veolia's UGAM team. Also included are recommendations for implementing alternatives to reduce excessive wet weather flows in the collection system or at the WWTF in order to ensure compliance with RIPDES permit flow limits and the hydraulic capacity of the facility. The purpose of this analysis is to document the steps that the City has taken to investigate and implement I/I removal measures. A qualitative assessment has been carried out in developing the recommended plan for further investigation and remediation of Infiltration/Inflow (I/I) sources in the Woonsocket collection system. The City submits Annual Reports on the Collection System Program to RIDEM and EPA Region 1 consistent with CMOM (Capacity, Maintenance, Operations, and Management) regulatory requirements.

The Consent Agreement included the possibility that the selected nutrient removal technology would require pilot testing. However, pilot testing is not necessary or required based on the selection of a proven full-scale nutrient removal technology. The two-stage activated sludge AB process for nutrient removal could not be adequately piloted without the biological sludge from the second stage. Therefore, the requirement for pilot testing was waived. Additionally, CH2M HILL is responsible for meeting the new nutrient permit limits.

At the City's request, this Facility Plan Amendment addresses both the immediate needs to upgrade the existing WWTF to achieve compliance with revised nitrogen and phosphorus limits of the new RIPDES permit, and also the capital improvement needs of the remainder of the WWTF over the 20-year planning period. Given the anticipated cost of the improvements to meet the revised nitrogen and phosphorus limits, the City has requested a phased approach with an initial capital project that includes those improvements that are immediately necessary for RIPDES permit compliance and to take advantage of the electricity produced by the Synagro co-generation project. The use of renewable energy at the WWTF and energy efficiency of the equipment and systems will be integrated withing the WWT capital improvements. The proposed electrical system connection with the Synagro co-generation facility will provide renewable energy for plan operation. Energy efficient features that can be implemented as part of the capital improvements include the use of efficient high speed turbo blowers for process aeration and the use of an enhanced nutrient removal process to optimize oxygen requirements.The remainder of the recommended improvements are expected to be implemented through a series of smaller capital projects in subsequent years or through repair and replacement funds in the operations contract.

1.3. EXISTING TREATMENT PROCESS

A process flow schematic of the WWTF is shown in Figure 1-1, and the basic design data is summarized in Appendix A. Figure 1-2 shows an aerial view of the Woonsocket WWTF with the major buildings and processes highlighted.

Wastewater enters the WWTF through three main gravity sewers; a 60-inch low level interceptor, a 24-inch high level interceptor, and an 18-inch industrial interceptor. These three sewer lines discharge into the influent structures at the head of the facility, where the wastewater receives preliminary screening. Septage is discharged into the high level interceptor in a manhole just before the influent structure. All of the wastewater that enters the WWTF passes through a coarse mechanically cleaned bar screen to remove any large solids or debris that would clog pumps or piping before entering the wetwell. There were formerly two comminutors following the mechanical bar screen, but they have been removed.

There are two raw influent wet wells located in the basements of the Operation and Administration Buildings, and separate pump rooms associated with each. The raw influent passes through screening to the Operations Building Wetwell. Recycle flows are discharged directly to the Wetwell in the Administration Building. The wet wells are hydraulically connected via a 30-inch ductile iron pipe. There are five existing raw influent pumps, three in the Operations Building pump room and two in the Administration Building pump room. Four of the pumps have 8-mgd capacity and one has an upgraded impeller to provide 10-mgd capacity. This provides 32-mgd of firm capacity with one pump as standby, and a total of 42-mgd with all units on-line.

FIGURE 1-1
PROCESS FLOW DIAGRAM

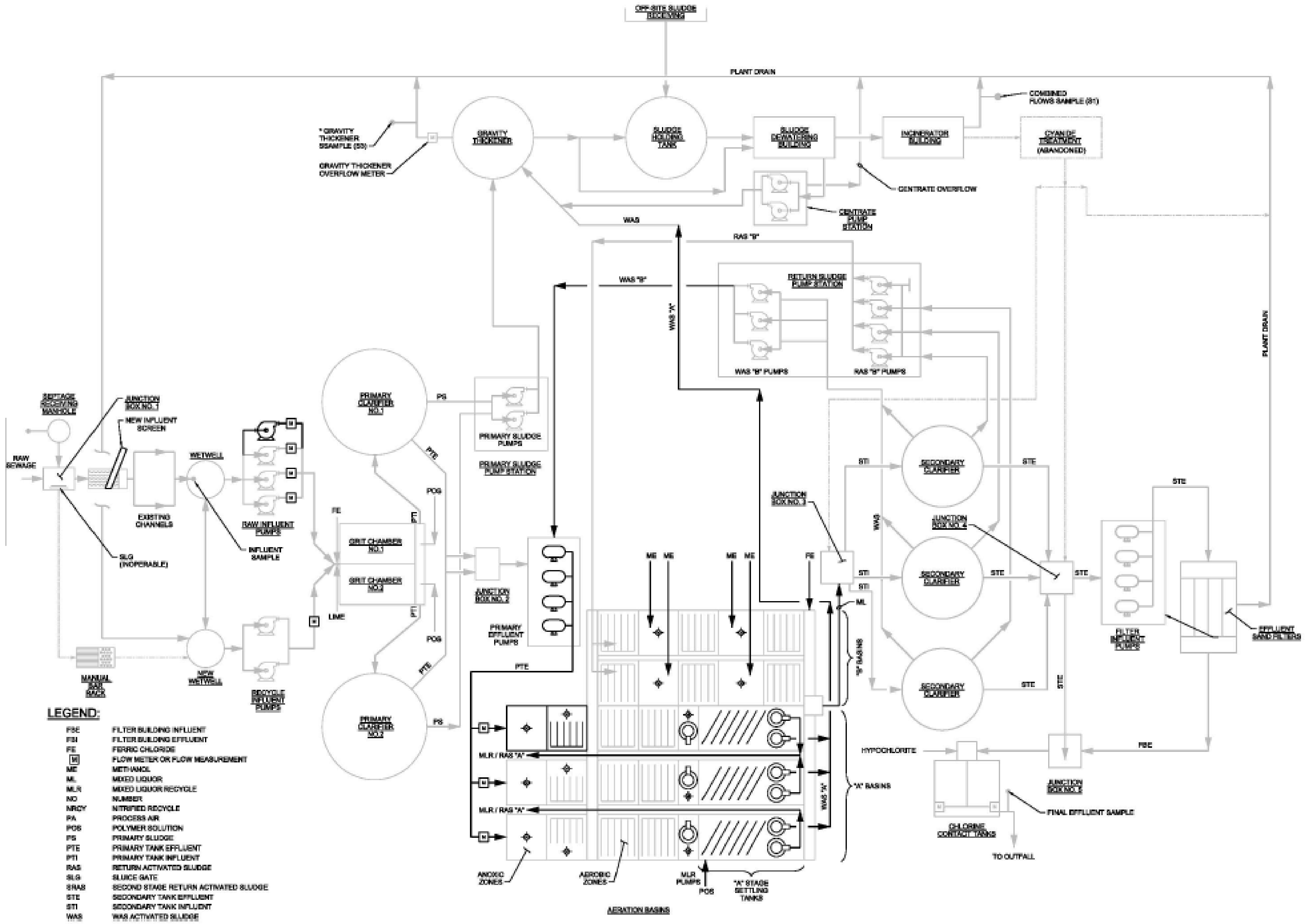


FIGURE 1-2
WWTF AERIAL VIEW

The wastewater is pumped from the wet wells to the aerated grit removal system which removes any heavy abrasive material such as sand from the wastewater. Operators have the ability to add ferric chloride and polymer to the wastewater at the aerated grit chamber to provide chemically enhanced primary treatment (CEPT), ferric alone to enhance phosphorus removal, and lime for alkalinity addition, which has the ability to promote biological nitrogen and phosphorus removal in the secondary system. The pretreated wastewater then flows by gravity to either of two primary clarifiers. The primary clarifiers further reduce solids in the wastewater, where typically about 75 percent of suspended solids and 40 percent of biochemical oxygen demand (BOD) are removed. The solids are removed from the bottom of the settling tank by a scraper and pump system. Oil and grease tend to float to the top of the tank as scum, where the surface of the water inside of the baffle is cleaned by a surface skimmer. The skimmer pushes the scum from the surface into a small trough. Scum from both the aerated grit chambers and primary clarifiers is collected in the scum well at the Primary Sludge Pumping Station and pumped to the Sludge Storage Tank.

Primary clarifier effluent flows into Junction Box No. 2 where it is then pumped by two screw pumps and two submersibles to the aeration tank influent channel. Primary effluent flow is measured in this channel using a Parshall Flume. The flume has a 36-inch throat width and measures the combination of raw influent, centrate from merchant liquid sludge, and plant recycle flows. The primary effluent flow, which is also the aeration tank influent flow, is the basis of design for any upgrade to the existing activated sludge process in order to meet the new effluent nitrogen and phosphorus limits.

The Woonsocket WWTF uses the MLE activated sludge process to remove nitrogen as well as carbonaceous demand from the wastewater. The process has also been successful in achieving biological phosphorus removal. A population of beneficial microorganisms (mixed liquor suspended solids or MLSS) is maintained in the aeration tanks that help to decompose and stabilize organic wastes, which in turn converts dissolved and suspended solids into settleable solids. The MLE process consists of two-zones in each of the six tanks which are operated in parallel. Each aeration tank includes an anoxic zone followed by an aerobic zone which is aerated. Nitrification occurs in the aerobic zones, while denitrification is accomplished in the anoxic zone. The anoxic zones have submersible mixers to maintain the MLSS in suspension. The aerobic zones are aerated through a fine bubble diffused aeration system. The diffusers are 9-inch membrane type. The blower system is located in the Return Sludge Building, and has recently been upgraded with two new Neuros turbo blowers. The two new turbo blower variable frequency drives were recently knocked offline due to a lightning strike near the plant. They are currently being retrofitted with surge protection to minimize the potential for an electrical surge. Two of the original four multi-stage centrifugal blowers remain to provide peak capacity. The process includes internal recycle pumps that draw from the discharge end of the aerobic zone back to the feed end of the anoxic zones, although currently these pumps are not functional. The mixed liquor flows by gravity to Junction Box No. 3 which distributes it to the three secondary clarifiers where the solids are separated.

The secondary clarifiers provide a quiescent environment with proper detention time allowing gravity to settle the solids to the bottom. The settled solids known as "activated sludge" are returned (return activated sludge-RAS) to the aeration tank influent channel to maintain the desired microbial population in the aeration tanks. Excess or waste activated sludge (WAS) is transferred by pumps to the gravity thickener. The clarified wastewater, called secondary effluent, flows to Junction Box No. 4.

Operators have the ability to distribute the secondary effluent to the influent wetwell of the Effluent Filter Building, or directly to the Chlorine Contact Tanks via Junction Box No. 5. Traveling bridge sand filters are used to polish the secondary effluent by removing small solids carried out of the secondary clarifiers. The secondary effluent is pumped up to the feed of the effluent sand filters using four submersible vertical impeller pumps, each with 8 mgd capacity. The flow is distributed to the four parallel traveling bridge filters. Backwash is discharged directly to the plant drain system. The filtered effluent flows by gravity through an under drain system to an effluent channel. The combined effluent from all four filters flows by gravity to the chlorine contact tanks for disinfection.

Disinfection of the wastewater is achieved using sodium hypochlorite at the inlet of the Chlorine Contact Tanks. The wastewater flows through the two parallel tanks in order to provide adequate contact time for disinfection. After disinfection, the wastewater flows over an effluent weir into the final effluent trough where it is dechlorinated using sodium bisulfite. The treated effluent is discharged through the outfall to the Blackstone River.

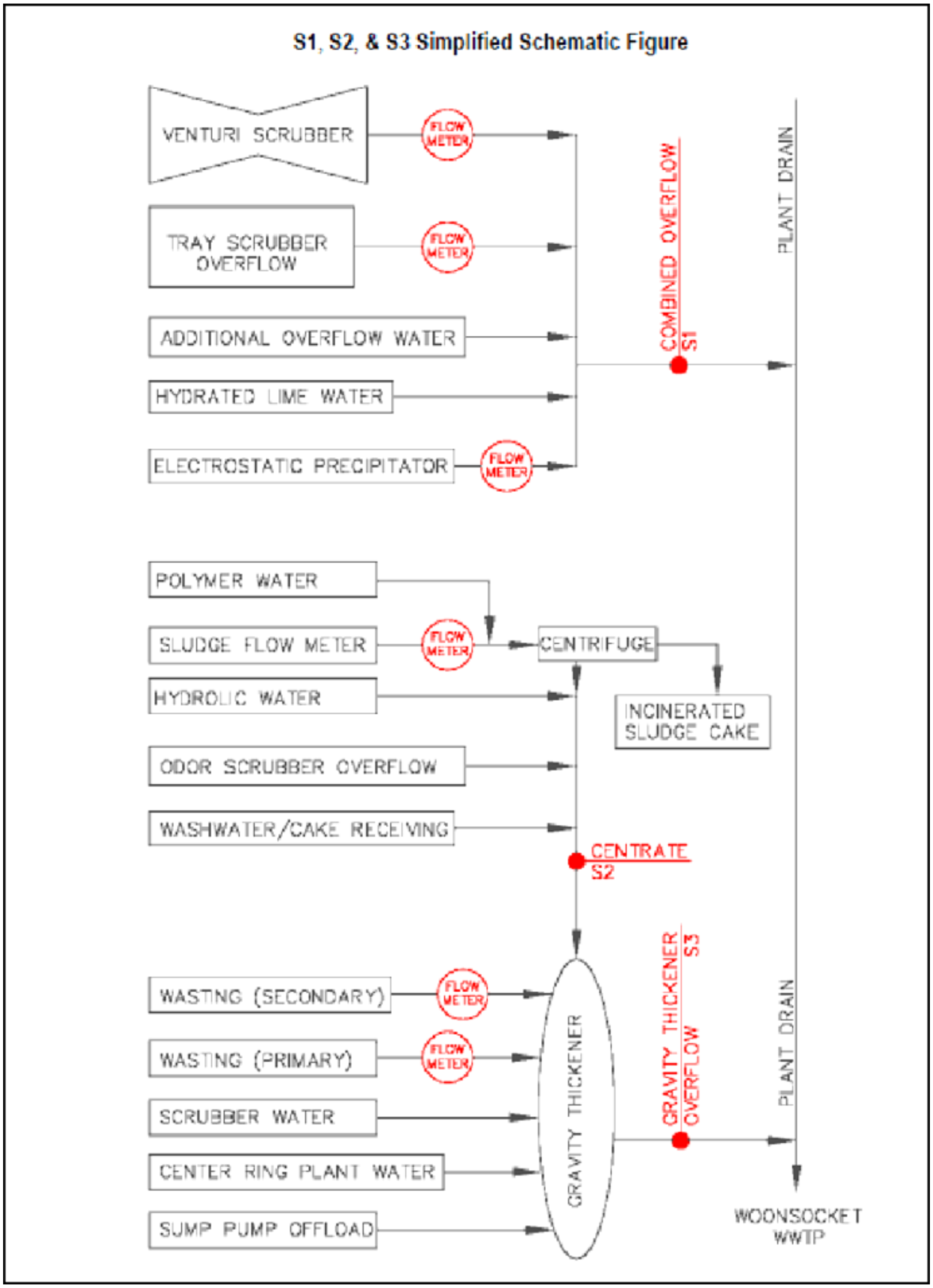
The solids handling and incinerator facilities are operated by Synagro. This Facility Plan Amendment does not assess the solids handling facilities which are operated and maintained by Synagro under a long-term agreement with the City. However, the plan will consider the return flows and loads from the solids handling facilities, including the impacts of gravity thickener operation and temperature effects on the biological removal process.

As noted above, primary and secondary sludge from the Woonsocket liquid train are discharged to a gravity thickener. Synagro has receiving facilities for liquid merchant sludge that is pumped into the sludge storage tank. Woonsocket scum is pumped directly to this sludge storage tank as well. The thickened sludge from the gravity thickener and the liquid merchant sludge are dewatered using centrifuges. The centrate and other washdown flows are discharged to the Centrate Pump Station, which transfers this flow to the gravity thickener. The cake is fed to the fluidized bed incinerator. Synagro also has facilities for receiving merchant cake solids as feed to the incinerator. The fluidized bed incinerator has a venturi scrubber followed by a tray scrubber for treatment of the exhaust air. The makeup water for these scrubbers is plant water drawn from the chlorine contact tank. The ash-laden portion of the scrubber effluent is transferred to an ash thickener and a portion of the effluent is discharged directly to the plant drain. Synagro adds lime for neutralization to the ash thickener feed. The overflow from the ash thickener is also discharged to the plant drain.

There are three (3) sampling locations where Synagro monitors process wastewater that is discharged to the WWTF. The sampling locations are displayed schematically in Figure 1-3. Sampling location, S-1, is known as the combined flow and includes discharge from the venturi and tray scrubbers as well as the overflow from the ash thickener. The second sampling location, S-2, is at the centrate pump station. The third sampling location, S-3 is the Gravity Thickener Overflow. It includes primary and secondary sludge from the WWTF, discharge from a local sump pump, plant water to help freshen the gravity thickener, and the centrate from dewatering operations. The local sump pump discharge includes the blow down flow from the packed bed scrubber treating the exhaust air of the gravity thickener and adjacent merchant sludge holding tank.

The gravity thickener overflow rate is measured by a flow meter that was recently installed in the spring of 2010. The flow rates of the S-1 and S-2 discharges are not measured directly, but are estimated based on measurement of inputs.

FIGURE 1-3
SCHEMATIC OF RECYCLE FLOWS
FROM SOLIDS HANDLING FACILITIES



1.4. HISTORICAL OVERVIEW OF FACILITY

The original treatment facility was constructed in 1897 and has been modified numerous times throughout the years. The first modern era upgrade was completed in 1930. It included modifications to the original 1897 pump station, construction of preliminary screening facilities, aeration tanks, final settling tanks, sludge digestion tanks, and sludge drying beds. The current WWTF layout and treatment processes bear little resemblance to the plant that existed in the 1930's.

In 1962, a major upgrade to the WWTF commenced and included construction of the Operations Building, where the main influent wet well is located. The aerated grit chambers, two circular primary clarifiers, Primary Sludge Pumping Station, two sludge digestion tanks, a Chlorination Building, chlorine contact tank, and the sludge dewatering building were all erected as part of WWTF improvements. The oldest equipment, tankage, and structures at the existing WWTF date back to this project.

The next major upgrade to the WWTF was completed in 1977. This project included the construction of the Administration Building and raw influent wet well, the primary effluent screw pump station, four new aeration tanks, three new secondary clarifiers, the Return Sludge Pumping Station, a new Chlorination Building and garage, modification of the sludge digesters to gravity thickener tanks, the original sludge dewatering and incinerator facilities, and new chlorine contact tanks.

Synagro's predecessor began operating the solids handling facilities to dewater and incinerate merchant liquid sludge in the 1980s. Subsequently, one of the gravity thickeners was converted to a liquid merchant sludge storage tank.

The aeration basins installed as part of the mid-1970s upgrade included both mechanical surface aerators and coarse bubble diffused aeration. The diffused aeration system was upgraded to fine bubble diffusers apparently in the 1990s, and the mechanical surface aerators were demolished. It appears that odor control projects for the Administration building wetwell and the aerated grit chamber and primary clarifier effluent weirs were installed in the 1990s as well.

Based on recommendations in the last Facility Plan Amendment in February 2000, a series of upgrades were implemented in the early 2000's to provide the capacity to meet a seasonal total nitrogen limit of 10 mg/L and a total phosphorus limit of 1 mg/L. These improvements were implemented in a two-phase design-build approach with supplemental improvements to each phase and represent approximately $20 million of capital expenditures. The previous Phase I improvements included the construction of a new chemical storage/feed building adjacent to the primary clarifiers. The building houses sodium hydroxide, ferric chloride, and polymer storage and feed facilities. The sodium hydroxide can be added in Junction Box No. 2 (influent wet well to the Screw Pump Station), while the ferric chloride and polymer can be fed into the aerated grit chambers. Provisions were also made to allow the addition of polymer to the aeration tank effluent channel. The Phase 1 improvements also included the construction of the pump station that transfers centrate from dewatering operations to the gravity thickener, and an odor control scrubber was installed for the exhaust from the gravity thickener and sludge storage tank.

The previous Phase II improvements included the construction of two additional aeration tanks bringing the total to six and the Effluent Filter Building. Modifications made to Junction Box No. 4 allow secondary effluent to be directed either to the sand filters or to the chlorine contact tanks. Other supplemental improvements to the facility included the retrofitting of the older aeration tanks (Nos. 1 through 4) with the same style of fine bubble diffusers as the new basins in 2002, construction of a lime silo in 2004 adjacent to the aerated grit chambers, and modifications to the chlorination/dechlorination systems in 2006.

Because of issues with cyanide in the scrubber water effluent from the incinerator, a scrubber water treatment facility was constructed adjacent to the Effluent Filter Building. This scrubber became unnecessary when Synagro agreed to upgrade the existing dewatering and incineration facilities with new centrifuges and a new fluidized bed incinerator to replace the original multiple hearth units. Two sodium hypochlorite storage tanks and feed pumps are housed next to the scrubber water reactor tanks, and are currently used to dose sodium hypochlorite to the common influent channel of the sand filters.

Synagro constructed the new dewatering and incineration facilities under a revised long-term agreement with the City. The new dewatering facilities utilize centrifuges with essentially the same dewatering capacity. The cake receiving facilities were also installed. The capacity of the new fluidized bed incinerator is greater than the former multiple hearth incinerator to account for the additional merchant cake solids. The new facilities were completed in May 2007. Synagro is constructing a 2MW co-generation system, under an amendment to the long-term agreement with the City, that will provide renewable energy to the new electrical facilities at the treatment plant and Synagro's electrical system.

1.5. DISCHARGE PERMIT LIMITS

The effluent from the WWTF discharges directly to the Blackstone River approximately twenty five miles upstream from its outlet to Narragansett Bay. Woonsocket's RIPDES permit (No. RI0100111) was updated by the RIDEM effective October 1, 2008. A summary of the key effluent discharge limitations contained in the current RIPDES permit is provided in Table 1-1, and the complete permit is attached in Appendix B.

TABLE 1-1
RIPDES PERMIT EFFLUENT LIMITATIONS

Effluent Characteristic	Discharge Limitations			Monitoring Requirements	
	Average Monthly	Average Weekly	Maximum Day	Measurement Frequency	Sample Type
Flow	16 mgd	--	--	Continuous	Recorder
BOD (Nov 1 – May 31)	30 mg/L 4,000 lb/d	45 mg/L --	50 mg/L 6,670 lb/d	3/Week	24-hr. Comp.
TSS (June 1 – Oct 31) (Nov 1 – May 31)	15 mg/L 2000 lb/d 30 mg/L 4,000 lb/d	20 mg/L -- 45 mg/L --	25 mg/L 3,335 lb/d 50 mg/L 6,670 lb/d	3/Week	24-hr. Comp.
Total Phosphorus (Apr 1 – Oct 31) (Nov 1 – Mar 31)	0.1 mg/L 1.0 mg/L	-- --	-- --	3/Week	24-hr. Comp.
Total Ammonia, as N (Jun 1 – Oct 1) (Nov 1 – Apr 30) (May 1 – May 31)	2.0 mg/L 15 mg/L 12 mg/L	-- -- --	49.4 mg/L 53.8 mg/L 53.8 mg/L	3/Week	24-hr. Comp.
Total Nitrogen (April) (May 1 – Oct 31) (Nov 1 – Mar 31)	10 mg/L 3.0 mg/L 400 lb/d --	-- -- -- --	– – – –	3/Week 1/Month	24-hr. Comp.
Total Cadmium	0.66 ug/L	--	4.32 ug/L	2/Week	24-hr. Comp.

Conditions of the latest permit have been modified through Consent Agreement RIA-368 with RIDEM. The modified Consent Agreement includes interim effluent discharge limitations and monitoring requirements that reflect the best possible treatment for nutrient removal with the existing facilities. Effluent limits for nitrogen, phosphorus, and cadmium were modified until the facility planning process and construction of plant upgrades for nutrient removal have been completed. A summary of the key interim permit limits is provided in Table 1-2.

TABLE 1-2
INTERIM RIPDES PERMIT EFFLUENT LIMITATIONS

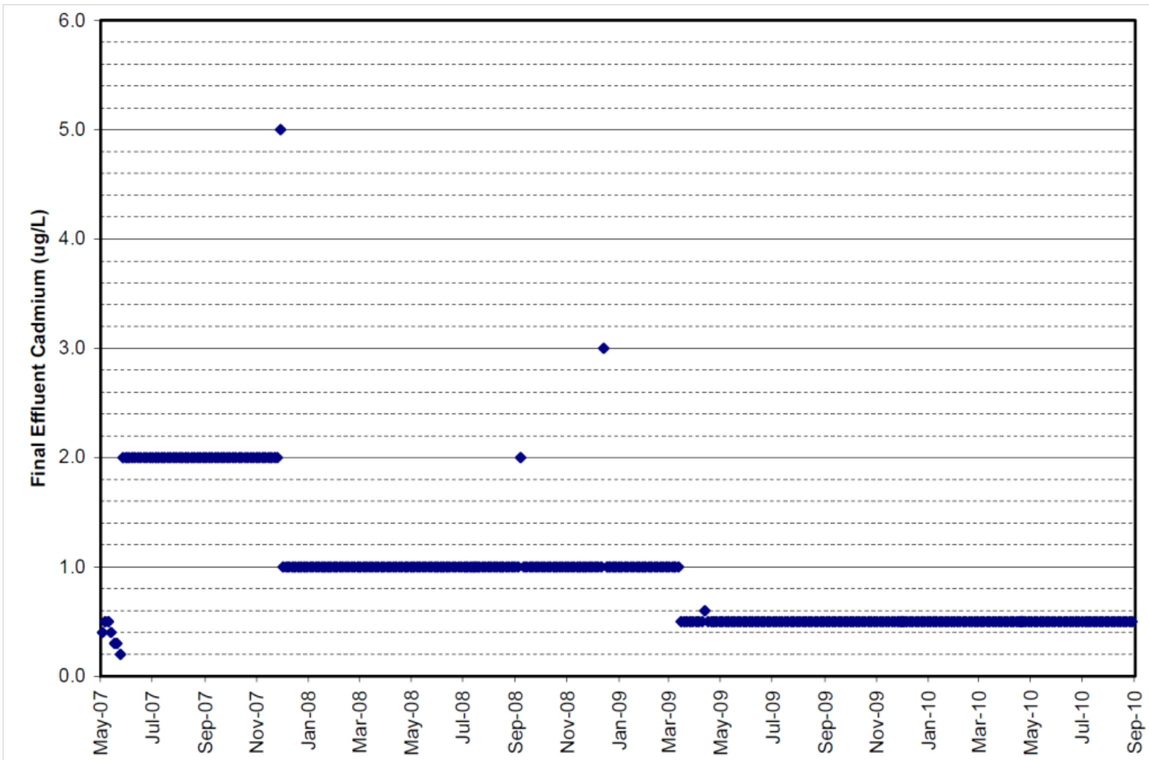
Effluent Characteristic	**Discharge Limitations**			**Monitoring Requirements**	
	Average Monthly	Average Weekly	Maximum Day	Measurement Frequency	Sample Type
Total Phosphorus (Apr 1 – Oct 31) (Nov 1 – Mar 31)	1.0 mg/L --	-- --	-- --	3/Week	24-hr. Comp.
Total Nitrogen (May – October)	10.0 mg/L	--	--	3/Week	Calculated
Total Cadmium	1.5 ug/L	--	4.32 ug/L	2/Week	24-hr. Comp.

1.6. TOTAL CADMIUM LIMIT

The latest RIPDES permit has a revised limit of 0.66 ug/L for effluent total cadmium, and identified a potential to exceed this limit based on monitoring data from January 2001 to December 2005. The monthly average final effluent cadmium was noted as 0.94 ug/L. An analysis of more recent data over the last 2 years reveals that average final effluent cadmium concentrations are below the revised permit limit 0.66 ug/L as shown in Figure 1-4, and there no longer appears to be a potential to exceed the revised effluent cadmium limit.

As can be seen in Figure 1-4, cadmium results have typically been at the detection limit. In March 2009, the analytical method used for measuring cadmium was changed, lowering the detection limit from 1.0 ug/L to 0.50 ug/L. Since the change in analytical methods, final effluent cadmium concentrations have consistently been below the detection limit of 0.50 ug/L. It is also important to note that the majority of the results in the RIDEM database from 2001 through 2005, the total cadmium level was at the detection limit. Consequently, based on the most recent results using methods with a lower detection limit, removal of cadmium was not addressed further in this Facility Plan Amendment.

FIGURE 1-4
TOTAL CADMIUM IN FINAL EFFLUENT (MAY 2007 – SEPTEMBER 2010)



1.7. IRON LIMITS

The allowable monthly average discharge level for iron is 4.1 mg/l, which is calculated using the same mass balance methodology as other water quality based limits in the Woonsocket permit. Assumptions used in this approach to determining water quality limits include the following:

- WWTF flow 16.0 MGD,
- 7-day 10-year (7Q10) receiving water flow is 102 cfs
- upstream concentration is 0 mg/L
- 80% allocation
- monthly average (i.e., chronic) criteria of 1.0 mg/l
- There is no acute (i.e. daily maximum) water quality criteria.

In the development of the recommended nutrient removal upgrades for the Woonsocket WWTF, CH2M HILL considered the monthly average water quality criteria of 4.1 mg/l for iron. Meeting this water quality criteria should not be a problem at the WWTF. The drinking water treatment plant in Woonsocket uses iron as part of the treatment process, which is thought to be mostly soluble and in the form of ferrous (Fe+2) iron. As the ferrous iron goes through the WWTF, it will all be oxidized to ferric iron (Fe+3), which is very insoluble in water. Thus, the only iron that will be present in the WWTF effluent is the fraction of the TSS from the filter effluent that is iron, which will be lower than the limit.

1.8. ORGANIZATION OF REPORT

This Facility Plan Amendment includes ten sections as follows:

- Executive Summary: provides an overview of the Facility Plan Amendment and its contents.
- Section 1 – Introduction: presents background information on the objectives of the facility planning process, the existing facility and treatment processes, and its RIPDES discharge permit limits.
- Section 2 – Existing Flows and Loads: presents a summary of existing flows and loads in the planning area based on data from 2007 to 2010 and additional data that was obtained in 2012 with special emphasis on the characteristics of the primary effluent including flows; BOD, TSS, nitrogen, and phosphorus loadings; wastewater temperatures; and alkalinity. The primary effluent flows and loads data will be used for evaluation of improvements needed to meet the revised nitrogen and phosphorus limits.
- Section 3 – Future Flows and Loads: presents a summary of future conditions in the planning area focusing on the future design flows and loads.
- Section 4 and 4A – Performance of the Secondary Treatment Facilities and Process Model Development: presents a summary of the development of the BioWin process model, developed by Wright-Pierce, and the Pro2D process model, developed by CH2M HILL. A description of the existing MLE treatment process is presented with a summary of existing operating data, and options for optimizing nitrification and denitrification processes to the extent possible. The process model development is presented along with modeling results for the calibration, and verification runs. A description of the two-stage sludge AB process and model development for the AB process is included.

- Section 5 – Screening of Nutrient Removal Alternatives: presents the preliminary screening of nutrient removal alternatives. Nutrient removal technologies for nitrogen and phosphorus are discussed including a preliminary cost analysis.
- Section 6 – Detailed Evaluation of Nutrient Removal Alternatives: presents a detailed evaluation of the short-listed nutrient removal alternatives outlined in Section 5. For each alternative, a detailed capital and O&M costs comparison of the short-listed treatment alternatives are presented. This section evaluates and ranks the selected alternatives.
- Section 7 – Development and Evaluation of Ancillary Wastewater Treatment Improvements: presents the development and evaluation of ancillary WWTF process improvements needed to support the new nitrogen and phosphorus removal processes. Evaluations of alternatives are provided for individual unit processes.
- Section 8 – Evaluation of Building and Support Systems: presents the development and evaluation of building and support system improvements. This section includes evaluations of alternatives for architectural, structural, HVAC, instrumentation, and electrical improvements to be incorporated in the WWTF upgrade.
- Section 9 – Plan Selection: presents the selected plan for improvements at the WWTF. A detailed description of all process and building improvements is also provided. Preliminary layout of proposed improvements, estimated costs (both capital and operational), and the environmental impact assessment are included.
- Section 10 – Plan Implementation: presents the implementation schedule for the recommended plan. Institutional responsibilities, public workshop awareness, and a tentative schedule of improvements are addressed.

SECTION 2
EXISTING FLOWS AND LOADS

2.1 INTRODUCTION

This section describes the assessment of the existing flows and loads to the Woonsocket Regional WWTF. As a regional wastewater treatment facility, the plant receives wastes from the City of Woonsocket as well as the neighboring towns of Bellingham, MA, Blackstone, MA, and North Smithfield, RI. As documented in the 2000 RIDEM-approved Facility Plan Amendment, the current facility has a design flow capacity of 16.0 mgd, maximum monthly flow capacity of 16.0 mgd and peak hourly flow capacity of 32.0 mgd. The plant currently receives an average daily influent flow of approximately 7.8 mgd. The design criteria include allowances for the significant internal recycle flows, most notably from the privately-operated Solids Handling Facilities, which result in significantly higher primary influent flows and loads than in the raw influent. An overall process flow schematic is shown in Figure 1-1 that identifies the key recycle flows.

The primary focus of this facility planning effort is to determine the improvements needed to provide enhanced effluent nitrogen and phosphorus removal to the levels required in the new RIPDES discharge permit. Under this new permit, the facility is now required to meet a more stringent seasonal limit of 3.0 mg/L total nitrogen and 0.1 mg/L total phosphorus. CH2M HILL used a steady state whole plant simulator, Pro2D, which was developed by CH2M HILL to perform complete wastewater treatment plant simulations and to calculate full-plant mass balances. Pro2D uses Microsoft Excel as its computational engine, implemented as a series of worksheets in a Microsoft Excel workbook. Biological modeling is based upon IWA's industry leading ASM activated sludge model. The use of Excel as a computation engine is not proprietary, but the contents of the workbook are. Pro2D guides the user through the sizing of unit process structures and equipment, in part by showing standard sizing criteria adjacent to calculated information from the Pro2D mass balance. However, Pro2D does not inherently size the tanks and equipment; rather, Pro2D maintains a plant mass balance and predicts the performance of all unit processes and for the plant effluent using structure and equipment sizes that are provided by the user. Additionally, process modeling using the BioWin process model has been used by Wright- Pierce to assess the performance of several of the treatment alternatives. The CH2M HILL recommended alternatives were modeled in Pro2D. The secondary influent loadings are an important factor in evaluating enhanced nitrogen and phosphorus removal requirements. CH2M HILL's analysis includes evaluation of wastewater characteristics and raw influent flows and loading data with appropriate recycles.

As noted above, the purpose of this facility planning effort was to assess the process improvements needed to meet the revised nitrogen and phosphorus limits in the new RIPDES discharge permit. Consequently, an analysis of the existing nitrogen and phosphorus data was carried out in addition to biochemical oxygen demand (BOD_5) and total suspended solids (TSS) loadings. The seasonal variation in wastewater temperature was also evaluated, since wastewater temperature is critical in establishing process requirements for nitrification. Alkalinity levels were also reviewed, because of the impact on nitrification and biological phosphorus removal.

The WWTF accepts minimal quantities of septage, which are blended into the raw influent flow, and measured as part of the influent flows and loads. It is not anticipated that septage volumes will increase significantly. Since the impact of current and projected septage volumes is negligible, a detailed evaluation of flows and loads from septage was not performed.

The facility includes internal recycle or return flows and loads from the Solids Handling Facilities located onsite. The Solids Handling Facilities are operated and maintained by Synagro under a long-term agreement with the City. Under this agreement, Synagro is responsible for the operation and maintenance of a liquid sludge storage tank, gravity thickener, centrifuge dewatering system, cake sludge receiving facility, and fluidized bed incinerator.

As indicated in Figure 1-1, all primary and secondary sludge generated by the WWTF is pumped as a liquid with approximately 1-3 percent solids and directed to the gravity thickener for thickening, dewatering and incineration. Synagro accepts other liquid sludge from outside sources as well, and these are directed to the liquid sludge storage tank. There is no overflow from the sludge storage tank and all liquid waste within the tank is pumped through the centrifuge dewatering facilities. The centrate from the dewatering operation is directed to the gravity thickener. This is intended to promote additional TSS removal and thus to minimize the recycle loadings back to the influent of the WWTF. The Solids Handling Facilities also have a large return flow from the incineration operation, primarily from the tray scrubber. This return flow does not go through the gravity thickener; instead it is routed directly to the common recycle drain which is conveyed to the influent of the treatment plant. The traveling bridge effluent sand filter backwash also returns flows to the common recycle sewer line or plant drain.

The flows and loads for the Wright-Pierce March 2011 Facility Plan Amendment were developed based on analysis of historic monthly Discharge Monitoring Reports (DMRs) and other daily operating data over a three year period, primarily from May 1, 2007 through September 30, 2010. This data was supplemented with additional wastewater characterization data obtained from plant staff in July-September t 2012 for the future flows and loads analysis provided in Chapter 3. This data also provided additional detailed characterization to be used for updating process modeling. The compiled data was analyzed for comparison to the following design conditions as follows:

- ***Annual Average*** **–** This is the average of all daily data for the entire period. This is important for benchmarking of capital and operation and maintenance cost issues, but is not a key design parameter.
- ***Maximum Month*** **–** This is the maximum 30-day moving average during the analysis period, and is the key sustained-flow design load criteria. As presented in subsequent sections, the maximum month value has been calculated for flow, BOD_5, TSS and nutrients independently, but does not necessarily occur during the same 30-day period for each parameter or for the same parameter when comparing raw loads and recycle loads. The maximum monthly data is the key sustained-flow design load condition, but it is important to be careful in applying the data for systems that may be affected by multiple parameters (e.g. flow, BOD, and TSS) such as the secondary and tertiary treatment systems. Since the maximum monthly flow, BOD, and TSS do not typically occur at the same time, it is important to utilize actual data in secondary treatment system analyses to avoid excessive conservatism. Uniquely for this facility, because of the Synagro operations, this means that it is not simply possible to add up the maximum month load for raw and recycles to get the primary influent load, since Synagro peak loads are independent of raw sewage peak loads.
- ***Maximum Week*** **–** The 98^{th} percentile value for loading conditions is sometimes considered acceptable for certain design considerations. It is equivalent to the

maximum week loading condition, and is normally used for sizing solids handling systems and for setting peak aeration demands.

- ***Maximum Day*** **–** This is the maximum for flow, BOD_5, and TSS that occurred in a single day during the analysis period. As with the maximum month, the maximum day condition does not necessarily occur on the same day for each parameter. In some instances the 98^{th} and 100^{th} percentiles for the maximum day loading for BOD_5 and TSS are considered when evaluating and designing wastewater treatment systems.
- ***Peak Hour*** **–** This is an important parameter for hydraulic capacity of the total influent flow to the WWTF.

As stated previously, the period selected for analysis of historic operating data begins in May 2007, which corresponds to the startup of the new fluidized bed incinerator at the Solids Handling Facilities. The start-up of the new incinerator corresponded with additional process improvements in the operation of the Sludge Handling Facilities.

During the analysis of available operating data, there were several anomalous conditions discovered either in terms of issues with the existing monitoring data or the actual operating conditions. Engineering judgment was used to exclude inappropriate data and produce an accurate basis for design criteria. For example, working closely with plant operations staff and the City while evaluating historic operating data, it was determined that the final effluent flow meter was notably out of calibration from June 3, 2009 thru December 4, 2009, and consequently this period has been excluded in the analysis of historic influent flows and loadings, but not primary influent/effluent loadings, which are based on the primary effluent flow meter. In addition, plant operations staff reported that one of the two primary clarifiers was disabled from March 16, 2010 to April 23, 2010, and consequently the primary effluent data from this period were excluded in the analysis of primary effluent loadings.

Another key consideration for the data period is that an extraordinary wet weather flood event that occurred on March 30 and 31, 2010. The peak day influent flow value recorded during this elevated wet weather flow event followed a series of notable peak wet weather events earlier in the month that created the necessary pre-condition for the flooding including high groundwater levels. The overall monthly rainfall in March 2010 was 13.62 inches in Woonsocket, and this included 6.23 inches from March 29 through March 31, 2010. Industry standard design guides such as TR-16 require that the WWTF be designed to maintain uninterrupted operation at flows up to a 25-year flood event, and that the facility be protected from damage during a 100-year flood event. The referenced March 30 and 31 flood event corresponded to the fifth highest historic crest in the Blackstone River, since records were established in 1929. By comparison, the peak flow to the WWTF of 29.1 mgd on March 30, 2010 exceeded the peak flow of 26.5 mgd on October 16, 2005. The October 2005 wet weather event resulted in the second highest crest on record, and occurred during a month with 15.73 inches of rainfall. The higher peak flows during the March 30, 2010 event are believed to be due to higher groundwater levels associated with the snow melt during the month (especially since Woonsocket had completed a notable inflow removal project in the interim, as discussed later in Section 2.2.7). It is believed that the March 30/31, 2010 flood exceeded the 25-year flood event given the antecedent conditions, and approached a 100-year frequency of occurrence. It is notable that the March 30/31 flood was considered to exceed a 100-year event in other parts of the state including downstream of Woonsocket in the Blackstone River Basin. Consequently, peak flows excluding the March 30, 2010 wet weather flow event were documented as well as the peaks on that date for assessment of the necessary design criteria.

As will be discussed further below, the City of Woonsocket has recently implemented an Underground Asset Management program that is expected to result in peak flow reductions through removal of inflow and infiltration. Although this work is in a preliminary stage, a number of important projects have been completed recently or will be in the near future. The impact and priority of these projects was the subject of a supplemental Inflow and Infiltration Program Paper that is attached in Appendix C, and discussed further in Section 2.2.7 below. The impact of these projects will be considered as part of the assessment of future growth needs in Section 3.

Industrial discharges and loadings are an important aspect of the flows and loads at the WWTF. Recently, the largest industrial user, Technic was consulted as to their current and future operations. This meeting established future flows and loads from the facility which were incorporated into the WWTF flows and loads analysis and the Local Limits analysis. The City is currently working with the industry to determine how BOD discharge will be addressed such as controlling the source; pretreatment at the industry; or treatment at the WWTF. The industry has been working on reducing the amount of biochemical oxygen demand load that they are discharging to the extent that the peak loads in 2010 and 2011 have been reduced by changes in their operations. The industry indicated during a conference call on November 2, 2012 that they would be modifying their process therefore reducing the loads to the WWTF. The recent information has been taken into consideration on the evaluation of future flows and loads to the WWTF. The discharges from the industry have a carbon to nitrogen ratio that is beneficial to nutrient removal at the WWTF. The data provided in August and September 2012 by the industry has allowed an evaluation to be included within the Facilities Plan and to make an assessment of the loads and characteristics of the loads.

The historical analysis of BOD load data was therefore examined, with the goal of removing any data that was unduly impacted by the high BOD discharges of Technics prior to the process changes. This was done by removing all BOD data that had a BOD/TSS ratio of greater than 2.0, which was the design point the team determined best reflected the industrial inputs to the facility.

Lastly, the Synagro operation is susceptible to upsets of their ash thickener which results in extremely high TSS loads being returned to the plant over short periods. However, these TSS loads settle out very readily in the primary clarifier, and do not impact plant capacity, other than in the sizing of the primary sludge pumps. These anomalous TSS (ash) events were therefore excluded from the recycles load analysis since they have little to no impact upon the design.

2.2 FLOWS

Table 2-1 summarizes the average, maximum monthly, peak daily, and peak hourly flows through the Woonsocket Regional WWTF for the period from May 2007 through September 2010, excluding the March 30, 2010 flood event.

The WWTF has two flow meters. One is a Parshall flume located downstream of the Primary Effluent Pump Station, and the second uses ultra-sonic measurement of the depth over a sharp-crested weir at the chlorine contact tanks for final effluent. The current facility does not have a raw influent flow meter. Consequently, the final effluent flow rate is considered to be the best source of information available to determine the raw influent flow rate at the WWTF. Peak hourly flow for each day is not recorded for the final effluent, only for the primary effluent flow meter due to the attenuation across the plant, which is higher than normal due to multiple pumping steps and the effluent filtration system. Consequently, the peak hourly influent flow rate was estimated from primary effluent flume data. Figure 2-1 shows the daily flow data for the primary effluent, the final effluent, and recycle as well as the daily precipitation data, and Figure 2-2 shows the 30-day moving average for each of these flows.

2.2.1 Recycle Flows

The recycle flow as shown in Figures 2-1 and 2-2 has been determined by subtracting the final effluent flow from the primary effluent flow, rather than by direct measurement. Thus, any error in the flow measurement of either the primary effluent or the final effluent manifests in the reported recycle flows. The method of measuring recycle flows accounts for any return flows drawn from downstream of the primary effluent Parshall flume that are returned to the plant drain including all plant water uses, waste activated sludge, secondary scum, and filter backwash. This method of determining recycle flows does not account for primary sludge, or outside sources of flow such as centrate from dewatering of liquid merchant sludge at the Solids Handling Facilities and any inflow and infiltration to the plant drain system. Table 2-1 shows the average, maximum month, and peak daily recycle flows based on analysis of the flow rates determined from the difference between the two flow meters. For comparison, Table 2-2 shows an analysis of the anticipated total recycle flows based on flow measurement and assessment of the individual recycle flow sources as follows:

TABLE 2-1
SUMMARY OF DESIGN AND CURRENT FLOWS

Item	Existing Design 2000	Current 2010[a]
Raw Influent Flows (MGD):		
Average Annual	16	7.8
Maximum Monthly	16	13.0[b]
Peak Daily	24	22.2[b]
Peak Hourly	32	~32[b]
Recycle Flows (MGD):		
Average Annual	2.6	3.9
Maximum Monthly		5.76
Peak Daily		5.7- 8.5
Peak Hourly		-

Item	Existing Design 2000	Current 2010[a]
Primary Influent Flows (MGD):		
Average Annual	18.6	11.7
Maximum Monthly	18.6	18.1[c]
Peak Daily	26.6	26.2[c]
Peak Hourly	34.6	35[c]

Notes:

[a] Based on data from May 2007 through September 2010.

[b] The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data resulted in a raw influent maximum monthly flow of 15.8 mgd, peak daily flow of 29.1 mgd, and peak hourly flow of approximately 38 mgd.

[c] The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data resulted in a primary influent maximum monthly flow of 22.3 mgd, peak daily flow of 34.6 mgd, and peak hourly flow of approximately 42 mgd.

TABLE 2-2
SUMMARY OF RECYCLE FLOW SOURCES

Item	Average, mgd	Peak Daily, mgd
Recycle Flows as reported by Synagro		
Incineration Facilities, S-1	1.9	2.3
Gravity Thickener Overflow, S-3	0.9	1.2
Secondary Scum	~0.1	~0.1
Effluent Filter Backwash	0 / ~0.4	~1.5
Total	2.9 to 3.3	~5.1
Total as measured by difference in flow meters	2.5 to 2.9	~4.7

FIGURE 2-1
PRIMARY EFFLUENT, FINAL EFFLUENT AND RECYCLE FLOWS
(MAY 2007 – SEPTEMBER 2010)

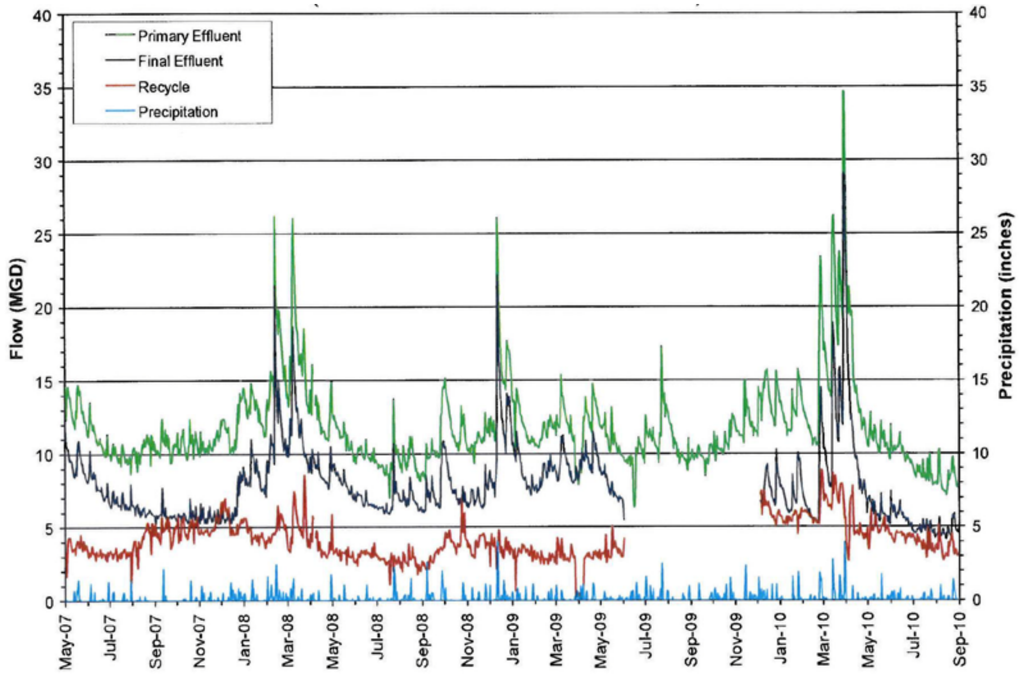
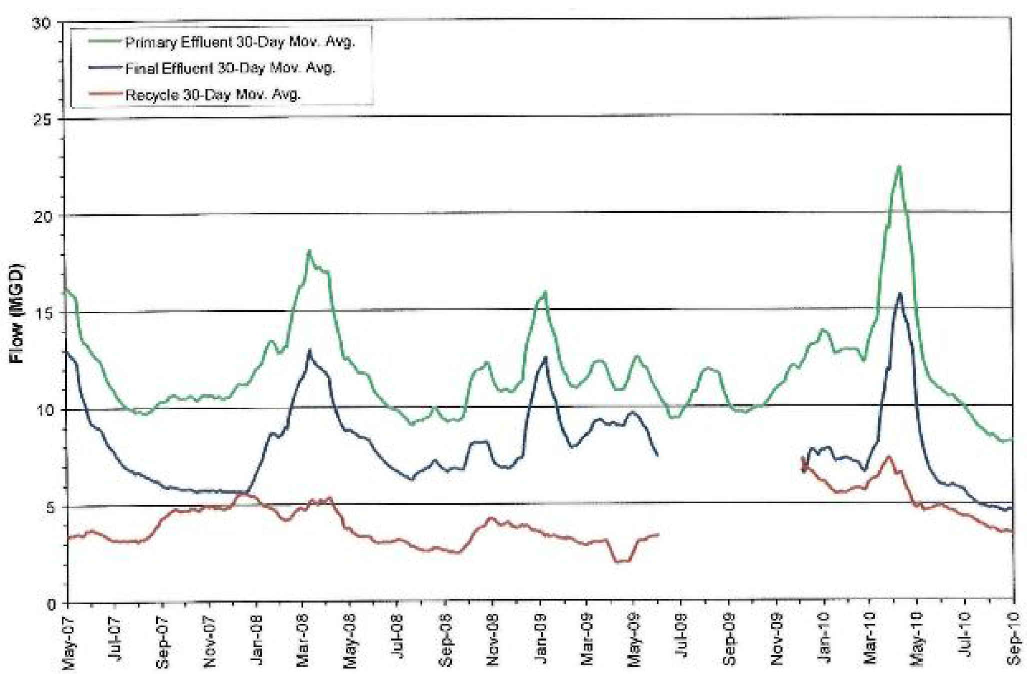


FIGURE 2-2
30-DAY MOVING AVERAGE FLOWS (MAY 2007 – SEPTEMBER 2010)



- **Solids Handling Facilities –** The solids handling facilities are operated by Synagro and have recycle flows as shown in figure 1-3 that include the following:
  - Incinerator Facilities, (S-1) – The recycle flows from the sewage sludge incineration facilities are referred to as the Combined Flow, S-1, and include the discharge from the venturi and tray scrubbers that treat the incinerator exhaust air as well as the overflow from the ash thickener. Synagro has reported an average flow rate of 1.94 mgd, and a peak hourly flow rate of 2.3 mgd based on their monitoring of these discharges. These rates are consistent with the design heat and mass balances for the incinerator. The plant effluent is used as the scrubber water for both the venturi and tray scrubbers. The heat and moisture balance predicts an increase of about 66,000 gpd in the discharged flow compared to the plant water flow when condensate and water losses are considered.
  - Centrate, S-2 – Centrate is generated from the dewatering of Woonsocket and other merchant liquid sludge delivered by trucks to the facility. Centrate is discharged to the Centrate Pump Station which has a maximum capacity of 1.0 mgd (700 gpm) with two pumps operating, but typically operates with only one pump cycling on and off. In addition to the centrate from the dewatering operation, the Centrate Pump Station handles the washwater from the cake receiving and dewatering area and the blowdown from an odor control scrubber for the dewatering area. The pump station discharge is directed to the gravity thickener, and the flow is discharged as part of the gravity thickener overflow. Synagro has reported an average centrate flow rate of 0.35 mgd and peak hourly rate of 0.58 mgd. The Woonsocket primary sludge includes about 4 dry tons per day that are attributable to Woonsocket influent flows, and the waste activated sludge has averaged about 6.6 dry tons per day. This corresponds to about 27% of the average 39.2 dry tons per day associated with the liquid sludge delivered to dewatering. The centrate from liquid merchant sludge is estimate at 250,000 gpd.
  - Gravity Thickener Overflow, S-3 – The gravity thickener overflow includes the primary and secondary sludge from the Woonsocket WWTF; the discharge from a local sump pump; plant water to help freshen the gravity thickener; and the centrate from the dewatering operation. The Woonsocket primary sludge flow

accounts for an average of 90,000 gpd, and the waste activated sludge has averaged 80,000 gpd. The local sump pump includes the blow down flow from the packed bed scrubber treating the exhaust air of the gravity thickener and adjacent merchant sludge storage tank. The gravity thickener overflow rate is measured by a flow meter that was installed during the spring of 2010. The available data from June 9th through November 29th indicates an average overflow rate of about 0.91 mgd, a maximum monthly flow of 0.94 mgd, and a peak daily flow of 1.22 mgd.

- A final potential source of recycle flows is inflow and infiltration to the plant drain system. Synagro reportedly has one catchbasin, which is located in front of a truck bay that is tied into the recycle flow system (believed to be tied into S-2). It appears appropriate that this catchbasin is connected to the recycle flow system for spill containment, and it should have only limited impact on peak flows. The plant drain line has been inspected with CCTV, and did not appear to have significant infiltration. Any I/I contributed to the plant drain system will not be differentiated by the current method of estimating recycle flows by the difference in flow meters.
- The available data indicates that overall recycle flows from the Solids Handling Facilities are in the range of 2.8 MGD with peaks in the range of 3.5 MGD. The peak flow rates are only impacted by wet weather events by the one catchbasin, and to the extent that there is an increase in primary scum and sludge quantities. Plant water is used as the supply water that ultimately results in the majority of the recycle flows from the Solids Handling Facilities including the feed water to the incinerator scrubbers, the feed water to the odor control scrubber, freshening water for the gravity thickener, and wash down water in the dewatering area and elsewhere. The plant water pumps serving Synagro have a maximum capacity of 2.88 mgd (2,000 gpm), which limits the potential magnitude of peak recycle flows. The other sources of recycle flow include the Woonsocket primary and secondary sludge; the centrate from merchant sludge; and inflow and infiltration sources to the plant drain system. These are all discharged back to the WWTF through the gravity thickener overflow.

  - The current method of estimating recycle flows by the difference between flow meters does not account for approximately 250,000 gpd of centrate from dewatering merchant sludge and approximately 90,000 gpd from dewatering of primary sludge; approximately 66,000 gpd of condensate from the incinerator exhaust; and inflow and infiltration to the plant drain system. Excluding the inflow and infiltration, these sources contribute a total of about 406,000 gpd of the flow from the Solids Handling Facilities, and an estimated 316,000 gpd of this contributes to the final effluent flow (i.e. excluding the primary sludge flow). If the 406,000 gpd from these sources is excluded from the estimated total recycle flows from the Solids Handling Facilities in Table 2-2, the remaining portion is about 2.5 to 2.9 mgd that should included in the current estimates based on the difference between flow meters.

- **Secondary Scum** – The secondary scum flow is returned to the plant drain for subsequent removal at the primary clarifier, where the primary scum is pumped to the sludge storage tank. The secondary scum flow is estimated conservatively high at 0.1 mgd. However, it is important to note that the scum trough for secondary clarifier No. 3 was submerged for an extended period, and repaired in the Spring of 2010. During this time frame, the flow from the scum troughs may have been much higher. For this reason, the recycle flows reported in Table 2-1 are based on the period from May 2007 through June 2009.

- **Backwash from the Effluent Sand Filters** – The sand filter backwash is also returned through the plant drain system. The effluent filtration system is a traveling bridge type that includes four units each with a backwash rate of 0.77 MGD or a combined capacity of 3.1 MGD. Because the effluent filters are not needed to meet the cold weather permit limits, the effluent filters are typically taken off-line during the cold weather season. The average backwash rate is estimated conservatively high at about 0.4 mgd, and the peak rate at about 1.5 mgd assuming that two units are backwashing at one time.

The secondary scum and backwash contribute to those flows measured by the method of estimating recycle flow rates as shown in Table 2-1 (determined by the difference in primary effluent and final effluent flow meters). The recycle flows reported in Table 2-1 are based on the period from May 2007 through June 2009 to exclude the time period when the scum troughs were submerged. The estimated recycle flows in Table 2-1 are notably higher than those determined by assessment of the individual sources in Table 2-2. The measured average recycle rate of 3.9 mgd is about 1.0 mgd higher than the corresponding estimated average recycle flow determined by assessment of the individual sources, and the gap appears to increase when the maximum month and peak daily values are considered. A variety of potential explanations were considered, in part the apparent cause of this discrepancy could be the differences in the accuracy of the two flow meters.

For example, the recycle flows, as shown in Figures 2-1 and 2-2, should follow a trend of higher return flows during the warm weather season when the effluent filters are on-line, and then dropping on average during cold weather conditions when the effluent filters are off-line. The data for the recycle flow in Figures 2-1 and 2-2 do not display the expected seasonal trend, which suggests that the sand filter backwash flows are estimated conservatively high, since higher backwash rates would have a more notable impact on recycle flows.

Overall, there is a significant unexplained difference between the estimated recycle flows based on the flow meters and by analysis of the sources. Veolia reported that the meters are recalibrated on an annual basis, but both present difficulties in calibration. The primary effluent flow rate is the key parameter for evaluation of the secondary system for meeting the enhanced nitrogen and phosphorus removal requirements. Since there is no data to determine whether the primary effluent or final effluent flow meter is more accurate, the primary effluent flow meter data has been used to estimate secondary influent flows.

It should be noted that the recycle flow allowance in the design criteria for the last upgrade was an average rate of 2.6 mgd. Thus, the existing recycle flows are higher than the design basis by at least 0.5 mgd on average based on the estimated return flows, and by 1.3 mgd based on the difference between flow meters.

2.2.2 Peak Hourly Flow

Figure 2-3 shows the daily average and peak instantaneous primary effluent flow, and Figure 2-4 shows the frequency distribution curve for daily average and peak instantaneous primary effluent flow as well as the daily final effluent flow.

As previously noted, the raw influent pump station handles both the raw influent and recycle flows using a total of five pumps. The peak hourly flow at the primary clarifiers is limited by the peak pumping capacity of the raw influent pump station, which is estimated to be approximately 42 mgd. The primary effluent Parshall flume was set up and calibrated for a maximum flow of 40.9 mgd over the period from May 2007 through September 2010, but has recently been recalibrated to measure up to 48 mgd. The primary effluent and final effluent pump stations have slightly greater capacity than the raw influent pump station, and thus do not limit the allowable flow through the WWTF.

Since the primary effluent flow data showed repeated peak instantaneous flow events at the maximum flow rate, the instantaneous flow measurements were reviewed in detail for all recent peak flow events. For the March 30, 2010 flood event, the recorded peak hourly flow was sustained at a maximum rate of 42 mgd, while for all other elevated flow events, the peak was an instantaneous event caused by the fifth raw influent pump coming on-line. For these other events, the peak hourly primary effluent flow was determined to be 35 mgd or less.

As shown in Table 2-1, the existing design basis for peak hourly raw influent flow to the WWTF is 32 mgd. Since recycle flows were anticipated to contribute an additional 2.6 mgd from solids handling and the effluent filters, the expected peak hourly primary effluent flow was 34.6 mgd for the existing facility design. However, assuming an actual current recycle flow of 3 to 5 mgd, the best estimate of the current peak hourly raw influent flow entering the WWTF is approximately 30 to 32 mgd, which is at the current design capacity.

The frequency distribution for peak hourly flow in Figure 2-4 indicates a plateau at about the 98th percentile. As noted above, detailed evaluation of the peak instantaneous flow data indicates that the fifth pump only activates for short periods of time.

FIGURE 2-3
DAILY AND PEAK INSTANTANEOUS PRIMARY EFFLUENT FLOWS
(MAY 2007 – SEPTEMBER 2010)

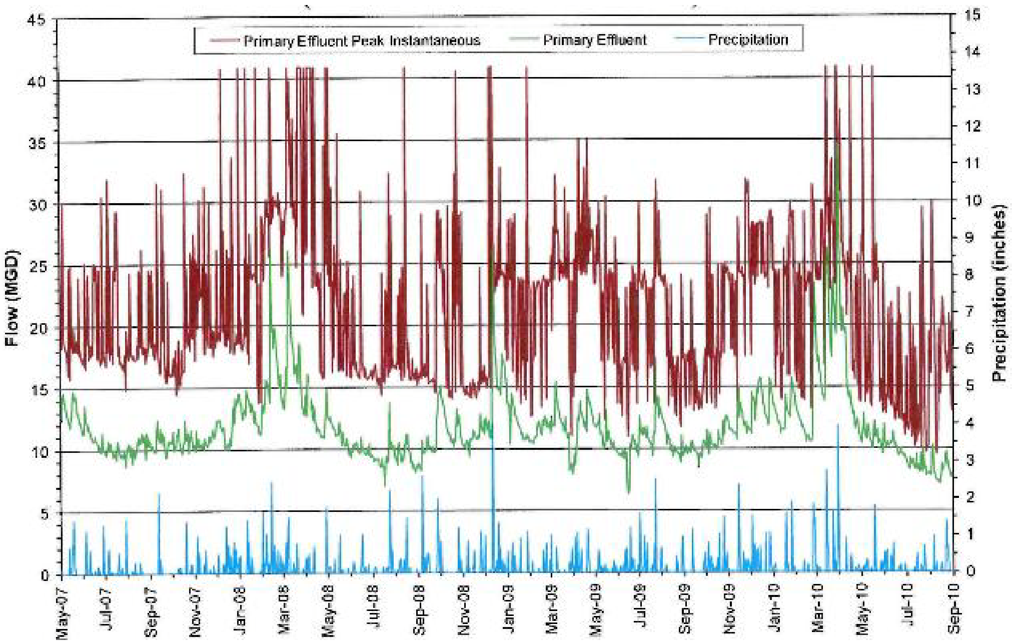
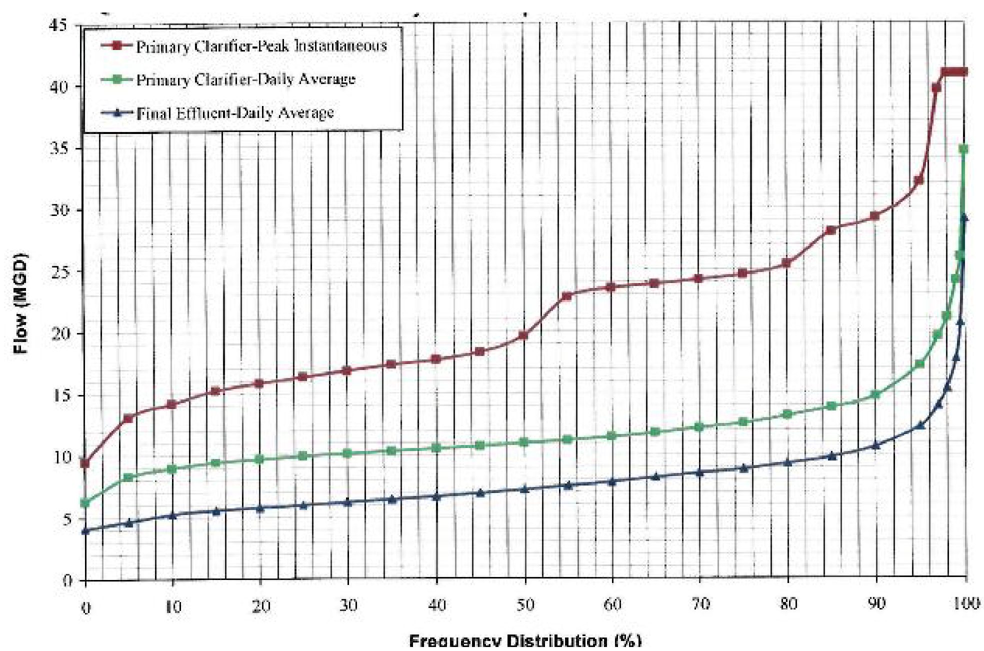


FIGURE 2-4
FREQUENCY DISTRIBUTION FOR – FLOWS (MAY 2007 – SEPTEMBER 2010)



2.2.3 Peak Daily Flow

For the peak daily raw influent flow, the historic peak of 29.1 mgd occurred during the March 30, 2010 flood event. Excluding this date, the peak daily flow was in the range of 22 mgd reaching this level on February 13, 2008; March 9, 2008, December 12, 2008, and March 16, 2010 (before actual flood event).

For the primary effluent, the peak daily flow was 34.6 mgd during the March 30, 2010 flood event, and ranged between 26.0 and 26.2 mgd during the four peak flow events noted above. As shown in Table 2-1, the peak daily raw influent flow of 22.2 mgd (excluding the March 30, 2010 flood) is below the design value of 24 mgd, and the primary effluent flow of 26.2 mgd is below the design value of 26.6 mgd.

Based on the 2000 RIDEM-approved Facility Plan, the peak daily flow for the period from July 1995 to June 1999 was 19.8 mgd. Thus, it appears that peak flows entering the facility have increased by about 2.4 mgd. This increase is not alarming as it is often typical of an aging collection system. An increase would be expected over time as a collection system ages. The 2000 RIDEM-approved Facility Plan noted the significant impact of high groundwater levels on flows to the WWTF. The impact of recent inflow and infiltration removal projects will be discussed in Section 2.2.7 below.

2.2.4 Maximum Month Flow

As noted in Section 2-1, the maximum month flow is defined as the 30-day moving average, and is the key sustained-flow design condition. Figure 2-2 shows the 30-day moving average flow for both the primary effluent and the final effluent.

The highest monthly flow rate recorded over the data period occurred in April 2010 following the March 30, 2010 flooding event, but additional peaks also occurred in March 2008 and January 2009. Following the March 30, 2010 flooding event, the maximum 30-day moving average primary effluent flow was approximately 22.3 mgd, while the final effluent maximum 30-day flow was approximately 15.8 mgd. It is important to note that this is also the period where the trough of the secondary clarifier scum collector was submerged, which further increased the primary effluent value. The peak for the remainder of the period was in March 2008 as shown in Table 2-1 with a primary effluent maximum month flow of approximately 18.1 mgd and a final effluent maximum month flow of 13.0 mgd. This raw influent maximum month flow of 13.0 mgd (excluding the March 2010 flood) is below the WWTF's current licensed design value of 16.0 mgd, and the primary effluent maximum month flow of 18.1 mgd is below the design value of 18.6 mgd.

2.2.5 Average Flow

As shown in Table 2-1, the average raw influent flow over the period from May 2007 through September 2010 was approximately 7.8 mgd. Generally speaking, this average influent flow rate does not change if the period from March 30 to April 1, 2010 is excluded from the data set, even though during this three-day period the average daily flows were greater than 22 mgd.

It is important to note that the existing design basis for the WWTF has generally been reported as an **annual average** flow of 16.0 mgd both in the 2000 RIDEM-approved Facility Plan Amendment, and in the original Intergovernmental Agreements between Woonsocket and the member communities of Bellingham, North Smithfield, and Blackstone. However, the RIPDES discharge permit is based on a **maximum monthly** flow of 16.0 mgd. A review of available information on the WWTF design basis does not indicate that the maximum monthly flow was defined separately, so 16 mgd has been shown for both flow conditions in Table 2-1. In accordance with industry published design manuals, the maximum monthly flow value is used for design of the sustained-flow treatment capacity.

It should be noted that in the 2000 Facility Plan Amendment the average flow was noted as about 8.7 mgd with maximum monthly flows of greater than 13 mgd during times of the year with high ground water tables. The recorded decrease in the annual average raw influent flow to 7.8 mgd is attributable to the loss of several large industries in Woonsocket. The maximum monthly flow has remained in the same range suggesting that infiltration during periods of high groundwater has increased as would be expected over time.

2.2.6 Breakdown By Member Community

The breakdown of existing flows by community was analyzed to help with the assessment of future growth allowance and flow allocations. Table 2-3 summarizes the existing design flow allocation, and provides a breakdown for each community for a number of recent maximum month and peak daily flow events at the WWTF. Woonsocket's flows are determined by the difference between the WWTF flow and the other community flows, which are individually metered.

TABLE 2-3
BREAKDOWN OF RAW INFLUENT FLOWS BY COMMUNITY
(MAY 2007 – SEPTEMBER 2010)

	Belling-ham	N. Smithfield	Blackstone	Woonsocket[a]	WWTF
Flow Allocation	0.80	1.92	0.48	12.8	16.0
Average	0.11	0.66	0.26	6.8	7.8
Maximum 30-Day (1-11-2009)	0.16	0.96	0.34	11.1	12.5
Maximum 30-Day (3-14-2008)	0.15	1.01	0.30	11.5	13.0
Peak Daily (2-13-2008)	0.22	0.92	0.25	20.1	21.5
Peak Daily (3-9-2008)	0.22	1.05	0.47	17.0	18.7
Peak Daily (12-12-2008)	0.23	1.06	0.30	20.6	22.2
Peak Daily (3-16-2010)		1.08	0.62	~17	18.9
Independent Maximum 30-Day[b]	0.47	1.37	0.55		15.8
Independent Peak Daily[b]	0.48	3.53	1.13		29.1
Peaking Factors to Average					
Average	1.0	1.0	1.0	1.0	1.0
Maximum 30-Day (1-11-2009)	1.4	1.5	1.3	1.6	1.6
Maximum 30-Day (3-14-2008)	1.3	1.5	1.2	1.7	1.7
Peak Daily (2-13-2008)	1.9	1.4	1.0	3.0	2.8
Peak Daily (3-9-2008)	1.9	1.6	1.8	2.5	2.4
Peak Daily (12-12-2008)	2.0	1.6	1.2	3.0	2.8
Peak Daily (3-16-2010)		1.6	2.4	2.5	2.4
Independent Maximum 30-Day[b]	4.1	2.1	2.2		2.0
Independent Peak Daily [b]	4.2	5.4	4.4		3.7

Notes:

[a] Woonsocket flows are determined by difference between the WWTF flow and the metered flows from Bellingham, N. Smithfield, and Blackstone.

[b] Historic peak flow data for each community on separate dates from the historic peaks for the WWTF flows.

The flow allocation for each community is based on a 1999 agreement between the City of Woonsocket and the communities contributing wastewater flow. The Intergovernmental Agreements between Woonsocket and the member communities of Bellingham, North Smithfield, and Blackstone are currently being conformed to reflect these changes. The revised flow allocations are a monthly average limit, and thus the overall flow allocation of 16 mgd is consistent with the RIPDES discharge permit.

In terms of average flow, the flows from the member communities have increased notably in comparison to the values in the 2000 Facility Plan Amendment. Bellingham's average flow has increased from 0.04 mgd to 0.11 mgd. North Smithfield has had the most limited increase from 0.63 mgd to 0.66 mgd. Blackstone has increased from 0.16 mgd to 0.26 mgd. By comparison, Woonsocket's average daily flow decreased from 7.9 mgd to about 6.8 mgd. This is attributed primarily to the loss of a major industrial discharge.

For each of the maximum monthly flow events at the WWTF, the flows from Bellingham, North Smithfield, and Blackstone were well within the flow allocation in the pending Intergovernmental Agreement. The breakdown for the two maximum month WWTF events in Table 2-3 (excluding the March 30, 2010 flood event) suggests that Woonsocket's current maximum monthly flow is in the range of 11.1 to 11.5 mgd, which is within its flow allocation as well.

The historic maximum month and peak daily value for each community independent of the maximum/peak events at the WWTF are also shown in Table 2-3. For the independent peak conditions, the WWTF influent flow is based on the March 30, 2010 flood event. For Bellingham, the independent maximum month and peak daily event was in May 2007, but the flow meter was reportedly off-line in March 2010. For North Smithfield, the peak daily event was on March 24, 2010, and the maximum month occurred on April 7, 2010. For Blackstone, the peak daily event was on the March 30, 2010 flood event, and maximum month was on April 13, 2010. Thus, for both North Smithfield and Blackstone, the independent maximum month data is inclusive of the March 30, 2010 flood. The data shows that even accounting for extreme wet weather events the Towns of Bellingham and North Smithfield are within their current flow allocations during their independent maximum month flow event. However, the Town Blackstone exceeded its monthly allocation with an actual monthly average flow of 0.55 mgd versus its allocation of 0.48 mgd for the period inclusive of the March 30, 2010 flood.

For the peak daily events, each of the communities appear to have relatively high wet weather flows. Table 2-3 also shows the peaking factor for each flow condition compared to the average. For comparison, the typical rules of thumb for new sanitary sewer design and construction are that the maximum monthly flow is generally about 1.2 times the average daily flow, the peak daily is approximately 2.0 times the average, and the peak hourly is approximately 3.0 times the average. The maximum month flows for each community exceed the typical peaking factor for new construction, especially for the independent maximum condition. For the peak daily events, Woonsocket had a peaking factor of 3.0 for the WWTF peak daily events. The other communities were at or below the typical 2.0 peaking factor for the WWTF peak daily events, but much higher for the independent peak condition. The peaking factors suggest that all of the communities could benefit from inflow and infiltration removal efforts. However, it should be noted that peaking factors are not unusual for an older collection system, but suggest that there are opportunities for inflow and infiltration removal.

2.2.7 Inflow and Infiltration Removal

The City of Woonsocket has been investigating inflow and infiltration (I/I) in the wastewater collection system based on the high levels of I/I observed in the 2000 RIDEM-approved Facility Plan Amendment (and likely earlier assessments). It should be noted that the Woonsocket collection system does not have any combined sewer overflow structures, and the WWTF no longer has a raw influent bypass structure. Reportedly, sewer overflows occurred during the flooding in March 30, 2010, but are not known to have occurred otherwise in the period from May 2007 through September 2010. The City has contracted with Veolia starting in 2009 for an Underground Asset Management (UGAM) program for the Woonsocket collection system. The inflow and infiltration removal of the UGAM program is building upon flow monitoring work completed by CDM in 2007. The UGAM program has identified various low cost priorities for inflow and infiltration removal that will be carried out over the next few years. As part of this facility planning effort, a supplemental inflow and infiltration program paper on the potential for these inflow and infiltration projects to reduce historic peak flows to the WWTF was developed and is attached in Appendix C.

The UGAM program has been focused to date on performing television inspection of the collection system, and repairing locations with observed significant infiltration. The estimated reduction in infiltration is 0.09 mgd through this program at the end of 2010. This reduction is anticipated to be realized in the maximum monthly flow with a lesser reduction in the annual average flow. In addition, 200 manhole inserts have been installed in areas prone to inflow, and 85 additional inserts will be installed in the near future. The anticipated reduction in peak daily flow is approximately 1.4 to 1.8 mgd during a 1-year storm event.

The City has also moved forward with a number of projects independent from the UGAM program. The new Woonsocket Middle School Campus was constructed at the site of a former mill complex on Hamlet Street. It appears that there were roof drains (or other drains) in some of these mill buildings that allowed rainfall and snow melt to drain into the wastewater collection system. With the demolition of the mill buildings, there has been a notable reduction in peak flows observed by the plant staff. Insufficient data was collected to allow precise quantification, but the flow reduction is estimated to be on the order of 2 to 4 mgd of inflow during the 1-year 30-minute storm. The City is also moving forward with the lining of a sewer that runs through a marsh, which is anticipated to result in a major reduction in infiltration.

Thus, the current peak flows in Table 2-1 from the period from May 2007 thru September 2010 should be reduced somewhat by these inflow and infiltration projects. The ramifications of these projects on design requirements will be addressed in Section 3.

2.3 LOADINGS

Table 2-4 shows the BOD, TSS, and nutrient loadings for each of the key design conditions in the raw influent (including septage), primary influent, and primary effluent for the period from May 2007 through September 2010 and under the Current 2010[a] column includes the comparison of the full data set to the revised data set with the cleaned BOD/TSS ratio of 2.0 or less for the raw sewage loads.

2.3.1 Biochemical Oxygen Demand

The daily biochemical oxygen demand (BOD) loading data are shown in Figure 2-5 for the raw influent, primary influent, and primary effluent. For this facility planning effort, the primary effluent loadings are the key design criteria, especially for the BOD. However, the raw influent BOD loading data are notable due to a pronounced increase starting in February 2010. The increase in loading is attributable to a significant increase in the BOD concentration in the raw influent as shown in Figure 2-6. Over the period from February 2010 to September 2010, the average raw influent BOD loading was 22,700 lbs/d, and the monthly average loading exceeded the previous maximum monthly loading (prior to February 2010) for 7 out of 8 months. A number of potential causes for the increase were investigated including the analytical methods, sampling locations, and the cleaning of the 60-inch main interceptor to the WWTF. It has recently been determined that the majority of the increase is due to an industrial discharger

(Technics). This industry is currently working with the City's pretreatment program as to how the BOD discharge will be managed. The alternatives include pretreatment at the industry and further control of the BOD source. The basis of design for the Woonsocket WWTF Biological Nutrient Removal BNR upgrade includes industrial loads that are reflective of some discharge from Technics and other city-wide industries; and is based on the plant data from January 2010 to July 2012 and the July 2011 through 2012 data from Technics, therefore the industrial loadings are considered conservative in the Facilities Plan. This industrial discharger has permanently reduced its BOD discharge to the levels typically seen in the last six months of 2011. Based on this, the BOD data was revised by removing all daily BOD data that exceeded a 2.0 BOD/TSS ratio on that particular day. It was observed that this cutoff produced a data set over the whole data period that was consistent with the expected operation of that discharger. This is reflected in the Revised BOD_5 data shown in Table 2.4 for the Raw and Primary Influent data. The following provides the methodology for removal of BOD/TSS data greater than a 2.0 ratio:

- A column in the Plant Data (Appendix G) was created for the BOD Raw (lbs/d).
- A column was created for the 30 day Raw BOD average called "Uncleaned" that calculated the 30 day running average of all data.
- A column was created for 30 day BOD "Cleaned" whereby if the BOD/TSS ratio was greater than 2.0, the value was not included in the average.
- Separate columns were created for Raw BOD, Recycle BOD and Raw + Recycle BOD.
- The method provided a statistical analysis that included only averages when the ratio was less than or equal to 2.0 BOD/TSS.

Figure 2-7 shows the frequency distribution of the BOD loadings for the raw influent, primary influent, and primary effluent, and Figure 2-8 shows the 30-day moving average. These frequency distributions include all the data. The maximum 30-day BOD for all three flows occurred during the same time frame in early March 2010 before the start of the flooding. The peak in the raw influent and the primary effluent are notable higher than other peaks in the 30-day moving average over the 3-year period. This suggests that the increase in the raw influent loading may be the driver for the peak in the primary effluent loading.

TABLE 2-4
SUMMARY OF DESIGN AND CURRENT LOADINGS

Item	Existing Design 2000	Current 2010[a]
Raw Influent Loadings (lb/d)		
BOD_5(Full Data/Revised Data[c])		
Annual Average	21,350	14,200 / 14,700
Maximum Monthly		27,100 / 23,200
Peak Daily 98th Percentile		37,000 / 29,900
100th Percentile		51,300 / 49,500
TSS		
Annual Average	16,013	11,600
Maximum Monthly		18,800
Peak Daily 98th Percentile		25,900
100th Percentile		39,200
Ammonia		
Annual Average	2,669	1,146
Peak Daily 98th Percentile		1,997
100th Percentile		1,997
Total Kjeldahl Nitrogen		
Annual Average	5,338	1,800
Peak Daily 98th Percentile		3,267
100th Percentile		3,629
Phosphorus		
Annual Average	2,268	237
Peak Daily 98th Percentile		354
100th Percentile		368
Primary Influent Loadings (lb/d)		
BOD_5 (Full Data/Revised Data[c])		
Annual Average	25,994	24,000 / 24,200
Maximum Monthly		42,500 / 32,800
Peak Daily 98th Percentile		50,000 / 50,100
100th Percentile		65,500 / 72,600
TSS (Full Data/Revised Data[c])		
Annual Average	22,895	34,100 / 25,000
Maximum Monthly		74,300 / 38,700
Peak Daily 98th Percentile		91,900 / 55,700
100th Percentile		170,800 / 73,300

TABLE 2-4
SUMMARY OF DESIGN AND CURRENT LOADINGS

Item	Existing Design 2000	Current 2010[a]
Ammonia		
Annual Average	4,499	1,057
Peak Daily 98th Percentile		1,605
100th Percentile		1,615
Total Kjeldahl Nitrogen		
Annual Average	5,338	2,772
Peak Daily 98th Percentile		4,438
100th Percentile		4,948
Phosphorus		
Annual Average	2,268	1,206
Peak Daily 98th Percentile		2,687
100th Percentile		2,989
Primary Effluent Loadings (lb/d):		
BOD_5		
Annual Average	18,095	13,900
Maximum Monthly		28,900[b]
Peak Daily 98th Percentile		30,600
100th Percentile		48,200[b]
TSS		
Annual Average	6,004	7,400
Maximum Monthly		12,700[b]
Peak Daily 98th Percentile		14,600
100th Percentile		28,500[b]
Ammonia		
Annual Average	5,410	1,231
Max. Month – 95th Percentile		2,091
Peak Daily 98th Percentile		2,445
100th Percentile		2,874
Total Kjeldahl Nitrogen		
Annual Average		2,415
Max. Month – 95th Percentile		5,188
Peak Daily 98th Percentile		6,585
100th Percentile		6,925

TABLE 2-4
SUMMARY OF DESIGN AND CURRENT LOADINGS

Item	Existing Design 2000	Current 2010[a]
Phosphorus		
Annual Average		677
Max. Month – 95th Percentile		1,363
Peak Daily 98th Percentile		1,692
100th Percentile		1,958

Notes:
[a] Based on data from May 2007 through September 2010.
[b] Highest value excluding period from March 16, 2010 to April 23, 2010 when primary clarifiers were disabled.
[c] Revised BOD data removes raw BOD daily data that exceeded a 2.0 BOD/TSS ratio for that day. Ash upset data was removed from Recycles analysis.

FIGURE 2-5
BOD LOADINGS FOR RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFLUENT (MAY 2007 – SEPTEMBER 2010)

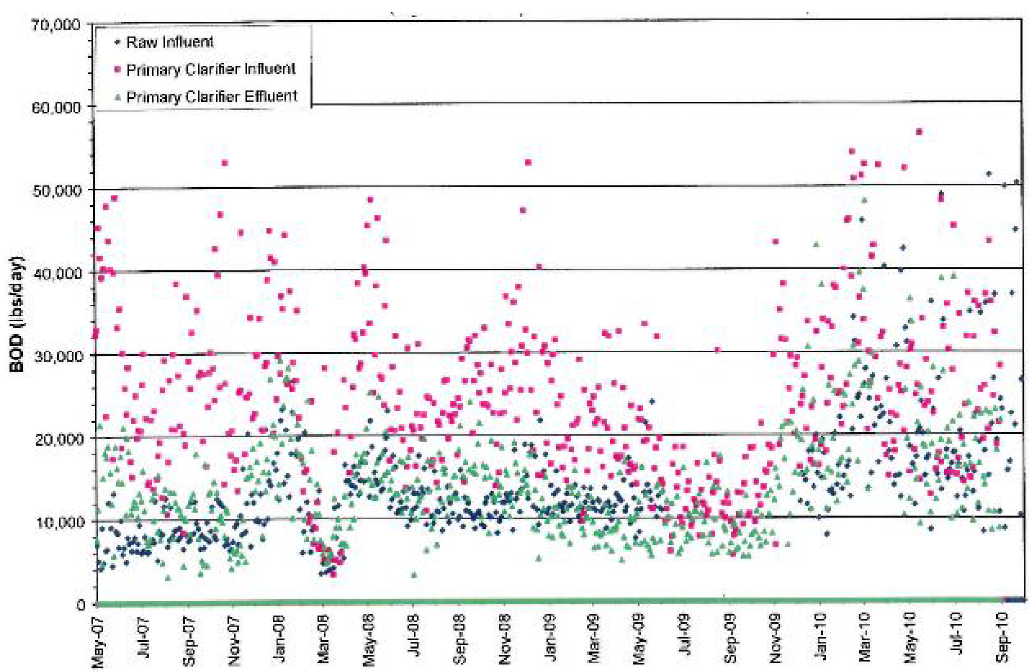


FIGURE 2-6
BOD LOADINGS FOR RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFLUENT (MAY 2007 – SEPTEMBER 2010)

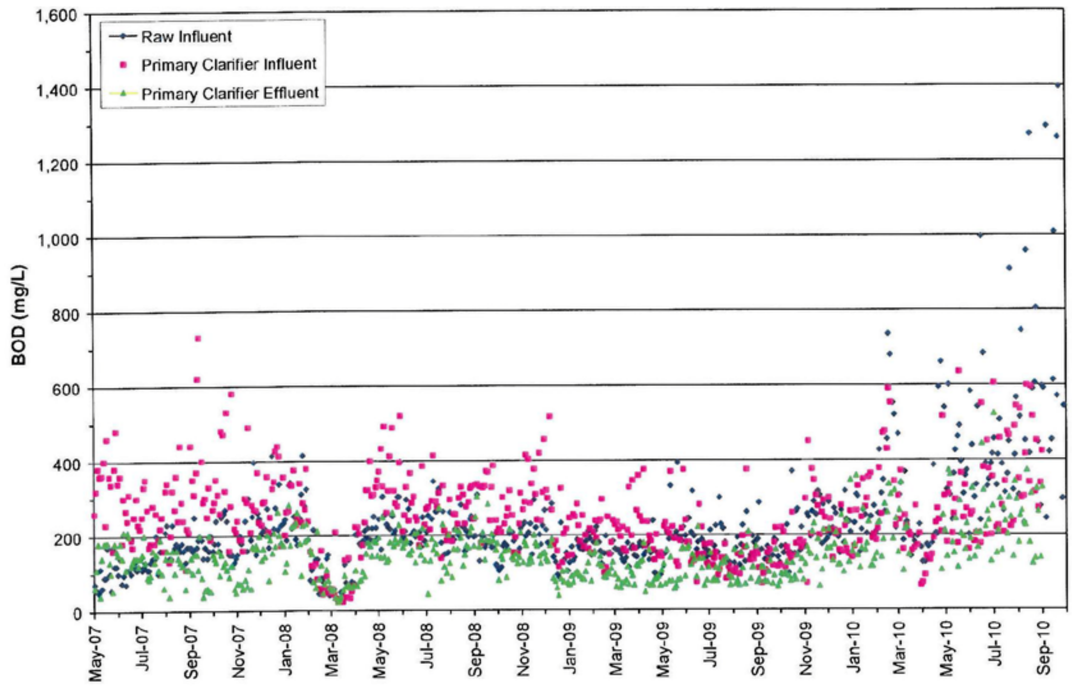


FIGURE 2-7
FREQUENCY DISTRIBUTION FOR BOD LOADING IN RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFFLUENT (MAY 2007 – SEPTMBER 2010)

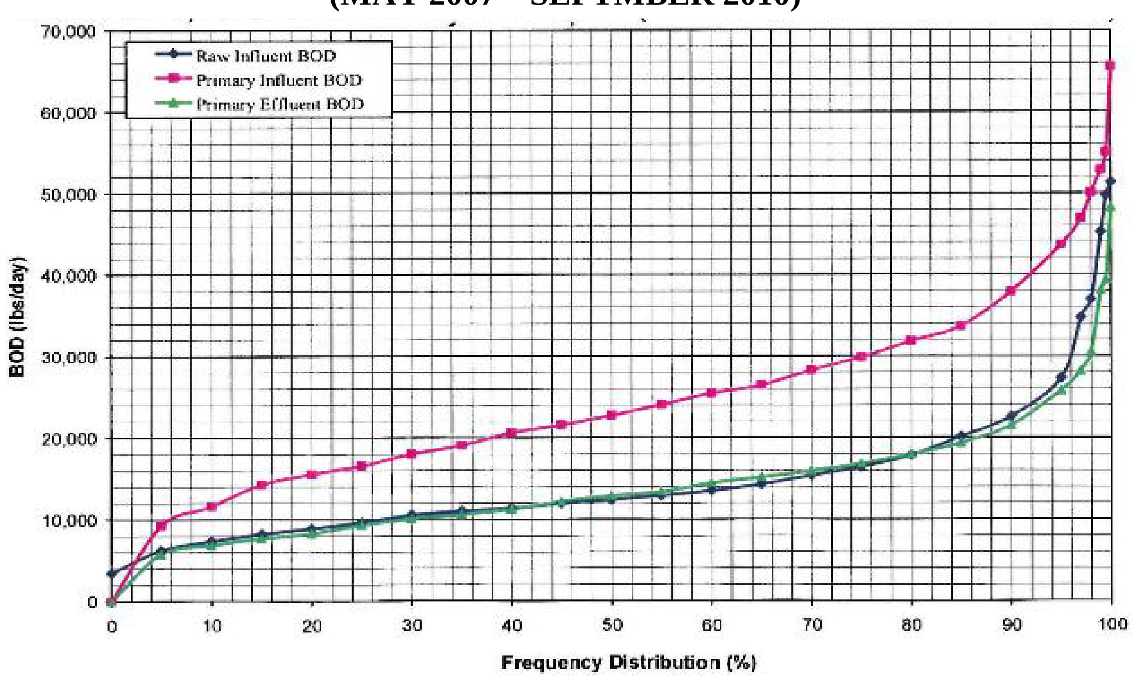
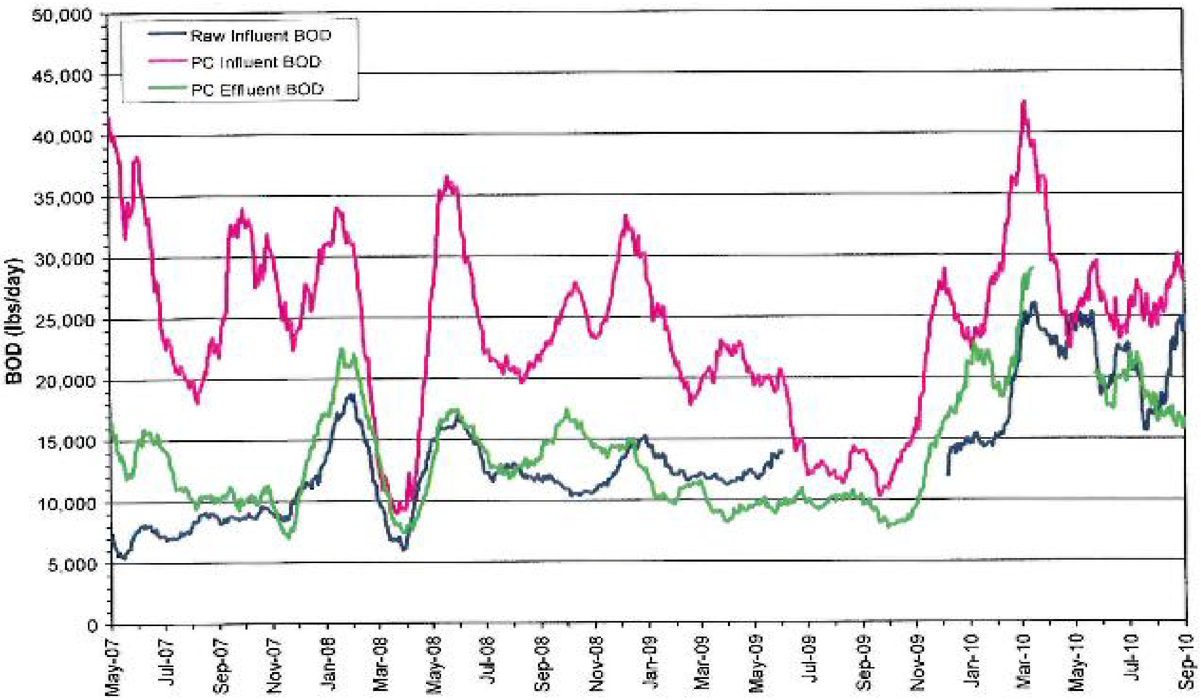


FIGURE 2-8
30-DAY MOVING AVERAGE BOD LOADING IN RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFFLUENT (MAY 2007 – SEPTEMBER 2010)



The 2007 – 2010 loadings were compared to the data in the 2000 Facility Plan Amendment to determine longer term trends. The 2007 - 2010 average raw influent BOD loadings of 14,200 lbs/d are at essentially the same level as was found during the 1998-99 data period with an average raw BOD of 14,100 lbs/d. Overall, it appears that BOD loadings decreased in the early 2000's with the loss of certain industries in Woonsocket, but have increased over the last few years to previous levels.

The primary influent BOD loading of 20,345 lbs/d from 1998-99 is lower than the current average primary influent BOD loading of 24,000 lbs/d. This appears to be attributable to an increase in the internal recycle loadings at the WWTF. As will be discussed further in Section 2.3.3 below, the centrate flow (S-2) has a noted high soluble BOD loading due to the anaerobic storage of the merchant sludge that is delivered to the facility. One of the changes implemented as part of the 2000 Facility Plan Amendment is that the centrate from the Synagro dewatering operations is discharged through the gravity thickener (S-3), which is used for co-thickening of the Woonsocket primary and secondary sludge. Co-thickening can also result in anaerobic conditions that promote the formation of volatile organic acids, and the addition of the centrate has the potential to further promote anaerobic conditions. Another consideration is that Synagro constructed a new fluidized bed incinerator with increased capacity since 1998-99. However, the quantity of liquid sludge processed during 1998-99 was equal to or slightly greater than current quantities (Synagro added cake sludge receiving as part of the upgrade, which provides the added capacity). This raises the question of whether the addition of the centrate to the gravity thickener has been effective in reducing loadings as intended or has promoted higher loadings by causing the gravity thickener to go septic more consistently. The performance of the gravity thickener will be addressed further in Section 2.3.3.

The 2000 Facility Plan Amendment does not document primary effluent loadings in 1998-99, but the available information indicates that the current average BOD loading of 13,900 lbs/d is generally lower than the loadings from the July 1995 through June 1999 period.

2.3.2 Total Suspended Solids

The daily total suspended solids (TSS) loading data are shown in Figure 2-9 for the raw influent, primary influent, and primary effluent, the frequency distribution data is shown in Figure 2-10, and the 30-day moving average in Figure 2-11. As previously noted, the daily TSS level in the raw influent does not display a significant increase starting in February 2010 like the BOD does. For example, the historical maximum in the 30-day moving average for the raw influent occurred in June 2008 as shown in Figure 2-11 rather than in March 2010 as occurred for the BOD.

The primary influent TSS data includes the impact of recycle loadings as well as lime that is added for alkalinity addition to promote phosphorus and nitrogen removal in the secondary system and to enhance settling in the primary clarifiers. Lime has historically been added at a rate of approximately 10,000 lbs/d for the period from April 1 through October 31, which corresponds to the time frame of the seasonal phosphorus limit from the prior RIPDES permit. The lime is added to the influent end of the aerated grit chamber for mixing, and the primary influent is sampled at the effluent end of the grit chambers. The recycle TSS loads were found to be heavily impacted by a few single day events where TSS levels were extremely high. These high levels were a direct result of Synagro ash thickener upsets at their facility. These high TSS events have little impact on the facility since these solids settle out very rapidly within primary treatment and are not transmitted to secondary treatment. The only facility impact is on the capacity of the primary sludge pumps. Based on this, these anomalous recycle TSS data were removed from the data set for purposes of this analysis. This is reflected in the primary influent TSS data shown in Table 2.4.

FIGURE 2-9
TSS LOADING FOR RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFFLUENT MAY (MAY200 2007 –SEPTEMBER 2010)

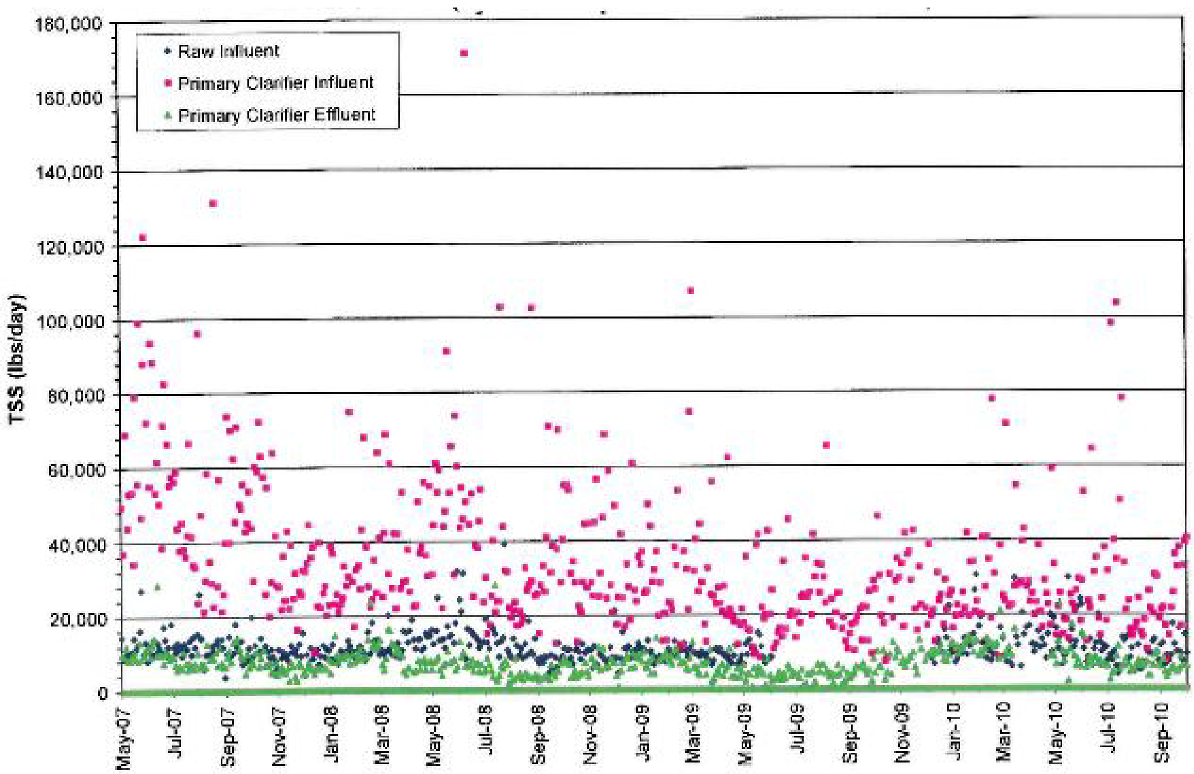


FIGURE 2-10
FREQUENCY DISTRIBUTION FOR TSS LOADING IN RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFFLUENT (MAY 2007 – SEPTEMBER 2010)

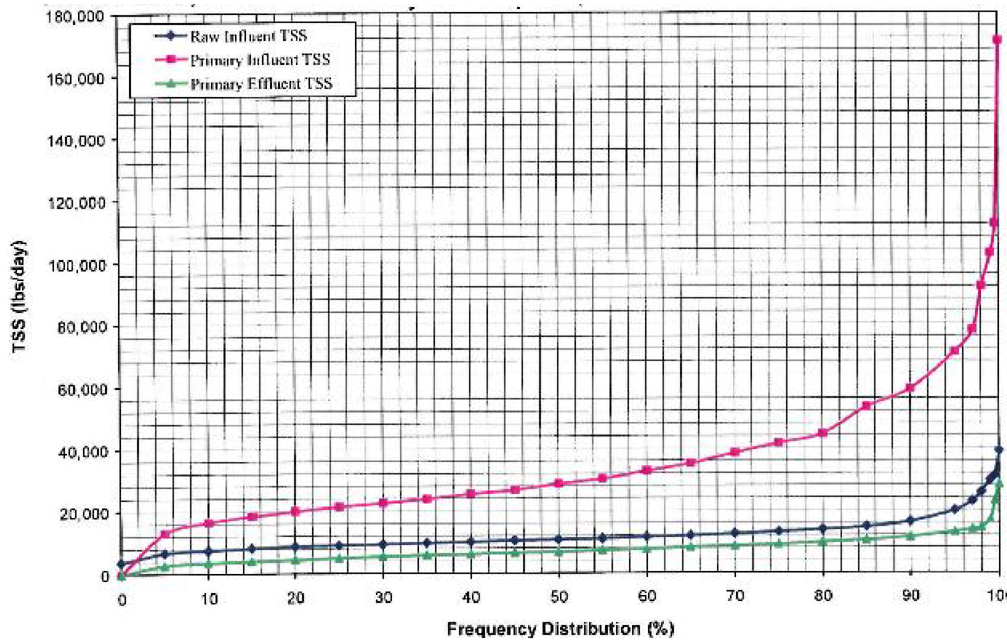
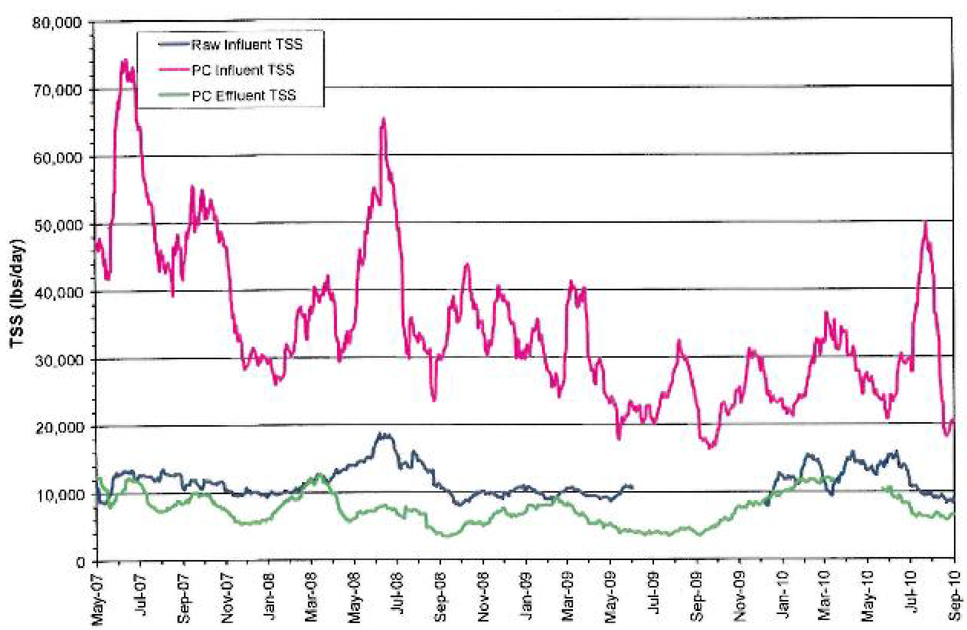


FIGURE 2-11
30-DAY MOVING AVERAGE TSS LOADING IN RAW INFLUENT, PRIMARY INFLUENT, AND PRIMARY EFFLUENT (MAY 2007 – SEPTEMBER 2010)



A review of historic operating data indicates that the primary clarifiers are very effective in removing the TSS loadings (approximately 75% removal on average, before recycle TSS data cleaning) and limiting the loadings in the primary effluent. This may be due in part to the lime addition for 7 months of the year, but lime dosage is less than would conventionally be considered for chemically-enhanced settling and the data does not show a conclusive trend corresponding to the periods with and without the lime addition.

The long term trends in TSS loadings indicate very little change in the average raw influent and primary influent levels based on comparison to levels in the 2000 Facility Plan Amendment for the 1998-99 period. However, the current TSS loading in the primary effluent appears to be lower based on the available data for July 1995 through June 1999 in the 2000 Facility Plan Amendment.

2.3.4 Recycle Loadings

As previously noted, recycle loadings represent a significant portion of the plant's primary influent loadings for BOD and TSS as well as for nutrients which are discussed in Section 2.4.

The estimated recycle flows are based on the difference between the metered primary effluent flow and the final effluent flow. As noted in Section 2.2.1, this method of estimating the recycle flow has inaccuracies both due to inability to measure new flows like centrate from liquid merchant sludge and due to apparent discrepancies between the primary effluent Parshall flume and the final effluent meter. Based on the analysis of recycle flows summarized in Table 2-2, it appears that the this estimate of recycle flows may be 30% higher on average than actual recycle flows (average measured recycle flow of 3.9 mgd versus estimated average of 3.1 mgd). The plant estimates the recycle loadings by measuring the BOD and TSS concentration in the recycle flows directly, and calculating the loadings using the estimated recycle flow rate. This can be compared to the loadings determined by subtracting the measured raw influent loading from the primary influent loading as shown in Table 2-5. In general, there is relatively good correlation of the recycle loadings determined in both ways with some notable exceptions.

For BOD loadings, the average condition closely correlates between the different flow measuring methods., and the maximum monthly condition also correlates relatively well. For the peak daily data, the approach of determining the loading by difference does not appear to work well with low results (e.g. levels less than the maximum monthly level).

On average, the BOD loading in the recycle flows is approximately 42% of the primary influent loading. As noted in Section 2.3.1, the majority of the BOD in the recycle flows is attributed to the gravity thickener overflow, and the centrate appears to be a key source of the high BOD. This is illustrated in Table 2-6 which shows the estimated breakdown of recycle loadings. The BOD loading attributable to the gravity thickener overflow is determined by the difference between the total loading and the sum of the reported BOD loading in the incinerator recycle flows, the secondary scum recycle flow, and the effluent filter backwash. Table 2-6 also shows the BOD loading in the centrate as reported by Synagro that is discharged to the gravity thickener. The BOD loading in the centrate is 87% of the total recycle loading and 94% of the loading attributed to the gravity thickener.

TABLE 2-5
RECYCLE LOADINGS DETERMINED BY DIRECT MEASUREMENT VERSUS DIFFERENCE

Item	Existing Design 2000	Current Direct Measured	Current By Difference[a]
Recycle Flows (MGD)			
Average Annual	2.6		3.9
Maximum Monthly			5.6
Peak Daily			8.5
Peak Hourly			-
Recycle Loads (lb/d)			
BOD5			
Annual Average	4,644	10,000	9,800
Maximum Monthly		19,400	15,400
Peak Daily 98th Percentile		26,700	13,000
100th Percentile		45,600	14,200
TSS			
Annual Average	6,882	15,900	22,500
Maximum Monthly		54,300	55,500
Peak Daily 98th Percentile		56,100	66,000
100th Percentile		190,600	131,600
Ammonia			
Annual Average	1,830	319	<0
Peak Daily 98th Percentile		575	<0
100th Percentile		580	<0

TABLE 2-5
RECYCLE LOADINGS DETERMINED BY DIRECT MEASUREMENT VERSUS DIFFERENCE

Item	Existing Design 2000	Current Direct Measured	Current By Difference[a]
Total Kjeldahl Nitrogen			
Annual Average		1,182	972
Peak Daily 98th Percentile		2,560	1,171
100th Percentile		2,849	1,319
Phosphorus			
Annual Average		861	969
Peak Daily 98th Percentile		2,145	2,333
100th Percentile		2,269	2,621

Notes:
[a] Determined by difference between Primary Influent and Raw Influent.

TABLE 2-6
BREAKDOWN OF RECYCLE BOD AND TSS LOADING[b]

Item	Annual Average	Maximum Month
BOD, lbs/d		
Total – Direct Measurement	10,000	19,400
Incineration Facilities, S-1[a]	94	323
Secondary Scum	>100	>100
Effluent Backwash	~500	~1,000
Attributable to Gravity Thickener Overflow, S-3	*9,306*	*17,977*
Centrate, S-2[a]	8,724	14,405
TSS, lbs/d Total – Direct Measurement	15,900	54,300
Incineration Facilities, S-1[a]	1,133	8,996
Secondary Scum	>100	>100
Effluent Backwash	~1,000	~2,000
Attributable to Gravity Thickener Overflow, S-3	*13,667*	*43,204*
Centrate, S-2[a]	2,765	6,630

Notes:
[a] Data as reported by Synagro
[b] TSS data includes ash upset data

For TSS loadings, the average condition shows a notably greater loading when determined by difference than by direct measurement. It is possible that the lime addition or the ash upsets contribute to the measured primary influent loadings and accounts for some of this difference. However, the maximum monthly condition has remarkably good correlation, and the peak daily TSS loadings are reasonably consistent between the two methods.

The TSS loadings from the recycle are a high proportion of the primary influent loading averaging 66% with even greater proportions at other loading conditions before the elimination of the anomalous TSS data. After the cleanup, the fraction of TSS load from recycles on average was 53%. Table 2-6 shows the estimated breakdown of recycle loadings by source. The recycle TSS loadings from the incineration facilities are relatively low under average conditions, but increase substantially at the maximum month condition. This is due to ash carry over from the ash thickener. Synagro was not able to document the peak daily TSS loadings from the incineration facilities, but they are reported to be very high during periods of operational problems with the ash thickener.

In contrast to the BOD loading, the centrate represents only a small proportion of the recycle TSS loading attributable to the gravity thickener overflow. The high recycle TSS loadings are common for gravity thickeners that are used for co-thickening, and indicate anaerobic conditions have caused deflocculation of the secondary sludge. The primary clarifiers have been remarkably effective in removing the solids, and returning them to the gravity thickener, but there are undoubtedly impacts to the primary effluent loadings from the high recycle loadings. There appear to be two contributing factors to the notably high TSS recycle loadings from the gravity thickener overflow as follows:

The solids loading rate to the gravity thickener exceed recommended loading limits for co-thickening of primary and secondary solids. For example, TR-16 recommends a maximum solids loading rate of 6 to 10 lbs/d/sq. ft. for co-thickening of primary and waste activated sludge. The estimated primary sludge production based on the difference in primary influent and effluent loadings is 26,700 lbs/d on average and 61,600 lbs/d for the maximum month condition. The waste secondary sludge quantity is estimated to be approximately 13,200 lbs/d on average. The centrate has an average of 2,765 lbs/d and 6,630 lbs/d at maximum month.

The combined sludge quantity results in a solids loading rate of 15 lbs/d/sq.ft under average loading conditions, and 29 lbs/d/sq.ft. under maximum monthly conditions.

The anaerobic condition of the centrate is believed to significantly promote the anaerobic conditions in the gravity thickener.

The estimated sludge quantities and TSS recycle loadings from the gravity thickener suggest an average solids capture of about 65% that deteriorates to about 44% at the maximum month condition.

2.4 NUTRIENTS

The nitrogen and phosphorus loadings to the secondary system were reviewed to assess the requirements for enhancing nitrification and denitrification performance, as well as phosphorus removal. The facility has extensive monitoring data for nitrogen and phosphorus in the final effluent, but much more limited data on the raw influent, primary influent, and primary effluent. Because of the desire to limit the data to the period after the start up of the new fluidized bed incinerator, the available data for nitrogen and phosphorus in these influent loadings was limited to the following:

- A special monitoring effort by Veolia and the City that characterized ammonia, total Kjeldahl nitrogen (TKN), nitrate, nitrite, ortho-phosphorus, and total phosphorus in the raw influent, recycle flows, primary influent, and primary effluent on 15 days in November and December of 2008 and January of 2009, and then again on 8 days in July and August of 2009.
- A supplemental monitoring program that was carried out as part of this facility plan effort on 9 days in October and November of 2010. This program included supplemental testing of the raw influent, recycle and primary effluent for nitrogen and phosphorus as well as a number of more specialized tests associated with calibration of the BioWin model.
- A raw wastewater characterization effort during the Fall of 2012 to determine the wastewater characteristics with a more typical industrial influent load.

Because of the limited database, it was not possible to determine the maximum monthly loadings based on an analysis of the 30-day moving average for these parameters. The primary effluent was the primary concern, because this represents the secondary influent loading for analysis of enhancements for nitrogen and phosphorus removal. The frequency distribution for BOD and TSS was analyzed to assess an appropriately conservative value for design. The maximum month condition for the BOD loading in the primary effluent corresponded to about the 97.5th percentile, while the TSS corresponded to about the 94th percentile. In order to avoid excessive conservatism, the 95th percentile was selected as representative of the maximum month condition for nitrogen and phosphorus loading, which is consistent with EPA statistical methods.

2.4.1 Nitrogen

Total nitrogen is considered to be the sum of the ammonia, organic nitrogen, nitrate, and nitrite. The total Kjeldahl nitrogen (TKN) is a measure of the combination of ammonia and organic nitrogen. Table 2-4 summarizes the TKN and ammonia levels in the raw influent, primary influent and primary effluent, and Table 2-5 summarizes the data for the recycle flows.

Figure 2-12 shows the frequency distribution of the TKN and ammonia loadings in the raw influent and the recycle flow, and Figure 2-13 shows the frequency distribution for the primary effluent. The measured TKN levels are reasonably consistent with raw influent of 1,800 lbs/d, recycle of 1,182 lbs/d, and primary influent of 2,772 lbs/d. The data is not as consistent for ammonia with a raw influent loading of 1,146 lbs/d, recycle of 319 lbs/d and primary influent of 1,057 lbs/d. The difference may be attributable to the fact that primary influent monitoring data is limited to only 23 days out of the total of 32 days in the database. **The available data indicates that recycle loadings account for 40% of the TKN in the primary influent and 22% of the ammonia.** The higher proportion for the TKN is attributable to the high level of TSS, and thus organic nitrogen, in the recycle flows.

The average primary effluent TKN level of 2,415 lbs/d is considerably higher than the median value of 1,900 lbs/d as shown in Figure 2-13. This is due to the impact of several high peak values with an overall peak daily TKN loading of 6,925 lbs/d. The ammonia loading displays a considerably lower peaking factor. The ammonia averaged 1,231 lbs/d in the primary effluent compared to the median value of 1,080 lbs/d and peak daily value of 2,874 lbs/d For the maximum monthly value, the 95th percentile was selected as noted above resulting in a TKN loading of 5,188 lbs/d and an ammonia loading of 2,091 lbs/d. This compares to the design value for the 2000 upgrade of 5,410 lbs/d of ammonia in the primary effluent, and TKN was not defined.

FIGURE 2-12
FREQUENCY DISTRIBUTION FOR TKN AND AMMONIA IN RAW INFLUENT AND RECYCLE FLOW

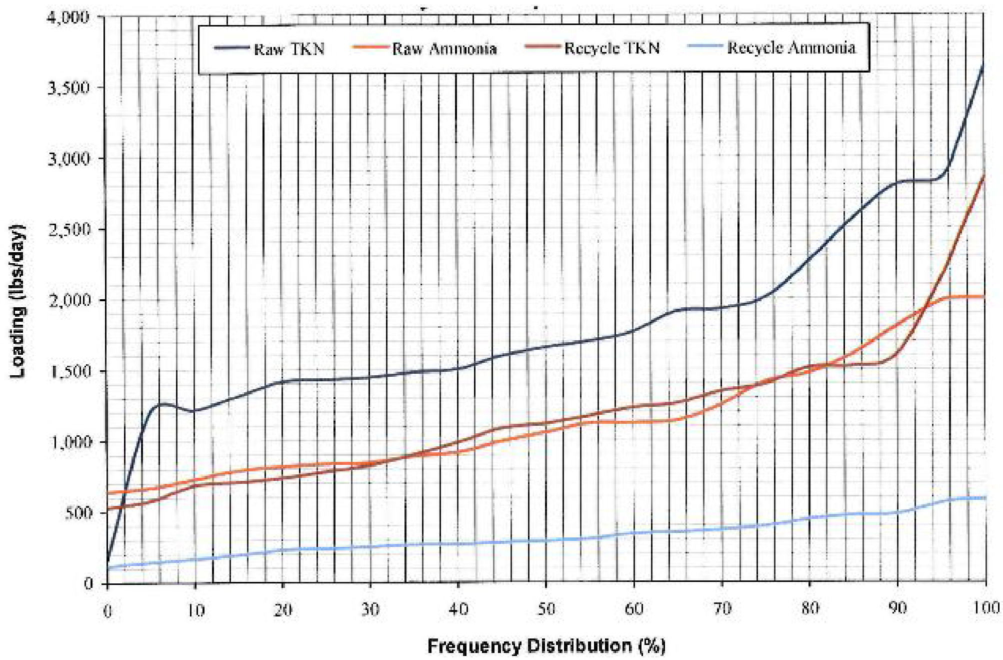
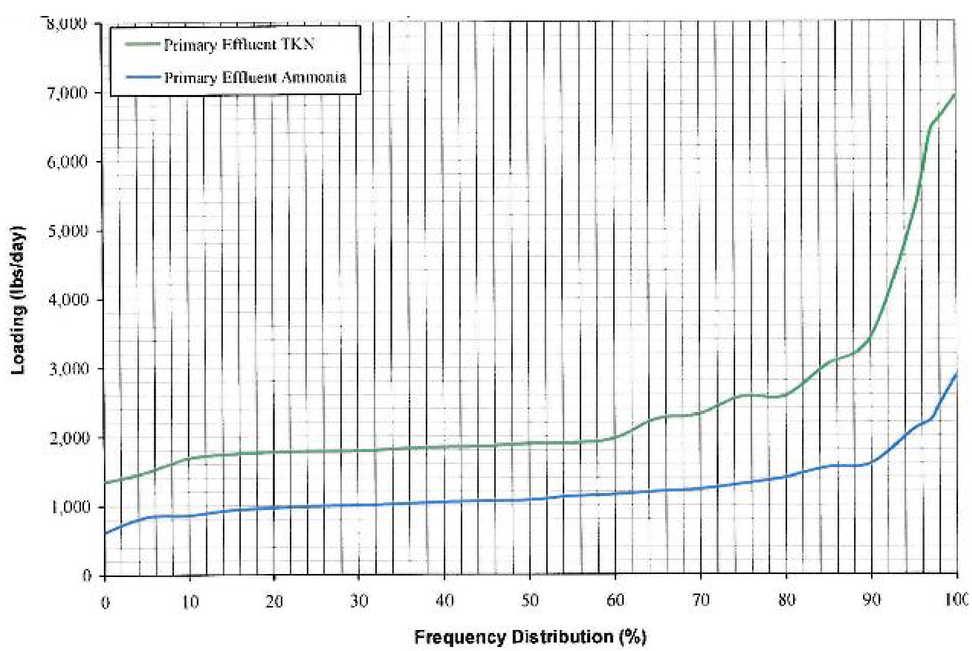


FIGURE 2-13
FREQUENCY DISTRIBUTION FOR TKN AND AMMONIA IN PRIMARY EFFLUENT



The loadings of nitrate and nitrite in the raw influent are low, but not negligible. The combination of nitrate and nitrite averaged 1.2 mg/l or about 99 lbs/d in the raw influent, and the peak day concentration was 2.75 mg/l. The recycle flows have nitrate/nitrite due to the extensive use of plant water at the Solids Handling Facilities as well as the backwash flows from the effluent sand filters. The nitrate/nitrite in recycle flows averaged 0.75 mg/l with a daily maximum of 1.61 mg/l. The primary influent had an average of 0.47 mg/l with a daily maximum of 2.66 mg/l. The primary effluent had an average level of 0.25 mg/l with a daily maximum of 1.78 mg/l. Thus, primary effluent had an average nitrate/nitrite loading of 26 lbs/d, which is about 1% of the average TKN loading of 2,415 lbs/d, and is considered a negligible level for the BioWin© modeling effort.

2.4.2 Phosphorus

Table 2-4 summarizes the total phosphorus levels in the raw influent, the primary influent, and the primary effluent and Table 2-5 summarizes the data for the recycle flows. Figure 2-14 shows the frequency distribution for total and ortho-phosphorus in the raw influent and recycle flows, while Figure 2-15 shows the frequency distribution for the primary effluent. The measured total phosphorus levels are reasonably consistent on average with raw influent of 354 lbs/d, recycle of 2,145 lbs/d, and primary influent of 2,687 lbs/d. The ortho-phosphorus results are also consistent on average with a raw influent loading of 151 lbs/d, recycle of 409 lbs/d and primary influent of 565 lbs/d. The available data indicates that recycle loadings account for 86% of the total phosphorus in the primary influent and 73% of the ortho-phosphorus. The high proportion for both total and ortho phosphorus is attributable to the anaerobic conditions in both the gravity thickener and the sludge storage tank for merchant sludge.

The average primary effluent level for total phosphorus is 677 lbs/d, and compares to a median value of 580 lbs/d as shown in Figure 2-14 and a peak daily loading of 1,958 lbs/d. The ortho-phosphorus loading averaged 389 lbs/d in the primary effluent compared to the median value of 168 lbs/d and peak daily value of 1,546 lbs/d. For the maximum monthly value, the 95th percentile was selected resulting in a total phosphorus loading of 1,363 lbs/d and an ortho-phosphorus loading of 983 lbs/d.

The recycle and primary influent phosphorus data during this period is not expected to be typical for the revised design. This is a result of the recycle phosphorus load being elevated through the cycling up of phosphorus in the system by biological phosphorus removal in secondary treatment. The anaerobic conditions in the gravity thickener will release biologically bound phosphorus, which then gets returned in the recycles. This phosphorus is then taken up again biologically in secondary treatment, along with the raw sewage phosphorus, and the cycle starts again. This cycling up of phosphorus will continue until a steady state is reached in the system that has a significantly higher recycle phosphorus load than would be expected in the proposed new design of the plant. This cycling up in the new plant will not happen because the metal salts added to primary and the second stage sludge system will absorb most of the biologically released phosphorus in the gravity thickener.

The higher influent phosphorus load in the original Facility Plan was artificially inflated through the cycling up of phosphorus in the system by biological phosphorus removal. The current loads were determined through a mass balance on the facility taking into account all the various inputs and the planned use of metal salts. The facility has the ability to remove more phosphorus if the loads are higher than the current design through both biological phosphorus removal in the first stage and additional metal salts in the second stage sludge system.

FIGURE 2-14
FREQUENCY DISTRIBUTION FOR ORTHO AND TOTAL PHOSPHORUS IN RAW INFLUENT AND RECYCLE FLOW

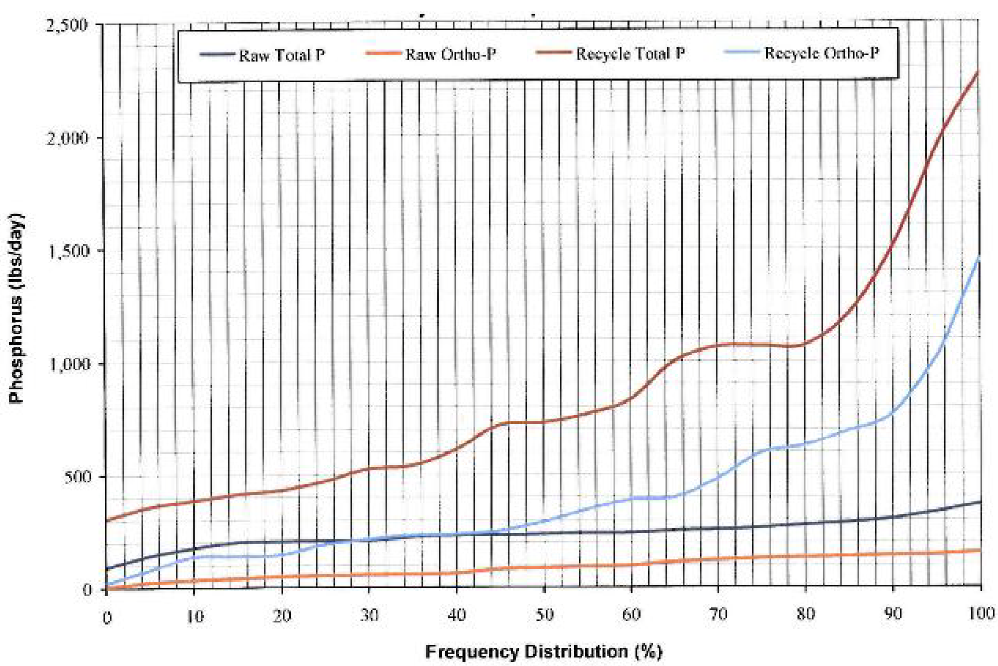
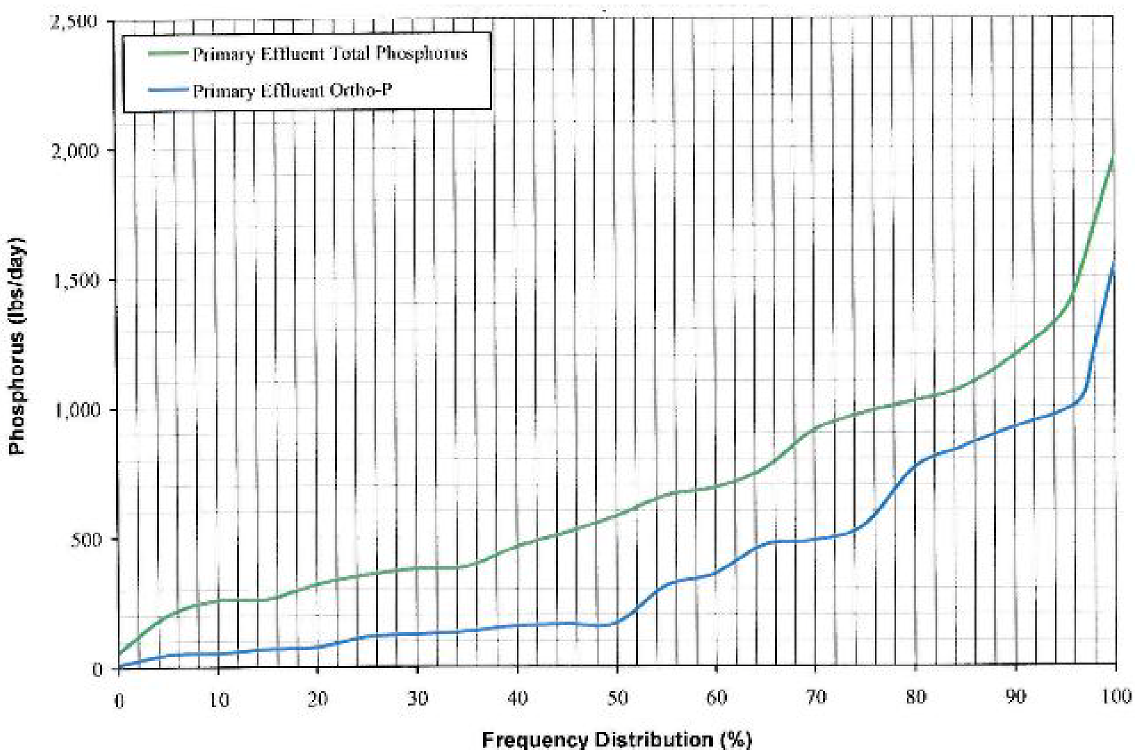


FIGURE 2-15
FREQUENCY DISTRIBUTION FOR ORTHO AND TOTAL PHOSPHORUS IN PRIMARY EFFLUENT



2.5 TEMPERATURES

Wastewater temperature varies seasonally, and has an important impact on the nitrification process and other process considerations. The wastewater temperature at the Woonsocket Regional WWTF increases from the raw influent to the aeration basins due to the effect of the scrubber water recycle flows (S-1) from the incineration facilities. Figure 2-16 shows the daily temperature data for the raw influent, the aeration basins, and the ambient air, and Figure 2-17 shows the monthly average values. The aeration basin temperatures are significantly higher than the raw influent due to the impact of the recycled scrubber flows (S-1). In fact, Synagro installed cooling towers in early 2008, because the peak temperatures in the aeration basins had exceeded levels that can be detrimental to the nitrifying bacteria (greater than 35°C). The cooling towers are now operated during warm weather (primarily July through September), and used to maintain the aeration basin temperature at maximum levels of about 28°C to 30°C during the warmest conditions.

As shown in Figure 2-16, there have been occasional short-term shutdowns of the incinerator where the wastewater temperature in the aerations basins has fallen to the raw influent temperature. The fluidized bed incinerator is relatively new, and has not had an extended shutdown, since start up in May 2007. Synagro indicates that they intend to maintain the incinerator in operation a minimum of 92% of the time, which they have achieved since the system started up. As the incinerator equipment ages, it will be important for Synagro to continue its maintenance program to avoid extensive shutdowns with resulting impacts to the raw influent temperature. Planned downtime for maintenance of equipment should be scheduled at times when the decreased recycle temperatures have the least impact on the nitrification process.

Table 2-7 shows the existing wastewater temperature for the two periods of the year with total nitrogen limits. The new RIPDES permit limit for April is a transitional limit with a total nitrogen requirement of 10 mg/l in the effluent. The minimum monthly average temperature for April occurred when the incinerator was off-line for the first seven days of the month. This is reflected in the minimum temperature information as well.

FIGURE 2-16
DAILY WASTEWATER AND AMBIENT TEMPERATURES
(MAY 2007 – SEPTEMBER 2010)

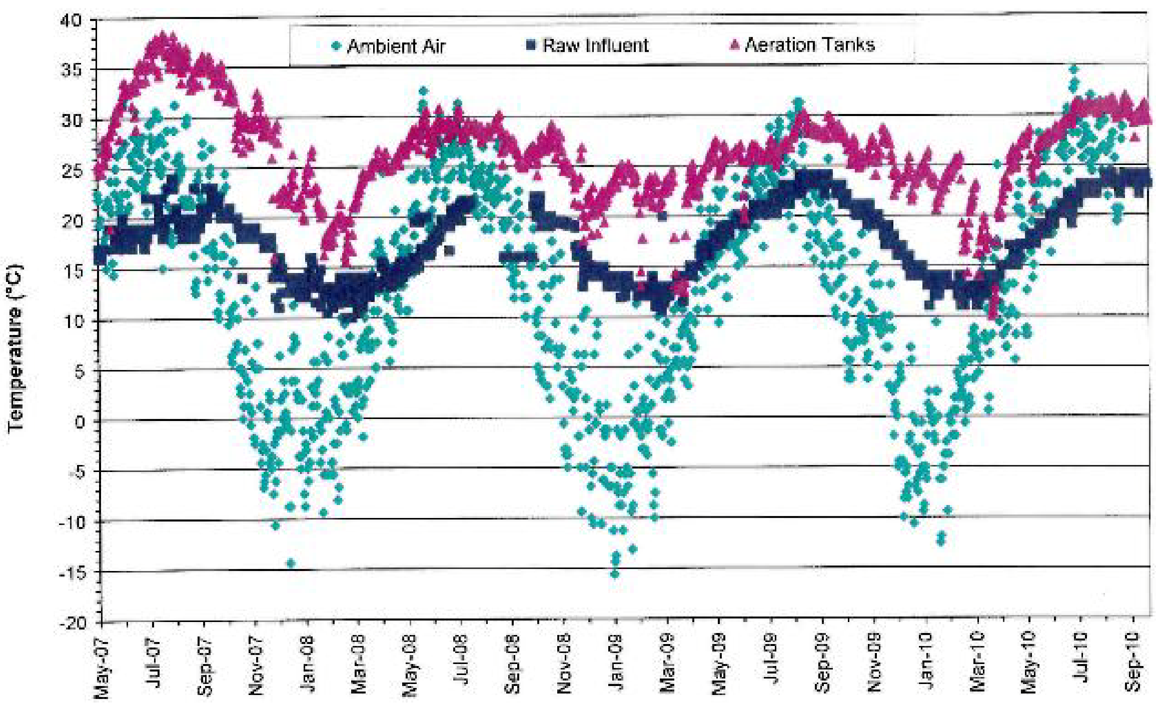


FIGURE 2-17
MONTHLY AVERAGE WASTEWATER AND AMBIENT TEMPERATURES
(MAY 2007 – SEPTEMBER 2010)

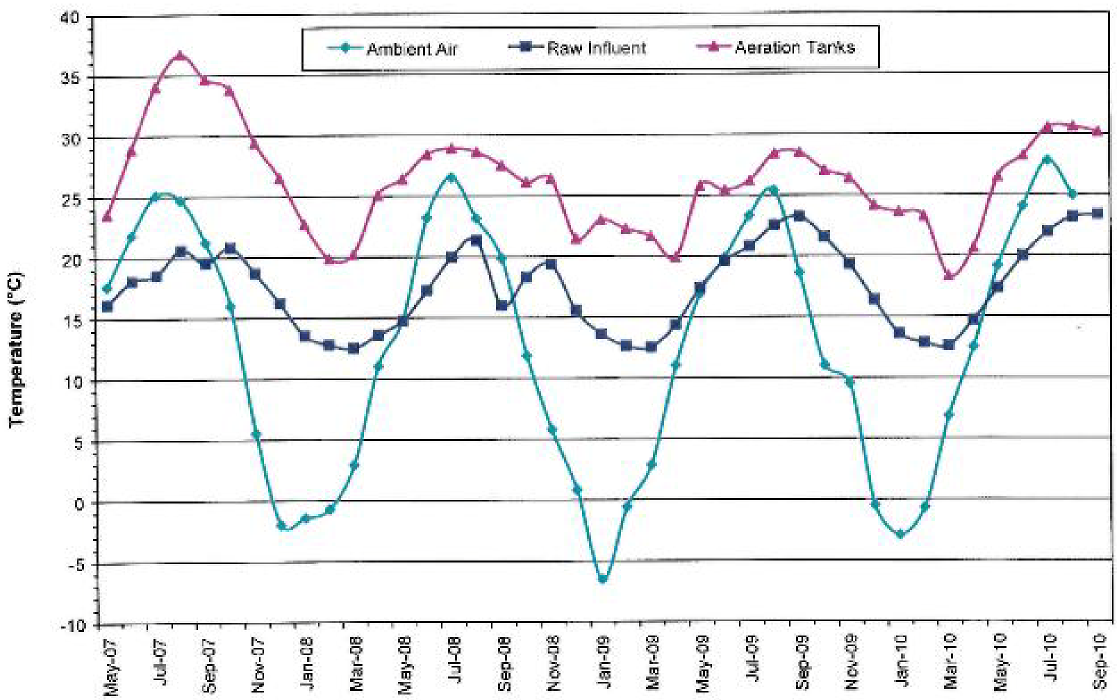


TABLE 2-7
EXISTING WASTEWATER TEMPERATURES

Condition	Raw Influent Degrees Celsius (^oC)	Aeration Basin Degrees Celsius (^oC)
Existing Conditions:		
April:		
Minimum Monthly	13.6	19.9
Minimum 7-Day	12.9	12.4/20.6[a]
Minimum Day	11.5	10.5/19.8[a]
May 1 to October 31:		
Minimum Monthly	16.2	23.7
Minimum 7-Day	13.9	22.4
Minimum Day	13.0	19.7

Notes:
[a] First value is minimum for available data, and second excludes periods that incinerator was off-line.

The new RIPDES permit requirements from May 1 through October 31 are for a total nitrogen limit of 3.0 mg/l. The initial month of operation of the incinerator in May 2007 had the lowest monthly average for this period, and reflected some short term shutdowns of the incinerator. Typical months when the incinerator was on-line for the entire period would be higher, but the proposed values are reflective of actual operating conditions. It should be noted that for the period from May 1 through October 31, the minimum monthly temperature occurred in May for all years in the data set. However, temperature data in June was the minimum for the 7-day moving average and the minimum day.

2.6 ALKALINITY

The alkalinity of the wastewater flow has been measured in the primary effluent for a significant portion of the data period as shown in Figure 2-18. The monthly average data for the periods with data are shown in Figure 2-19. Figures 2-18 and 2-19 show the significant difference in alkalinity during the period from April 1 through October 31 when the facility adds approximately 10,000 lbs/d of lime. Alkalinity is consumed by the nitrification process, and can cause the pH to drop if too much alkalinity is consumed. The facility measures the pH of the effluent continuously as well as the raw influent, primary effluent, and aeration tank effluent to confirm that the alkalinity addition is sufficient.

The monthly average pH of the raw influent, primary effluent, aeration basin effluent, and final effluent are shown in Figure 2-20. The raw influent has averaged a pH of 7.3 and ranges from a pH of 6.4 to 7.9. When lime addition is carried out, the primary effluent pH is typically in the range of 8.5 to 9.0. The pH drops significantly across the aeration basins due to the consumption of alkalinity by the nitrification process. During the period with lime addition, the pH in the aeration basin effluent drops back to about the raw influent pH in the range of 7.0 to 7.5. During the period from November 1 to March 31, it appears that the nitrification process continues on a more limited basis. The pH has dropped to as low as 6.1 during the period without lime addition. Interestingly, the pH rebounds significantly in the final effluent, which averages a pH of 7.7. The potential mechanisms include denitrification in the secondary clarifiers and effluent sand filters (when on-line), and a small impact from addition of hypochlorite for disinfection.

The lime addition to the primary clarifier maintains the pH in the aeration basin in the ideal range for nitrification as well as for biological phosphorus removal. This is usually most important for the coldest months, and then less critical during warmer conditions. During the period of the total nitrogen limits from April 1 through October 31, the highest addition rates are needed in April, May, June, and October. However, it may be possible to reduce alkalinity addition during the warmest conditions from July through September. The proposed improvements to enhance nitrogen removal (i.e. denitrification) are not likely to have a significant impact on the resulting final alkalinity and pH, and will not impact compliance with the RIPDES discharge permit limits for pH.

FIGURE 2-18
DAILY PRIMARY EFFLUENT ALKALINITY
(MAY 2007 – SEPTEMBER 2010)

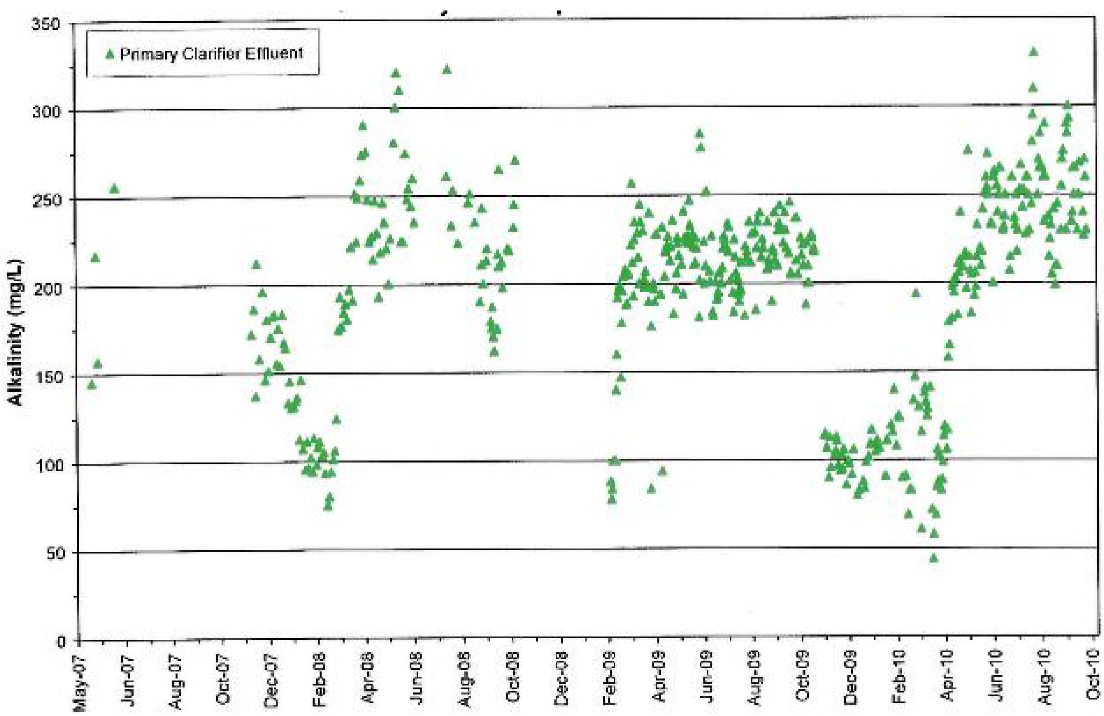


FIGURE 2-19
MONTHLY AVERAGE PRIMARY EFFLUENT ALKALINITY
(MAY 2007 – SEPTEMBER 2010)

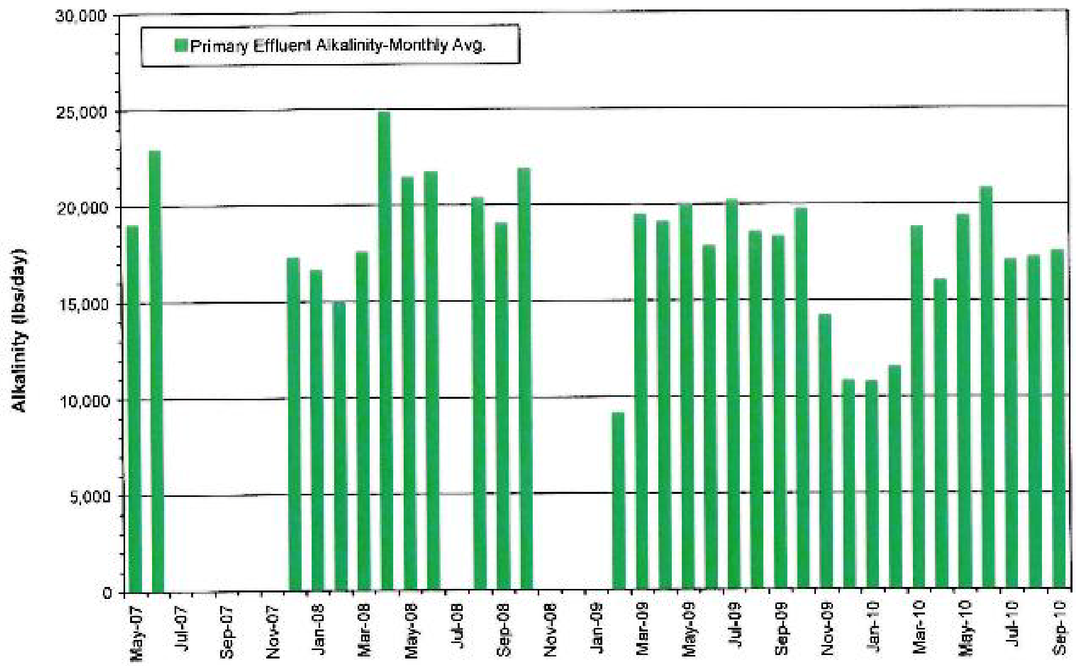
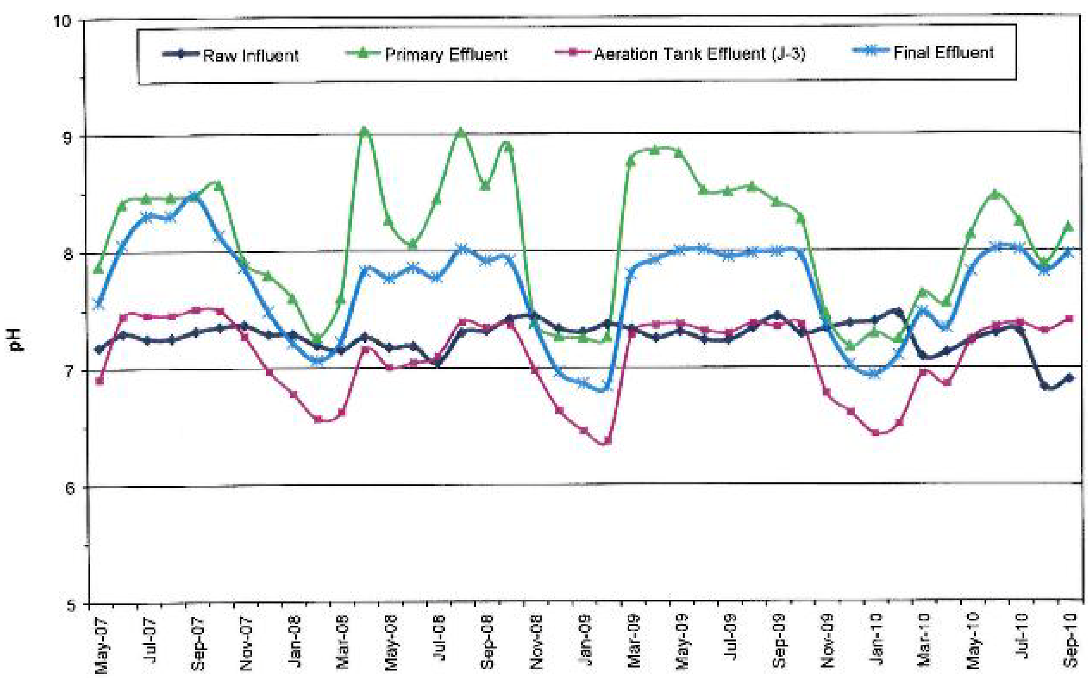


FIGURE 2-20
MONTHLY AVERAGE WASTEWATER PH
(MAY 2007 – SEPTEMBER 2010)



SECTION 3
DESIGN FLOWS AND LOADS

3.1 INTRODUCTION

In Section 2 – Existing Flows and Loads, the existing flows and loadings were summarized for the raw influent, recycle, primary influent and primary effluent. In this section, the design year flows and loadings are presented including the allowances for future growth and the methodology of estimating the primary effluent flows and loads, which are the same as the secondary influent loadings. The primary effluent (secondary influent) loadings are the key input for the biological process modeling of the improvements needed to meet the enhanced nitrogen and phosphorus discharge limits.

It is important to note that the current flows and loads have been determined based on the independent maximum month and peak daily conditions for flows, BOD, TSS and nutrient loadings. The proposed design criteria are based on projections of the current flows and loadings, but it is not anticipated that the peak flows and loading conditions will occur simultaneously for all conditions. The use of the current and design flows and loading data is clarified further in Section 4 on the development of the process model and in Section 5 on the evaluation of nitrogen removal alternatives.

As noted in Section 2 – Existing Flows and Loads and discussed further below, the City of Woonsocket has initiated an inflow and infiltration (I/I) removal program as part of an underground asset management program that is expected to result in wet weather flow reductions. Although this work is in a preliminary stage, a number of important projects have been completed recently or will be in the near future. The impact of the projects completed to date, and the priority of additional I/I work was the subject of a supplemental Inflow and Infiltration Program Paper that is attached in Appendix C, and discussed further in Section 2.2.7. The impact of these projects will be considered as part of the assessment of future growth needs in Section 3.3.1.

The Woonsocket Regional WWTF serves the City of Woonsocket, and the Towns of North Smithfield, Blackstone and Bellingham. The capacity allocation of the WWTF for the Towns of North Smithfield, Blackstone and Bellingham and the City of Woonsocket are summarized in Table 3-1, and were discussed in Section 2.2.6 including a more comprehensive breakdown of flows in Table 2-3. In Table 3-1, each community's current maximum monthly flow has been presented based on its contribution during the two highest maximum monthly flow events at the WWTF during the period from May 2007 through September 2010. Since each community's contribution varied during the two events, the maximum monthly flows for each community are greater than the total of 13.0 mgd at the WWTF. It is also important to note that each community has contributed higher maximum monthly flows during extreme wet weather events, and this is addressed further in Section 2.2.6. As shown in Table 3-1, the Towns of North Smithfield and Bellingham have significant reserve capacity. The Town of Blackstone has adequate reserve capacity, but has exceeded its allocation during extreme wet weather events. The City of Woonsocket has limited reserve capacity as a percentage of existing flows, although quantitatively it is greater than the other communities.

TABLE 3-1
CAPACITY ALLOCATION AND CURRENT FLOWS

Community	**Capacity Allocation (MGD)**	**Current Maximum Monthly Flow (MGD)[a]**	**Reserve Capacity (MGD)**	**Reserve Capacity as Percent of Current Maximum Month Flow (%)**
Woonsocket	12.8	11.5	1.3	10%
North Smithfield	1.92	1.01	0.91	47%
Bellingham	0.80	0.16	0.64	80%
Blackstone	0.48	0.34	0.14	29%
Total	16.0	13.0	3.0	19%

Note:
[a] During extreme wet weather events, North Smithfield has had monthly average flows of 1.37 mgd, Bellingham has had monthly average flows of 0.47 mgd, and Blackstone has had monthly average flows of 0.55 mgd.

3.2 FUTURE GROWTH – DEMOGRAPHICS AND ECONOMIC TRENDS

The projected growth in population over the 20-year planning period is one indicator of expected growth in flows and loads. However, population growth does not always correspond to increases in sewered population. For example, North Smithfield has prepared a facility plan for sewering some existing neighborhoods that currently rely on on-site disposal systems. Conversely, growth in rural areas of a community will often rely on on-site systems rather than connection to the collection system.

The population projections from the U.S. Census are utilized by various state agencies for planning purposes, and the 2000 U.S. Census results are still the data on record until the projections from the 2010 U.S. Census become available. Some 2010 Census data has been released and is included in the analysis in this section. However, population projections based on the 2010 data are not available at this time. Consequently, the projections for 2030 based on the 2000 population census are the best available data. The population estimates and projections for each community are summarized in Table 3-2.

The City of Woonsocket is at or near build-out in terms of existing land use, and anticipates very low to negative growth in population and wastewater flows. The City of Woonsocket was projected to experience a 6% reduction in population by 2030 based on the results of the 2000 U.S. Census as shown in Table 3-2. Recently released population estimates by the U.S. Census indicate that Woonsocket's population decreased from 43,224 in 2000 to 41,186 in 2010, which corresponds to a 4.7% decrease. It is anticipated that when the population projections based on the 2010 U.S. Census results are finalized, the Woonsocket projections may continue to indicate a negative growth rate over the next 20-year planning period. The projected growth in residential and commercial flow and loads would be expected to be at this same level.

TABLE 3-2
POPULATION ESTIMATES AND PROJECTIONS

Community	U.S. Census 2000 Population[a]	U.S. Census 2010 Population[b]	Increase or Decrease from 2000 to 2010[c]	R.I./MA./U.S. Census 2030 Population Projection[a]
Woonsocket	43,224	41,186	-4.7%	40,772
North Smithfield	10,618	11,967	+12.7%	11,207
Bellingham	15,314	16,332	+6.6%	16,642
Blackstone	8,804	9,026	+2.5%	9,852
Totals	77,960[d]	78,511[d]	+0.7 %	78,473[d]

Notes:
[a] U.S. Census 2000; Rhode Island State Planning Program Technical Paper 154, August 2004; and Metropolitan Area Planning Council, January 2006.
[b] U.S. Census 2010 – Recently released.
[c] Population change from 2000 to 2010
[d] Population figures are not for the WWTF service area but rather for the entire communities.

The 2010 Census information for North Smithfield indicates a 12.7% increase since 2000 to 11,967 as shown in Table 3-2. This exceeded the Rhode Island / U.S. Census population projections for the year 2030 as shown in Table 3-2. It is anticipated that when the population projections based on the 2010 U.S. Census results are finalized, the North Smithfield projections will indicate a significant growth rate over the next 20-year planning period exceeding the levels in Table 3-2.

The 2010 Census information for Bellingham indicates a 6.6% increase since 2000 to 16,332 as shown in Table 3-2. The current 2030 population projection of 16,642 appears somewhat low, since it represents only a 1.9% increase over the current estimate. It is anticipated that when the population projections based on the 2010 U.S. Census results are finalized, the Bellingham projections will indicate a growth rate exceeding the levels in Table 3-2 over the next 20-year planning period.

The 2010 Census information for Blackstone indicates a 2.5% increase since 2000 to 9,026 through 2010 as shown in Table 3-2. The current 2030 population projection of 9,852 represents a 9.2% increase. It does not appear that a significant change in the population projections should be anticipated when the 2010 results become available.

The 2010 Census information for all of the communities indicates a 0.7% increase since 2000 to approximately 78,511 through 2010 as indicated in Table 3-2. The current 2030 population projection for all of the communities is 78,473 representing a slight decrease over the 20-year planning period. This projection decrease may not be accurate and will likely be changed when new population projections are available using the 2010 Census data. However, it does suggest that population growth is not expected to be large, and likely to be less than 5% overall.

It should be noted that these population figures are not those of the WWTF service area, as some portions of the member communities do not discharge to the WWTF. However, the population changes of the member communities as a whole are assumed to be indicative of the population changes within the service area.

As noted in Section 3.1, existing flows from each of the communities are within the current allocations. The available population growth data suggests that the member communities will not reach their available allocations. However, it is also important to note that population growth does not necessarily correlate with sewered population growth. As noted above, North Smithfield in particular has plans for extending its collection system into areas served by on-site disposal system. The Facility Plan for North Smithfield projects that monthly average flows will increase to 1.57 mgd due to the increase in the sewered population. This represents a 55% increase over current flows, and greatly exceeds the anticipated population growth. The Towns of Bellingham and Blackstone and the City of Woonsocket have not indicated any plans to extend the collection system in their communities.

Even though current population trends do not support the need over the existing 20-year planning period, all of the communities have indicated that they would like to maintain their current reserve capacity for the proposed WWTF upgrade. For example, the City of Woonsocket would like to maintain reserve capacity to help promote economic development such as growth in industrial flows and loads. Consequently, it was determined that the existing WWTF design criteria for hydraulic capacity will be utilized as discussed further in Section 3.3. The allowance for growth in loads and average flows will be limited to 15%.

3.3 DESIGN FLOWS

As noted in Section 2 – Existing Flows and Loads, the flow conditions that limit capacity are the maximum 30-day moving average (as a measure of sustained flows, referred herein as the maximum monthly flow), peak daily and peak hourly flows. For the Woonsocket WWTF, like virtually every other municipal WWTF, these flow conditions occur during wet weather periods due to the impact of inflow and infiltration (I/I).

3.3.1 Inflow and Infiltration

An analysis of inflow and infiltration levels in the Woonsocket collection system is summarized in Appendix C. One of the key findings of the I/I program evaluation is that there are excessive levels of both inflow and infiltration in the collection system served by the Woonsocket Regional WWTF. For infiltration, the average non-rain weather flow was determined to range from 181 gallons per capita per day (gpcd) to 278 gpcd for the worst applicable period in the years 2007 through 2010. For inflow, the wet weather flow ranged from 213 gpcd to 518 gpcd for the worst applicable period in the years 2007 through 2010. The portion of the collection system in Woonsocket is the oldest within the entire regional service area, and thus is believed to have the most significant issues. However, as noted in Section 2.2.6, the flow data also indicates that the other communities also have excessive I/I levels. Consequently, it appears that all of the communities have justification to perform sewer system evaluation surveys of their collection systems, and to perform repairs to remove sources of excessive I/I.

The specific recommendations for the City of Woonsocket are addressed in greater detail in Appendix C. It is believed that recently completed, and near-term I/I removal projects which include the installation of manhole inserts to minimize inflow from perforated manhole covers, and spot repairs of infiltration "gushers", and rehabilitation of a section of sewer in a low lying swampy area of the City will have a significant impact on reducing the peak wet weather flows to the WWTF.

3.3.2 Design Raw Influent Flows

The design year flows are intended to remain at the allocation levels shown in Table 3-1 for the maximum monthly condition. The plant is currently designed for a maximum monthly flow rate of 16 mgd and its existing infrastructure systems can readily handle this hydraulic flow rate. The provision for reserve capacity over the 20-year planning period (as presented in section 3.4) will be limited to a potential 15% increase from the existing conditions, which changes the average annual design flow only and; therefore, will result in changes to the design loadings (as discussed in Section 3.4). Table 3-3 presents the proposed design flows for the raw influent. For peak hourly flows, the current level has been estimated at 30 to 32 mgd, and the proposed design year peak hourly flow is 37 mgd. This is the flow condition that most directly reflects the expected impact of recently completed and near-term I/I removal projects, as well as the need for continued commitment to I/I removal in order to avoid the need for hydraulic capacity improvements at the WWTF. The updated design peak daily value of 24 mgd also reflects anticipated reductions from the I/I removal projects. The maximum monthly flow is proposed to be 16 mgd, which reflects the full allocation for the communities over the 20-year planning period, and is the permitted maximum monthly flow of the WWTF.

The updated design average flow of 9.0 mgd in Table 3-3 directly reflects the proposed 15% future reserve capacity. This 1.2 mgd increase in average flows over the current conditions is more than the anticipated growth in the communities over the planning period. Only the Town of North Smithfield has quantified an increase in flows over the planning period, and this allowance provides for the anticipated growth in monthly average flows to 1.57 mgd, which corresponds to a 0.56 mgd increase over its current maximum monthly flow. However, it is important to reiterate that hydraulically the WWTF capacity is not limited by the average flow condition, and the design year sustained flow condition will be based on the maximum monthly flow of 16.0 mgd.

By limiting the anticipated growth (or future reserve capacity allowance) to 15% over the 20-year planning period, the magnitude of the anticipated increase in peak hourly and peak daily flows will also be reduced somewhat, and this limits the level of I/I removal that must be achieved. However, it is also important to note that the current maximum monthly average flow has reached 15.8 mgd during the extraordinary flooding event of 2010 and 14.7 mgd during another extremely wet weather month. Therefore, it will be important for Woonsocket to continue their I/I remediation program in order to: (1) ensure that future maximum month average wet weather flows are within the 16 mgd RIPDES permit limit; (2) accommodate the member communities' flow allocations; (3) provide sufficient reserve capacity to promote possible future economic growth in Woonsocket and; (4) ensure that peak wet weather flows are within the design parameters for the wastewater treatment processes.

The proposed design criteria have been established to help limit the extent of the improvements that are needed for the immediate nitrogen and phosphorus removal projects, and to allow the project to be affordable for the City of Woonsocket and the member communities.

TABLE 3-3
DESIGN YEAR FLOWS

Item	Existing Design 2000	Current 2010[a]	Design 2030
Raw Influent Flows (MGD):			
Average Annual	16	7.8	9.0
Maximum Monthly	16	13.0[b]	16.0
Peak Daily	24	22.2[b]	24
Peak Hourly	32	~32[b]	37
Recycle Flows (MGD):			
Average Annual	2.6	3.9	3.9
Maximum Monthly		5.6	5.1
Peak Daily		8.5	5.1
Peak Hourly		-	5.1

Item	Existing Design 2000	Current 2010[a]	Design 2030
Primary Inf./Eff. Flows (MGD):			
Average Annual	18.6	11.7	12.9
Maximum Monthly	18.6	18.1 [c]	21.1
Peak Daily	26.6	26.2[c]	29.1
Peak Hourly	34.6	35[c]	37.1

Notes:

[a] Based on data from May 2007 through September 2010.

[b] The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data for the raw influent indicates a maximum monthly flow of 15.8 mgd, a peak daily flow of 29.1 mgd, and a peak hourly flow of approximately 38 mgd.

[c] The value shown excludes March 30, 2010 flood event. During the March 30, 2010 flood, the flow data for the primary influent indicates a maximum monthly flow of 22.3 mgd, a peak daily flow of 34.6 mgd, and a peak hourly flow of approximately 42 mgd.

3.3.3 Recycle Flows

The proposed design year recycle flows are shown in Table 3-3. As noted in Section 2, the existing recycle flows are estimated as the difference between the primary effluent and final effluent flow meters. The resulting values are less than the level estimated from analysis of the data available for the individual sources. There are certain recycle flows (e.g., centrate from merchant liquid sludge, primary sludge and condensate) that are not measured by the flow meter differential method, which may account for the lower estimated recycle flows. This difference in the two estimation methods could also be due to differences in the accuracy of the two flow meters, and there is no clear data to ascertain whether the data from one of the flow meters is more reliable than the other. Thus, it is necessary upon evaluation of the available data and projected estimates to rely on engineering judgment in establishing the updated design recycle flows.

It is also important to note that the proposed WWTF improvements for enhanced nitrogen and phosphorus removal are anticipated to result in changes in the return flows, although the overall magnitude of the change is not anticipated to be great. Consequently, the changes in return flows based on the recommended plan will be addressed in the Preliminary Design Report that will be part of the implementation plan for this Facility Plan Amendment.

The design year average recycle flow is recommended to be the same as the current measured value, rather than estimated recycle flows based on the analysis of the sources summarized in Table 2-2. This is based on the primary effluent flume data as providing the more conservative flow data. For the maximum monthly, peak daily and peak hourly recycle flow conditions, the recommended design value for all three conditions is 5.1 mgd based on the total of the individual sources presented in Table 2-2. The resulting primary influent design year flows appear reasonable in comparison to both the current and existing design flows.

3.3.4 Primary Influent/Effluent Flows

The design year primary influent/effluent flows are based on the sum of the raw influent and the recycle flows as shown in Table 3-3, and are the same as the secondary influent flow. The proposed design year primary influent/effluent flows represent a notable increase over the previous design flows, as well as the actual current flows.

3.4 DESIGN LOADINGS

The design year loadings for BOD, TSS, and nutrients are shown in Table 3-4 for the raw influent, recycle, primary influent and primary effluent. It was expected that the loadings would change from the initial design values due to the combination of the 15% reserve capacity flow allowance (for member communities and Woonsocket) for raw influent loadings and the increase in the centrate flow requested by Synagro to allow for increased quantities of merchant liquid sludge, as well as the revised data for influent BOD and Recycle TSS. The intent of the design raw influent loads is to provide sufficient reserve capacity to support future economic development opportunities, as well as to accommodate the increased anticipated growth in loads from the member communities over the 20-year planning period.

As discussed in Section 2 Existing Flows and Loads, over the past year, the facility has had a significant decrease in BOD loadings which has been identified as a result of recent operational changes at a local industry that discharges to the WWTF. This change in influent loadings will be considered when developing design criteria and the criteria reflects some discharge from Technics (and other city-wide industries); therefore, the industrial loadings are considered conservative in the Facility Plan Amendment.

In addition, the requirements for the full future allocation for all communities (future flows and loads) will be evaluated during the preliminary design process to provide for future expandability of critical process equipment systems, such as the ability to provide additional aeration capacity (additional or larger blowers, locations for additional diffusers, etc.) in the future to handle variations in organic loadings, if necessary. The recommended design approach is to maximize the re-use of all existing unit processes to the extent possible, however further evaluations will be performed during detailed design to confirm the adequacy of unit process sizing and determine whether changes are necessary to improve process performance.

Table 3-4 summarizes the WWTF design loadings from 2000, the 2010 observed loadings, as well as the proposed design loadings. The "revised updated design 2030" loadings are the recommended design criteria for the WWTF for the planning period, which has been revised based on additional data through July 2012.

TABLE 3-4
UPDATED DESIGN LOADINGS

Item	Existing Design 2000	Observed in 2010 [a]	Updated Design 2030[c]	Revised Updated Design 2030[c]
Raw Influent Loadings (lb/d):				
BOD5:				
Annual Average	21,350	14,200	16,330	19,715
Maximum Monthly		27,100	31,170	31,170
Peak Daily 98th Percentile		37,000	42,550	d
100th Percentile		51,300	59,000	66,481
TSS:				
Annual Average	16,013	11,600	13,340	13,340
Maximum Monthly		18,800	21,620	23,158
Peak Daily 98th Percentile		25,900	29,790	d
100th Percentile		39,200	45,080	60,825
Ammonia:				
Annual Average	2,669	1,146	1,320	1,320
Peak Daily 98th Percentile		1,997	2,300	d
100th Percentile		1,997	2,300	2,300
Total Kjeldahl Nitrogen:				
Annual Average	5,338	1,800	2,070	2,070
Peak Daily 98th Percentile		3,267	3,760	d
100th Percentile		3,629	4,170	4,242

TABLE 3-4
UPDATED DESIGN LOADINGS

Item	Existing Design 2000	Observed in 2010 [a]	Updated Design 2030[c]	Revised Updated Design 2030[c]
Phosphorus:				
Annual Average	2,268	237	270	270
Peak Daily 98th Percentile		354	410	d
100th Percentile		368	420	427
Recycle Loadings (lb/d):				
BOD5:				
Annual Average	4,644	10,000	10,830	9,028
Maximum Monthly		19,400	20,760	19,162
Peak Daily 98th Percentile		27,700	28,690	d
100th Percentile		45,570	47,560	46,783
TSS:				
Annual Average	6,882	15,900	16,160	13,218
Maximum Monthly		54,300	54,930	19,740
Peak Daily 98th Percentile		56,100	58,390	
100th Percentile		190,600	198,770	34,941
Ammonia:				
Annual Average	1,830	319	360	360
Peak Daily 98th Percentile		575	640	640
100th Percentile		580	640	640
Total Kjeldahl Nitrogen:				
Annual Average		1,182	1,230	1,230
Peak Daily 98th Percentile		2,560	2,640	2,199
100th Percentile		2,849	2,930	2,930
Phosphorus:				
Annual Average		861	890	72
Peak Daily 98th Percentile		2,333	2,380	154
100th Percentile		2,621	2,670	216

TABLE 3-4
UPDATED DESIGN LOADINGS

Item	Existing Design 2000	Observed in 2010 [a]	Updated Design 2030[c]	Revised Updated Design 2030[c]
Primary Influent Loadings (lb/d):				
BOD5:				
Annual Average	25,994	24,000	26,960	28,742
Maximum Monthly		42,500	47,930	38,895
Peak Daily 98th Percentile		50,000	57,540	59,512
100th Percentile		65,500	75,190	86,233
TSS:				
Annual Average	22,895	34,100	36,100	26,558
Maximum Monthly		74,300	77,750	40,985
Peak Daily 98th Percentile		91,900	98,080	59,073
100th Percentile		170,800	178,970	77,705
Ammonia:				
Annual Average	4,499	1,057	1,270	1,6800
Peak Daily 98th Percentile		1,605	1,970	2,183
100th Percentile		1,615	1,980	2,619
Total Kjeldahl Nitrogen:				
Annual Average	5,338	2,772	3,090	3,300
Peak Daily 98th Percentile		4,438	5,010	4,718
100th Percentile		4,948	5,570	5,949
Phosphorus:				
Annual Average	2,268	1,206	1,240	342
Peak Daily 98th Percentile		2,687	2,740	680
100th Percentile		2,989	3,040	974
Primary Effluent (Summer Season)				
Loadings (lb/d):				
BOD5:				
Annual Average	18,095	13,900	15,610	18,000
Maximum Monthly		28,900[b]	32,590	22,000
Peak Daily 98th Percentile		30,600	35,210	[d]
100th Percentile		48,200[b]	55,330	[d]

TABLE 3-4
UPDATED DESIGN LOADINGS

Item	Existing Design 2000	Observed in 2010 [a]	Updated Design 2030[c]	Revised Updated Design 2030[c]
TSS:				
Annual Average	6,004	7,400	7,830	7,500
Maximum Monthly		12,700[b]	13,290	11,600
Peak Daily 98th Percentile		14,600	15,580	[d]
100th Percentile		28,500[b]	29,860	[d]
Ammonia:				
Annual Average	5,410	1,231	1,380	1,600
Max. Month – 95th Percentile		2,091	2,360	1,770
Peak Daily 98th Percentile		2,445	2,810	[d]
100th Percentile		2,874	3,300	[d]
Total Kjeldahl Nitrogen:				
Annual Average		2,415	2,560	2,430
Max. Month – 95th Percentile		5,188	5,430	2,660
Peak Daily 98th Percentile		6,585	7,030	[d]
100th Percentile		6,925	7,260	[d]
Phosphorus:				
Annual Average		677	720	167
Max. Month – 95th Percentile		1,363	1,430	207
Peak Daily				
98th Percentile		1,692	1,810	[d]
100th		1,958	2,050	[d]

Notes:

[a] Based on data from May 2007 through September 2010.

[b] b. Highest value excluding period from March 16, 2010 to April 23, 2010 when one of the primary clarifiers was disabled.

[c] Based on data from January 2010 through July 2012. Revised data is based on a primary clarifier TSS removal of 70% and chemically enhanced primary treatment for phosphorus removal

[d] The 98th percentile and 100th percentile loadings for nutrients are not being revised, as they do not affect the plant design. The annual average and 98th percentile TSS and BOD loadings are also not being revised for the same reason.

Table 3-4A Design Loads for Raw Influent, Industrial Flows and Design Plant Influent provides the design loads for the raw influent into the treatment plant, the industrial loads and the total basis plant full loads. The “No Industrial” loads are based on engineering judgment as to typical municipal wastewater characteristics at the Woonsocket WWTF. These loads are also consistent with municipal standard per-capita loads. The allocations for industrial loads are based on monthly maximum loads attributable to the largest industrial discharger (Technics) in the system with a 15 percent additional factor for other industrial facilities. The design basis full load adds the estimated raw influent and industrial loads.

TABLE 3-4A
DESIGN LOADS FOR RAW INFLUENT, INDUSTRIAL FLOWS, AND DESIGN PLANT INFLUENT

Condition	TSS	BOD	TKN	Ammonia	Total Phosphorus
Raw Influent Loads (lb/day) - No Industrial					
Average	12,500	15,000	1,900	1,200	240
Max Month	21,700	23,800	2,400	1,500	280
Max Week	29,200	30,500	2,800	1,600	300
Max Day	57,000	50,600	3,900	2,100	380
Industrial Loads (lb/day)					
Average	840	4,715	170	120	30
Max Month	1,458	7,370	212	142	34
Max Week	1,962	9,585	245	159	37
Max Day	3,825	15,881	342	209	47
Design Basis Full Load (lb/day) (Raw + Industrial)					
Average	13,340	19,715	2,070	1,320	270
Max Month	23,158	31,170	2,612	1,642	314
Max Week	31,162	40,085	3,045	1,759	337
Max Day	60,825	66,481	4,242	2,309	427

3.4.1 Raw Influent

The design year loadings for the raw influent are shown in Table 3-4 and Table 3-4A. The design loadings are based on the revised current loadings, which were discussed in Section 2 – Existing Flows and Loads. These revised loads include a 15% reserve capacity flow allowance. As discussed in Section 2, the loadings from the local industry have been accounted for in the design flows presented in Table 3-4 and are further broken down in Table 3-4A. If significant operational changes were under taken at the industrial discharger, it is likely that there would be an impact to the raw influent loadings values that the City would need to address with the industry discharger. However, the industrial load allocation has been included to better understand how the Local Limits Analysis is integrated with other plant loads. At this time, the actual needs of this industry are fairly well understood and therefore the design loads presented on Table 3-4 and Table 3-4A with the future reserve capacity allocation are considered adequate based on meetings with the industry discharger and the historic influent loading data as discussed in Section 2 – Existing Flows and Loads.

3.4.2 Recycle

The current recycle loadings are based on sampling and analysis of the combined recycle flows as discussed in Section 2 – Existing Flows and Loads and summarized in Table 3-4. The Solids Handling Facilities represent the largest single source of loadings, therefore any expected changes in the return loadings from the facilities are important and need to be taken into consideration in terms of impacts to the nutrient removal capabilities of the WWTF in the future.

3.4.3 Primary Influent

The current primary influent loadings are measured at the outlet of the aerated grit chamber as discussed in Section 2 – Existing Flows and Loads and summarized in Table 3-4.The loadings are also measured on the raw stream and the recycle streams to the WWTF. The design year primary influent loadings were based on adding the average raw influent and recycle loadings together. The historical peaking factors for each primary influent component were then applied to the average values to develop the monthly/weekly/peak day primary influent loads. It is important to note that the sum of the raw and recycle peak loads for the various conditions will not add up to the revised maximum month/maximum week/maximum day primary influent loads. This is a direct result of the fact that peak recycle loads do not happen at the same time as the peak influent loads, as shown in the historical data set.

3.4.4 Primary Effluent/Secondary Influent

The updated design year primary effluent loadings and secondary influent loadings are based on the design year Pro2d Model Mass Balance for the summer nutrient removal season, which assumes a 70 percent TSS removal across the primary clarifier and some degree of soluble phosphorus removal resulting from ferric chloride addition in the primary feed.

3.5 TEMPERATURES

As noted in Section 2 – Existing Flows and Loads, the wastewater temperature at the Woonsocket Regional WWTF increases significantly from the raw influent to the aeration basins due to the effect of the scrubber water recycle flows (S-1) from the incineration facilities. The design wastewater temperature is a key concern because of the impact on nitrification rates. During warm weather conditions, a cooling tower for the scrubber water recycle flow is used to prevent excessively high wastewater temperatures. During cold weather conditions, the key concern is the minimum temperature, and thus whether the incinerator is operating. With the proposed discharge permit limits, the low temperature concerns can be further broken down to April when there is a total nitrogen limit of 10 mg/l and then from May through October when there is a 3 mg/l total nitrogen limit. Table 3-5 shows the projected design temperatures for April and then for the period from May through October.

TABLE 3-5
PROJECTED DESIGN YEAR WASTEWATER TEMPERATURES

Condition	Existing Raw Influent	Existing Aeration Basin	Design Year Aeration Basin	Design Year Aeration Basin w/CoGen
April:				
Minimum Monthly	13.6	19.9	19.9	19.1
Minimum 7-Day	12.9	12.4/20.6[a]	12.4	12.4
Minimum Day	11.5	10.5/19.8[a]	10.5	10.5

TABLE 3-5
PROJECTED DESIGN YEAR WASTEWATER TEMPERATURES

Condition	Existing Raw Influent	Existing Aeration Basin	Design Year Aeration Basin	Design Year Aeration Basin w/CoGen
May 1 to October 31:				
Minimum Monthly	16.2	23.7	23.7	22.9
Minimum 7-Day	13.9	22.4	13.9	13.9
Minimum Day	13.0	19.0	13.0	13.0

Note:
[a] First value is minimum from available data, and second excludes periods that incinerator was off-line.

As noted in Section 2 – Existing Flows and Loads, there was one extended and presumable unplanned shutdown of the incinerator facilities during the 3-year period evaluated in establishing the current flows and loads. This occurred in early April 2010, and appears to be in response to the flooding that started on March 30. Based on discussions with Synagro, it appears that an unplanned shutdown of up to a week is a potentiality that must be accounted for in design criteria. It should be noted that the available data shows that the aeration basin temperatures were lower than the raw influent temperatures during the shutdown period in April 2010. This is apparently due to cooling of the wastewater due to lower ambient temperatures. For the period from May through October, the ambient temperature will be higher, and it was assumed that there would not be a significant cooling effect.

Synagro has been approved by the City and is currently installing energy recovery facilities on the exhaust of the incinerator. This will result in electrical power generation that will benefit the City and Synagro. The installation of the energy recovery facilities will slightly reduce the temperature of the tray scrubber recycle but will not significantly affect the nitrogen removal upgrades. These facilities and changes to influent temperature have been taken into consideration in the design of the upgrades. The impact of the energy recovery facilities on the expected aeration basin wastewater temperature for both April and from May 1 to October 31 was analyzed. Synagro prepared heat balances for different operating scenarios over a range of monthly average flows to determine the estimated increase in aeration basin temperatures as shown in Figure 3-1. The heat balances assume a raw influent temperature of 12°C, and the actual temperature increase will be greater at higher raw influent temperatures. This is conservative for the current temperature considerations.

Figure 3-1 includes: a curve that shows the predicted temperature increase when the incinerator is operated at 100% load, which is the typical operating condition; a curve showing the impact of the proposed co-generation facilities when the incinerator is at 100% load; and a curve showing the predicted temperature increase when the incinerator is operated at 80% load, which is not common. At the current maximum monthly flow of 13 mgd, the temperature increase is typically 7.8°C for the incinerator at 100% load. In Table 3-3, the temperature increase in April is only 6.3°C due to the incinerator being off-line for 7 days in the month of record, while the temperature increase in May is only 7.5°C due to the incinerator not being on-line continuously. The 80% load curve in Figure 3-1 is another way of illustrating the impact of the incinerator operating at less than full capacity.

FIGURE 3-1
INCREASE IN PRIMARY INFLUENT TEMPERATURE FROM INCINERATION OPERATING ALTERNATIVES

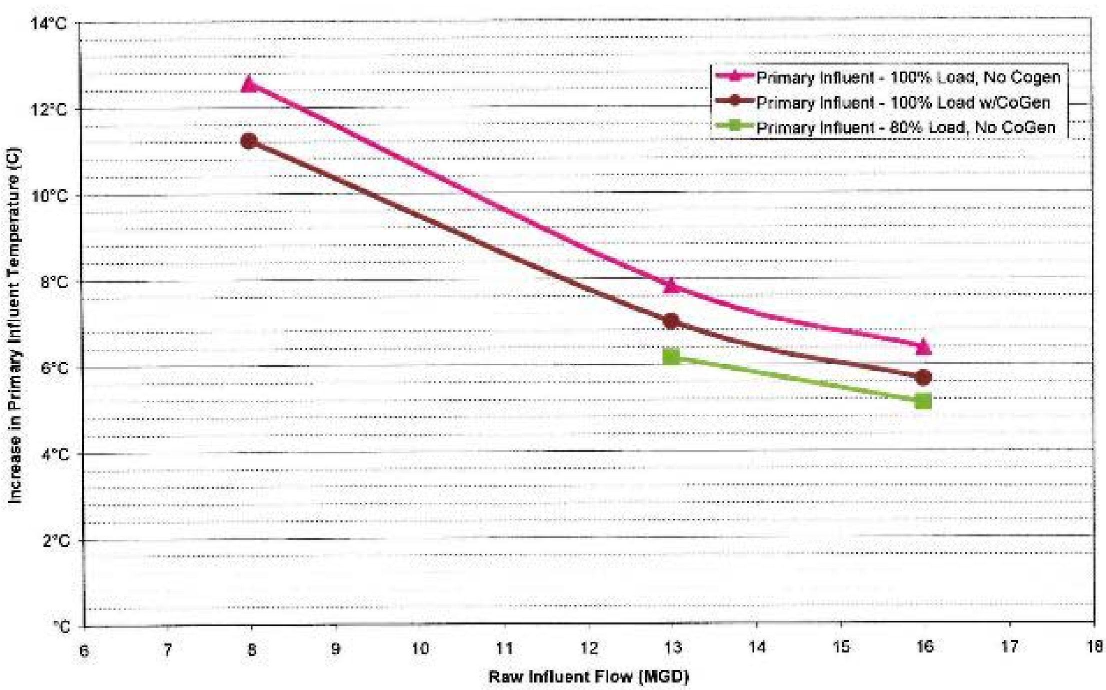


Figure 3-1 shows the impact of the proposed co-generation facility when the incinerator is operating at 100% load. For the current maximum monthly flow condition, the typical increase is 7.0°C, which corresponds to a 0.8°C decrease compared to operating without the proposed cogeneration system. In comparison to the April design condition in Table 3-3, the impact of co-generation would be less than the impact of the incinerator being off-line for 7 days in the month of record. A 0.8° C reduction in the influent wastewater will not significantly impact the nitrogen removal performance of the facilities. However, the conservative approach for evaluating the impact of the co-generation facilities is to assume a 0.8°C decrease for the design monthly conditions shown in Table 3-5. This provides a level of conservatism to account for the incinerator being off-line during the critical temperature period of the year, which is typically April and May.

SECTION 4
PERFORMANCE OF THE SECONDARY TREATMENT FACILITIES AND PROCESS MODEL DEVELOPMENT

The process model development included in Section 4 has been superseded by CH2M HILL in Section 4.A.

4.1 INTRODUCTION

This section describes the development of the BioWin process model and includes an operational review of the existing MLE activated sludge system. This MLE process provides carbonaceous BOD removal, nitrification, and denitrification. Based on the specific operational approach, the system also achieves enhanced biological phosphorus removal.

The goals of the operational review include:

- Present the typical operation of the facility and assess whether operational changes would be beneficial for nitrogen and phosphorus removal;
- Review, evaluate and summarize the performance of the existing unit processes;
- Provide operational knowledge for the development of a biological process model of the secondary treatment system to evaluate plant capacity and evaluate enhanced nutrient removal alternatives.

4.2 EXISTING SECONDARY TREATMENT SYSTEM OPERATION

The MLE activated sludge process is comprised of two individual unit treatment processes; carbonaceous BOD and nutrient removal in the aeration tanks and secondary clarification as shown in Figure 4-1. Biological conversion of the organic material and nutrients occurs in the aeration tanks, while solids separation takes place in the secondary clarifiers. Both unit processes are interdependent and therefore must be evaluated as an integrated system. A complete summary of current flows and loads is provided in Section 2.

4.2.1 Description of the Existing Treatment Process

The existing aeration basins are configured for the Modified Ludzack-Ettinger (MLE) nitrogen removal process as shown in Figure 4-1. The MLE process is used extensively for the biological removal of nitrogen down to levels of 6 to 10 mg/l of total nitrogen. The original basis of design is summarized in Table 4-1, and more detailed basic design data in provided in Appendix A.

The MLE process is configured to have one or more anoxic reactors precede the aerated reactor(s) of the activated sludge system. The primary effluent, return activated sludge (RAS), and an internal nitrate recycle stream are all fed into the anoxic reactor. This is intended to maximize the use of the organic carbon present in the influent wastewater for denitrification. To achieve biological nitrogen removal, the ammonia must be converted to nitrate (nitrification) in the oxic or aerated zone of the activated sludge system. The nitrates produced in the aerobic zones are recycled back to the anoxic zone through a pumped internal recycle system and also via return activated sludge (RAS). Denitrification requires a carbon source which is provided by the soluble BOD_5 in the secondary influent.

The four most easterly aeration tanks shown in Figure 4-2 were constructed in the mid-1970s upgrade. These tanks originally included both mechanical surface aerators and coarse bubble aeration. The diffused aeration system was converted to fine bubble in the 1 990s, and the mechanical surface aerators were abandoned. The existing tanks were modified for the MLE process and two additional tanks (Nos. 5 and 6) were added as part of the capital improvements to implement the 2000 Facility Plan Amendment. The existing coarse bubble system was converted to fine bubble membrane diffusers as part of the improvements.

FIGURE 4-1
EXISTING MLE PROCESS FLOW DIAGRAM

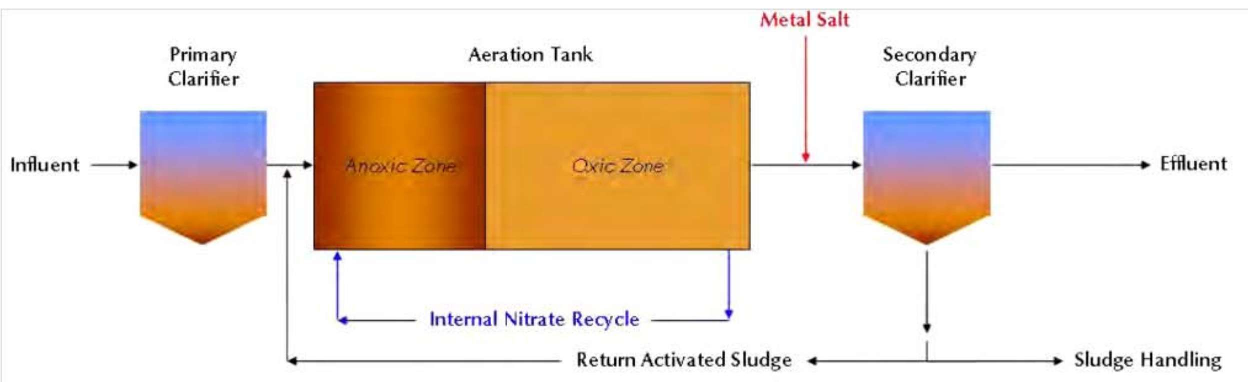


TABLE 4-1
MLE ACTIVATED SLUDGE PROCESS – BASIS OF DESIGN

	Design Operating Conditions
Aeration Tanks	
Number of Units	6
Unit Dimensions	
Total Width, ft	46
Total Length, ft	184
Side Water Depth, ft	15
Anoxic Volume, mgal	1.42
Total Volume, mgal	5.69
Aeration	
Number of blowers	4
Aeration Capacity, cfm @ 7.5 psig	6,500
Type of Blower	2 x Variable-speed multistage centrifugal (1977) 2 x Variable-speed multistage turbo centrifugal (2010)
Maximum HP, each	300
Diffusers	Fine-bubble membrane
Nitrate Recycle Pumps, per Tank(1), (2)	1
Rating Point, Minimum	1850 gpm @ 0.4 ft TDH
Rating Point, Maximum	5500 gpm @ 3.4 ft TDH
Secondary Clarifiers	
Number of Units	3
Unit Dimensions	
Diameter, ft	110
Sidewater Depth, ft	11.6
Volume, mgal, each	0.8 17
Sludge Removal Mechanism	plow and rake
Design Criteria	
Surface Overflow Rate(16 mgd), gpd/sf	561
Surface Overflow Rate (32 mgd), gpd/sf	1,122
Solids Loading Rate, lbs/sf-day (3)	28.1
Detention Time (16 mgd), hrs	3.7

Notes:
[1] Current operation does not utilize nitrate recycle pumps
[2] Nitrate recycle pumps can provide recirculation rate of up to 4Q, while typical rates range from 2Q to 3Q
[3] Solids loading rate based upon 16 mgd, 100% RAS, 3,000 mg/L MLSS

FIGURE 4-2
AERIAL VIEW – EXISTING AERATION TANKS

Flow is distributed to the six tanks via a common influent channel. Tank No. 5 is rectangular and similar in shape to the original four tanks, while Tank No. 6 is U-shaped. As discussed further below, it appears that there is uneven flow distribution to the tank, which may be due in part to the asymmetrical configuration of the channel inlets. The elevations of the outlet weirs have been checked via survey by Veolia staff and there seems to be some discrepancy from what is shown on the 1977 upgrade record drawings.

Each aeration tank includes three submerged concrete baffle walls that separate the basins into zones. The initial zone of each tank is an anoxic zone that is mixed using three submersible mechanical mixers per tank, and also includes diffusers for aeration if air or oxygen is required. The remaining zones contain diffusers for aerobic operation. The aeration air was originally provided by four multistage centrifugal blowers each with a capacity of 6,500 cfm at 7.5 psi and automated inlet damper control. Just recently in 2010, two new variable-speed turbo blowers have been installed to replace two of the existing units. The aeration blowers are controlled via an automatic dissolved oxygen system to maintain the desired oxygen concentration in each tank. The dissolved oxygen control system utilizes individual dissolved oxygen sensors, automatic throttling of inlet valves for the older blowers and variable speed drives for the new blowers. Airflow meters and control valves allow operators the ability to maintain and balance the system.

Internal recycle is provided by propeller pumps that transfer the mixed liquor from the last aerobic zone of each tank to the anoxic zone at the influent end of the tank. The purpose of this recycle flow is to transfer nitrates back to the anoxic zone for denitrification. This is typically preferred over higher return sludge rates, because it does not impact the solids loading to the secondary clarifiers. The internal recycle is not currently utilized at the Woonsocket WWTF due to problems with high oxygen levels in the mixed liquor returned to the anoxic zone.

The mixed liquor flows from the six aeration basins to Junction Box No. 3 for distribution to the three secondary clarifiers. The clarifiers allow the solids to be settled out and either returned to the aeration tanks or wasted from the system, while the secondary effluent flows on to another distribution structure where it can be directed to either the Effluent Filter Building or the Chlorine Contact Tanks. The three secondary clarifiers were constructed in the mid-1970s upgrade, and are 110-foot in diameter circular tanks with 11.6-foot side water depth as shown Figure 4-3. The clarifiers have rapid sludge withdrawal type mechanisms for solids removal.

Settled sludge is withdrawn from the RAS well in the center column of each clarifier using the return activated sludge (RAS) pumps located in the Return Sludge Pump Station. There are four 3,500-gpm RAS pumps in total, and with three units operating (one dedicated to each clarifier) the total return sludge capacity is 15.1 mgd. The RAS can be pumped back to the influent channel of the aeration tanks or directly to the influent end of each tank which is currently the preferred operational approach of plant staff; it allows a more even distribution of RAS to the tanks. Currently RAS rates are measured to each gooseneck return line by a mag meter. This is an efficient way to maintain food-to-microorganism (F:M) ratios in each individual basin and also allows measurement of removal rates for ammonia and phosphorus treatment.

Waste activated sludge is removed from a hopper at the bottom of each clarifier using a common 10-inch waste sludge pipe and waste activated sludge (WAS) pumps. The suction piping is typically dedicated to a single clarifier for wasting. WAS is pumped to the gravity thickener for co-thickening with primary sludge.

FIGURE 4-3
AERIAL VIEW – EXISTING SECONDARY CLARIFIERS

4.2.2 Typical Operating Parameters

The key operating parameters utilized to evaluate the MLE activated sludge system include mixed liquor suspended solids (MLSS) concentration, waste activated sludge (WAS) production, sludge yield, sludge residence time (SRT), secondary clarifier surface overflow rate, and the secondary clarifier solids loading rate. A summary of these major operating parameters for the period from 2007 to 2010 are presented in Table 4-2.

4.2.2.1 Mixed Liquor Suspended Solids Inventory

The MLSS inventory has been controlled by wasting to maintain a mixed liquor setpoint of about 4,000 mg/l as shown in Table 4-2. Most recently, the MLSS setpoint has been increased to 5,000 mg/L as shown in Figure 4-4 based on the increase in loadings in 2010. The food-to-microorganism (F:M) ratios in the aeration tanks has generally ranged from approximately 0.1 to 0.2 lbs BOD/lbs MLVSS per day, which is typical for the MLE nutrient removal process. As also shown in Figure 4-4, the recent increase in MLSS has maintained the F:M in this typical range. Supplemental sampling of RAS and WAS during October/November 2010 indicated a volatile content in the range of 78 to 87 percent.

TABLE 4-2
MLE PROCESS – TYPICAL OPERATING PARAMETERS
(MAY 2007 – SEPTEMBER 2010)

	Operating Conditions
Aeration Basin Influent	
Average Daily Flow Rate, mgd	11.7
Peak Daily Flow Rate, mgd	26.2
Average Daily Influent BOD5 Load, lbs/day	13,900
Aeration Basin	
MLVSS, mg/l	3,232
MLSS, mg/l	4,040
Food to Mass Ratio (BOD5/MLVSS)	0.10 – 0.20
Dissolved Oxygen, mg/l	1 to 7
Hydraulic Residence Time, hrs	11.7
Sludge Residence Time, days	19 to 40
Average Temperature, °C	18.0
Nitrate Recycle Flow, MGD	0 2
Return Activated Sludge Flow, MGD	9.5
Secondary Clarifier	
Average Surface Overflow Rate, gal/ft^2-day	410
Peak Daily Surface Overflow Rate, gal/ft^2-day	918
Average Day Solids Loading Rate, lbs/ft^2-day	28
Peak Day Solids Loading Rate, lbs/ft^2-day	62
Waste Sludge, lbs/day	5,000-13,000

Notes:
[1] Solids loading rate based upon 100% RAS, current average or peak day flows, 4,040 mg/L MLSS
[2] Current Operation does not utilize the nitrate recycle pumps

4.2.2.2 Waste Activated Sludge Quantities

Operators have recently changed wasting operations from intermittent wasting to continuous wasting. Continuous wasting maintains a more consistent SRT in the system, and provides overall stabilization to the biological process and gravity thickener operation. Figure 4-5 presents wasting data from September to November 2010 at the facility, along with aerobic solids residence times for the secondary system. As shown, the facility wasted an average of approximately 13,000 lb/d during this period. The corresponding aerobic SRTs for this period ranged from 15 to 25 days, which is typical for an MLE nutrient removal process. However, given the higher wastewater temperature at the Woonsocket facility, the activated sludge system should be able to perform successfully at a lower SRT. The Water Environment Federation (WEF) Manual of Practice indicates that a minimum aerobic SRT between 8 and 15 days (dependent on temperature) should be sufficient for complete nitrification.

FIGURE 4-4
MIXED LIQUOR SUSPENDED SOLIDS AND F:M RATIOS (2007 – 2010)

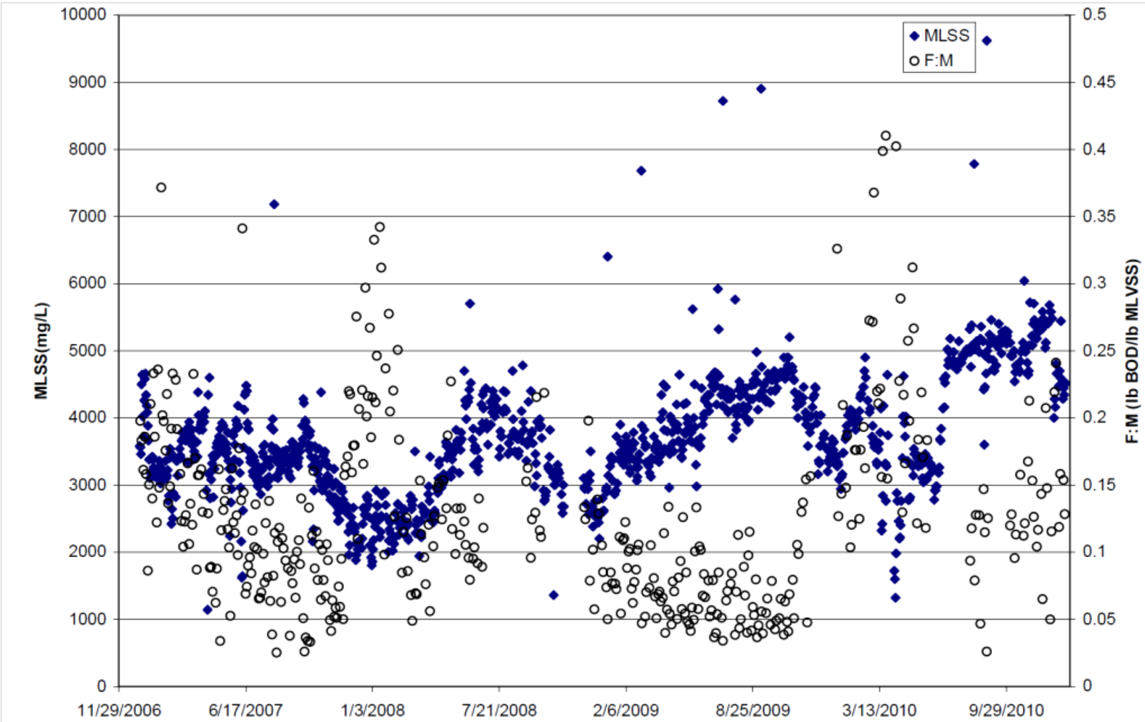
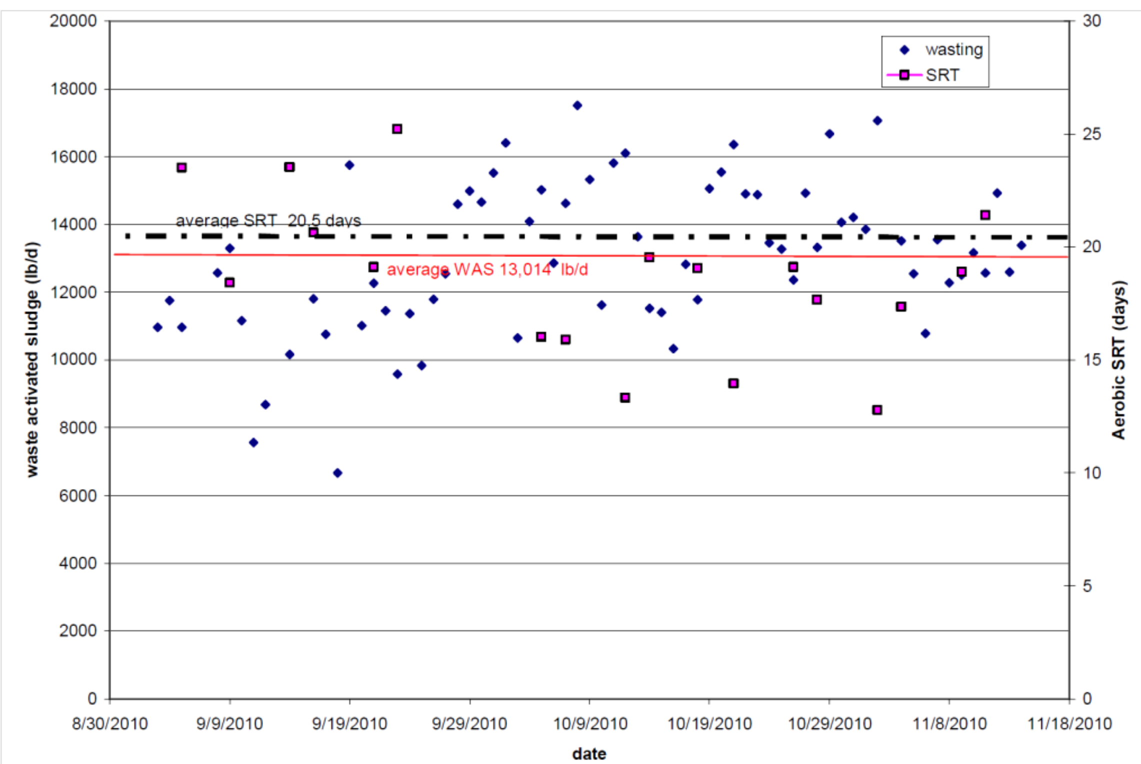


FIGURE 4-5
WASTE ACTIVATED SLUDGE (SEPTEMBER – NOVEMBER 2010)



Prior to September 2010 WAS rates were assumed to be approximately equal to RAS rates as plant staff did not measure the waste activated sludge suspended solids concentration. However, an estimate of the aerobic sludge retention time and corresponding sludge yield was developed for the four-year period from 2007 through 2010 as shown in Figures 4-6 and 4-7. An analysis

of recent plant data revealed that in general the waste activated sludge concentration was 30% greater than the measured return activated sludge concentration.

4.2.2.3 Aeration

Average blower airflow rates at the facility range from 10,000 to 13,000 cfm, which can be accomplished with two blowers in service. Dissolved oxygen (DO) concentrations are measured in all six aeration tanks to ensure adequate oxygen is supplied to the system. Figure 4-8 shows recorded DO levels for the tanks during the summer of 2010. The variability in DO levels in Figure 4-8 indicates that there are difficulties in both distributing airflow evenly and maintaining a set point DO concentration.

FIGURE 4-6
AEROBIC SOLIDS RETENTION TIME (2007 – 2010)

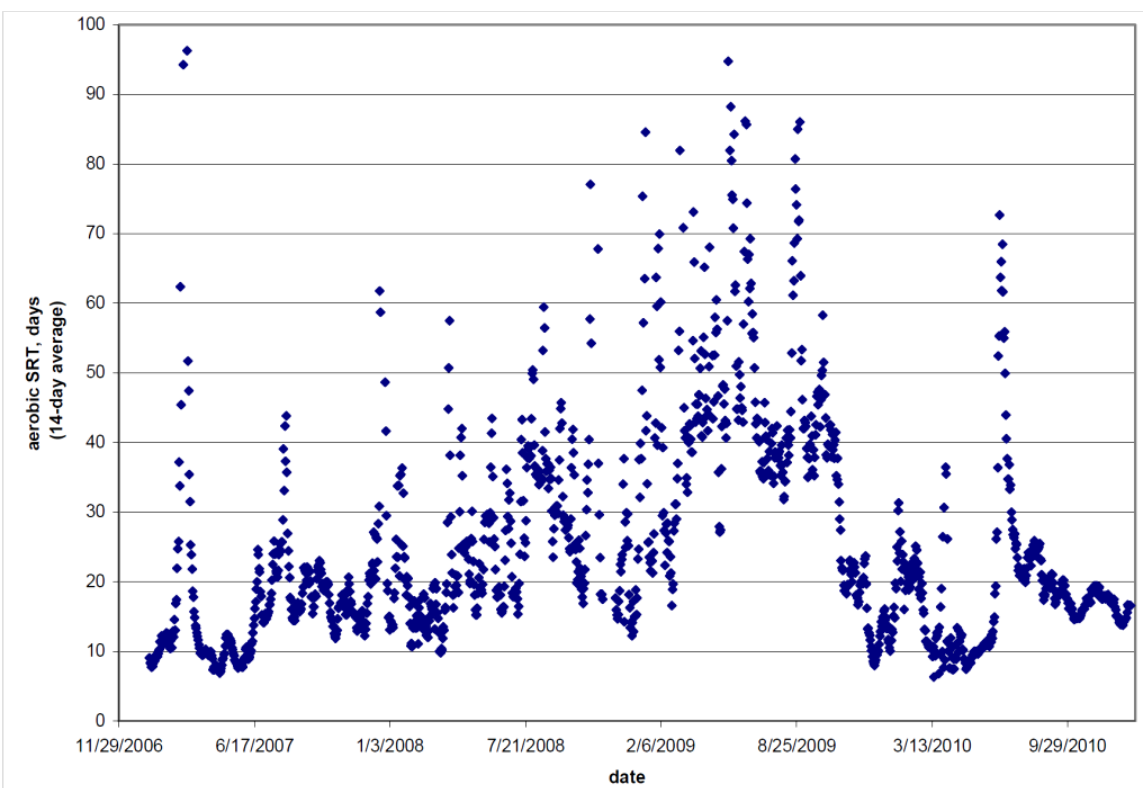


FIGURE 4-7
BIOLOGICAL YIELD (2007 – 2010)

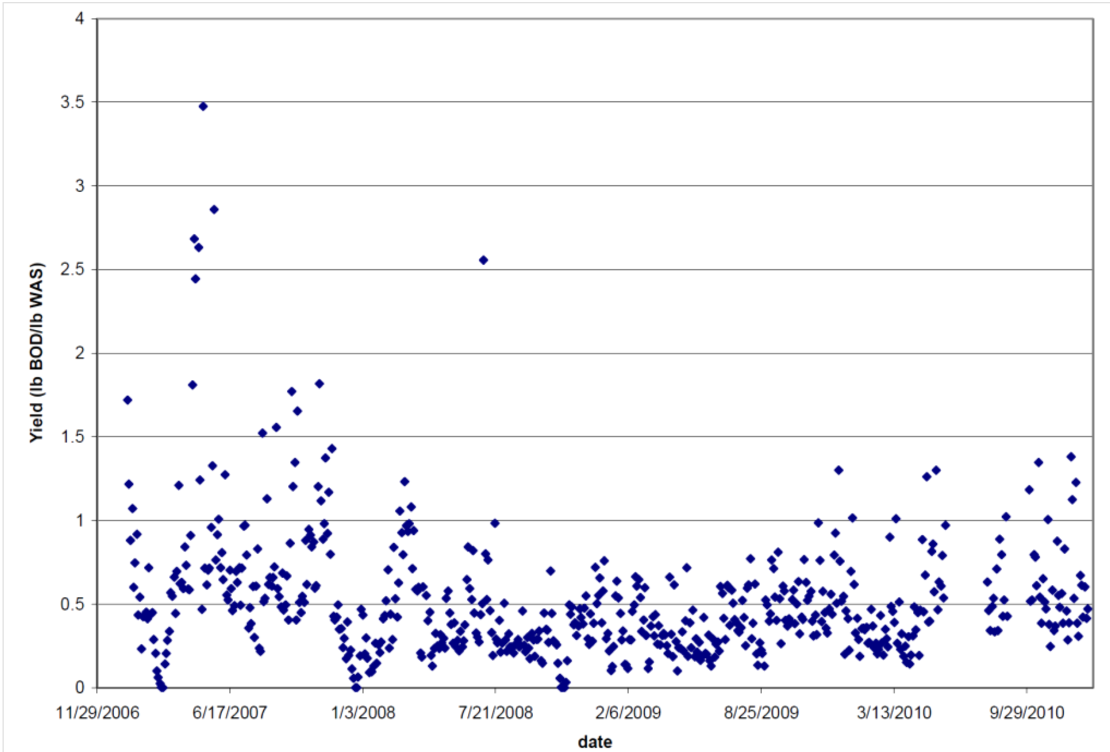
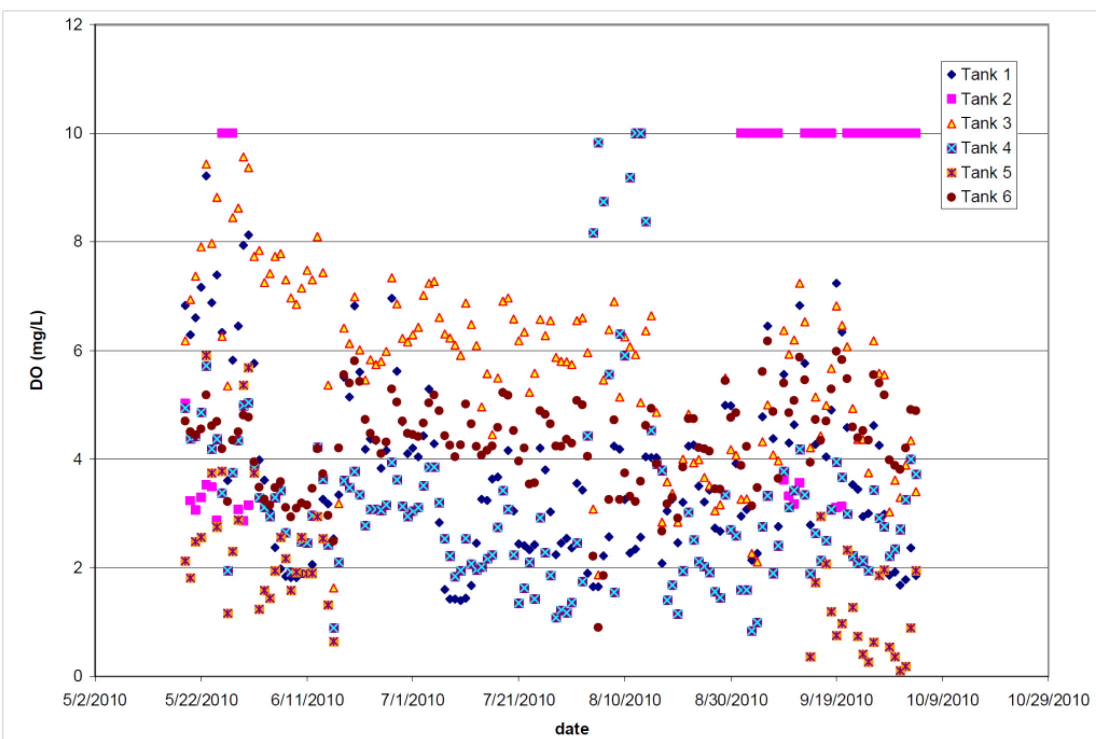


FIGURE 4-8
DISSOLVED OXYGEN LEVELS IN AERATION TANKS



Uneven distribution of airflow poses problems including inefficient energy usage and the potential for incomplete treatment. Over-aeration, particularly as shown in tanks 1, 3, and 6, results in unnecessary use of energy. The dissolved oxygen levels recorded in Tanks 4 and 5 (bulk concentration of <2 mg/L) may result in incomplete nitrification and high ammonia concentrations from these tanks. During the supplemental sampling carried out in October 2010, the ammonia level in the secondary effluent averaged 1 mg/l, and the low DO in these tanks may have contributed to these higher effluent ammonia concentrations. Tanks 4 and 5 are the last ones supplied from the aeration header.

To address these existing aeration distribution issues, a detailed review of the aeration piping and control characteristics will be conducted during preliminary design. It is anticipated that modifications to the aeration piping and control system will provide more uniform delivery of oxygen to all of the aeration tanks. In addition, the new recently installed turbo variable-speed blowers may allow better turndown to match overall aeration demand and improve efficiency. It is important to note that the varying DO levels recorded could also be exacerbated by uneven wastewater flow distribution to the aeration tanks and corresponding variations in oxygen demand. During preliminary design, strategies to address this issue will also be investigated including improvement for flow split as well as aeration control provisions.

The lack of process control of the DO setpoint results in the inability to optimally remove nitrogen using the existing MLE configuration. Currently, the internal nitrate recycle capability is not being used due to the return of high dissolved oxygen concentrations from the aerobic to the anoxic zone. This limits the nitrates returned to the anoxic zone, ultimately reduces the level of denitrification, and thus total nitrogen removal achieved by the MLE Process. It can also affect the performance of the secondary clarifiers. Thus, improved DO control may enable improved or more consistent nitrogen removal performance by the existing MLE process.

4.2.2.4 pH and Alkalinity

As described in Section 2, during the warmer months the plant adds lime to the grit chambers for alkalinity and enhanced solids removal in the primary clarifiers. The plant's operations staff has established a grit effluent pH target of 8.8 to 9.2 as optimal for promoting biological phosphorus removal. The primary effluent pH and alkalinity is 7.0 and 50 mg/L (as $CaCO_3$) during the wintertime without lime addition, respectively. However, the pH has been measured at 9.3 and alkalinity can climb to 350 mg/L (as CaCO3) with lime addition. While this level of alkalinity addition is helpful for nitrification and should not pose a problem to future biological processes being considered, it does have implications on nitrogen removal, as discussed below.

4.2.2.5 Secondary Clarifier Overflow and Loading Rates

Typical average secondary clarifier overflow rates range from 400 to 800 gal/day/sq. ft. The secondary clarifier overflow rates ranged averaged 746 gal/day/sf with a peak of 1,300 gal/day/sf from 2007 to 2010. Figure 4-9 shows surface overflow rates (SOR) versus final effluent TSS, which is measured after the effluent filters (little data exists for secondary effluent TSS). Effluent filters are typically employed from April through October. The final effluent TSS data indicates instances of elevated TSS (>20 mg/L) during high SOR events.

As recommended by TR-1 6, secondary clarifiers following activated sludge systems designed for nitrification should have a design average solids loading rate between 10 to 20 lb/day/sf with a peak of 35 lb/day/sf. The secondary clarifiers are loaded at an average rate of 22 lb/day/sq ft with a peak of 54 lb/day/sq. ft. as shown in Figure 4-10. Thus, the solids loading rate is a bit higher than recommended, and this is the likely cause of the elevated effluent TSS during the high flows that cause high surface overflow rates. If the mixed liquor inventory can be reduced through improved process control, the TSS in the effluent should be reduced.

FIGURE 4-9
SECONDARY CLARIFIER SURFACE OVERFLOW RATE AND PERFORMANCE

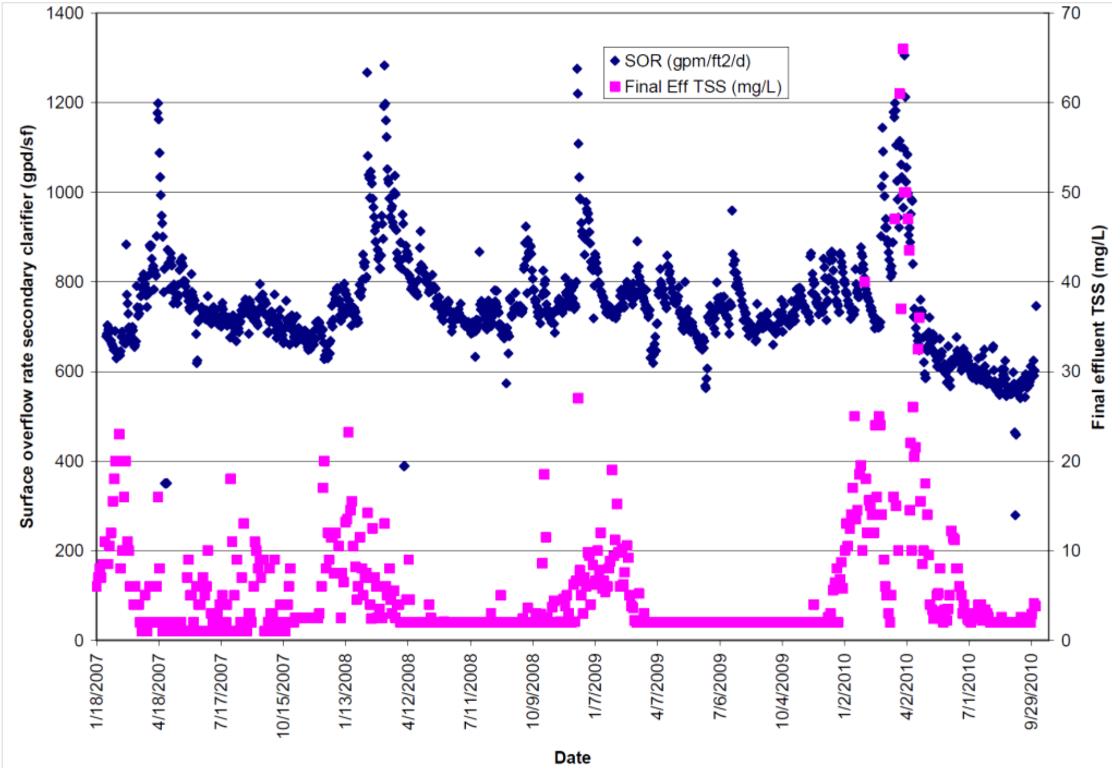
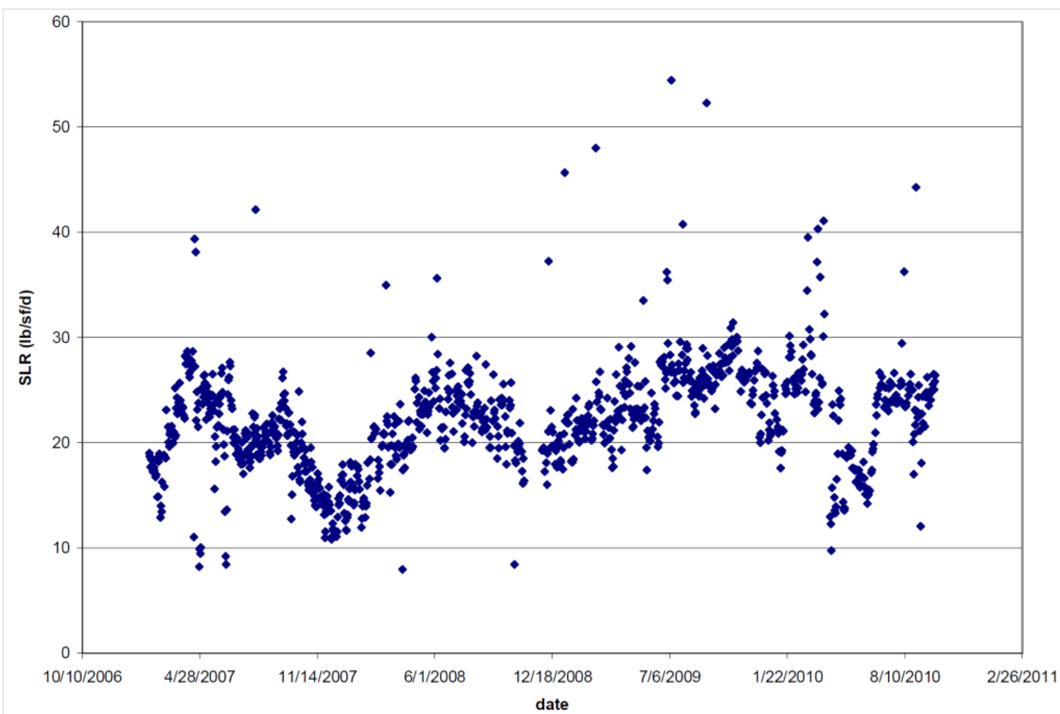


FIGURE 4-10
SECONDARY CLARIFIER SOLIDS LOADING RATE



4.2.2.6 Sludge Volume Index

The ability of the clarifiers to effectively separate solids during higher flows is predicated on several factors, most notably the settleability of the mixed liquor suspended solids. Figure 4-11 shows the historic data on the sludge volume index (SVI), which is an indicator of sludge settleability. The average SVI levels have been very good, but have varied significantly. High values for the SVI indicate periods of poor settleability that can result in elevated effluent TSS levels.

Figure 4-11 also shows the depth of the sludge blanket in the secondary clarifiers, and shows the strong correlation between high blanket levels and high SVI levels. A high blanket level is often an indicator of excessive mixed liquor inventories. It is generally desirable for clarifier blankets to be 2 feet or less, but the typical levels of 2 to 4 feet as shown in Figure 4-11 are common and acceptable levels.

4.2.2.7 Nitrification and Nitrogen Removal

Figure 4-12 shows final effluent ammonia and nitrate/nitrite from January 2007 to December 2010. As shown in Figure 4-12, nitrification is readily achieved during the months of June through October, when the facility has had to comply with reduced ammonia limits. Final effluent ammonia during those months is generally below 1 mg/L with an average of 0.6 mg/L. The ability to achieve low ammonia levels is predicated on several operational factors including solids retention time, dissolved oxygen concentration, pH, and temperature. As shown in Figure 4-4, the WWTF staff process control strategy has maintained MLSS concentrations ranging from 3,000 to 5,500 mg/L during the months of June through October, which has resulted in an SRT that is sufficiently conservative to achieve ammonia removal to below 1.0 mg/l.

Final effluent nitrate/nitrate levels are generally below 8 mg/L with the existing MLE process. This is exceptional performance given the plant does not currently employ the internal nitrate recycle. The return activated sludge does contain nitrates that are denitrified in the anoxic zone. In addition, there may be some simultaneous nitrification/denitrification occurring in aeration tanks. Phosphorus removal is discussed in Section 4.2.2.8 below.

FIGURE 4-11
SVI AND SECONDARY CLARIFIER SLUDGE BLANKET DEPTH

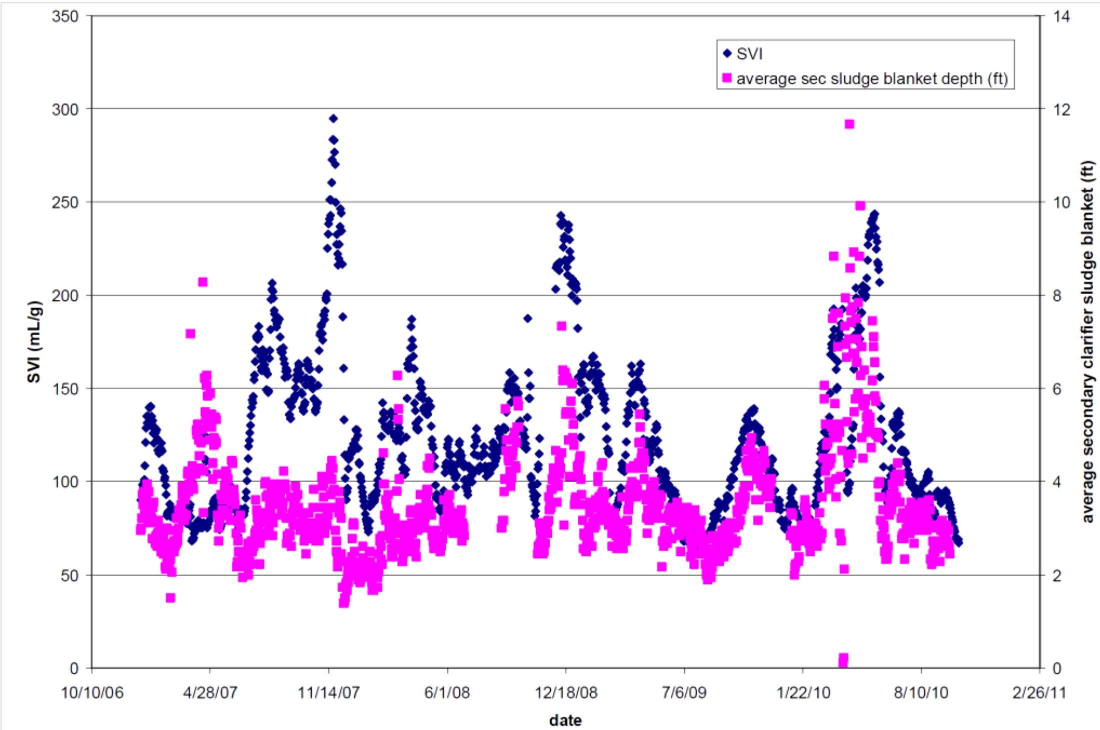
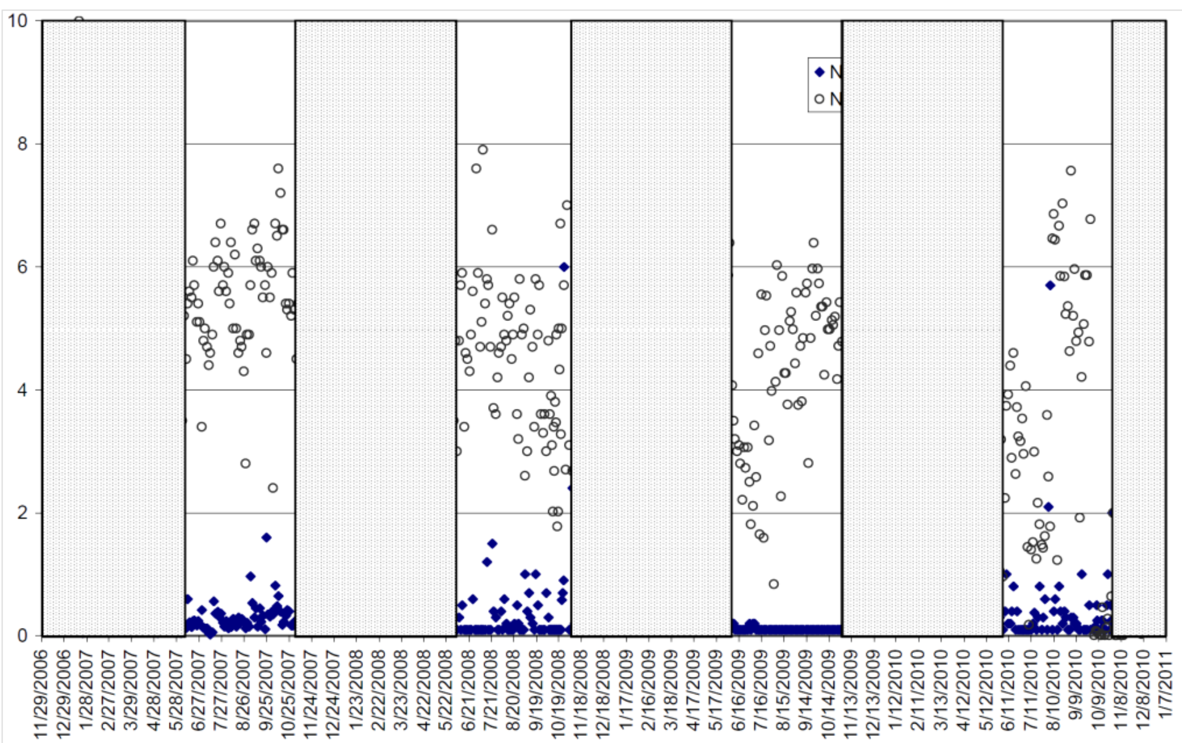


FIGURE 4-12
FINAL EFFLUENT AMMONIA AND NITRATE CONCENTRATIONS



4.2.2.8 Phosphorus Removal

As previously noted, lime is added upstream of the primary clarifiers in order to provide alkalinity and achieve optimum pH levels for biological nitrogen and phosphorus removal as well as to enhance solids removal in the primary clarifiers. Enhanced biological phosphorus removal (EBPR) is occurring in the secondary system, although this is not typically achieved with the MLE configuration. EBPR is facilitated by an anaerobic zone (oxygen and nitrate-free) rich with volatile fatty acids, which encourages soluble phosphorus release by microorganisms. When the anaerobic zone is followed by an aerobic zone, these microorganisms (called phosphate-accumulating organisms, or PAOs) exhibit phosphorus uptake above normal stoichiometric levels required for growth, often resulting in low effluent soluble phosphorus concentrations.

Because the internal nitrate recycle is not being employed with the MLE process, it is possible that an anaerobic zone is formed within the anoxic zone when the nitrates from the return activated sludge flow are depleted. PAOs then uptake the phosphorus in the subsequent aerobic tanks and phosphorus is removed from the wastewater with the waste activated sludge. Final particulate phosphorus removal is accomplished with the tertiary sand filters.
Figure 4-13 shows final effluent total phosphorus levels from April 2007 to October 2010. The seasonal average for final effluent total phosphorus from April to October for these three years is 0.6 mg/L.

4.2.2.9 Effluent Quality and Treatment Performance

Table 4-3 provides a summary of the average final (post-effluent filter) effluent quality for the period of April 2007 to December 2010. The effluent limits for BOD_5, TSS, ammonia and phosphorus are more stringent during the months of June through October. The facility achieves excellent removal of TSS and BOD_5 year-round. Nitrification is strong in the stricter permit limits period with a drop in performance during the winter due to colder temperatures and the lack of lime addition. The majority of routine repair and maintenance performed on equipment required for the seasonal phosphorus removal (i.e. sand filters) is done in the winter months and contributes to the decreased treatment performance. Phosphorus removal drops off during the winter for the same reasons. The total nitrogen and total phosphorus levels are significantly higher than the revised permit limits, but in compliance with the previous permit limits.

FIGURE 4-13
FINAL EFFLUENT TOTAL PHOSPHORUS

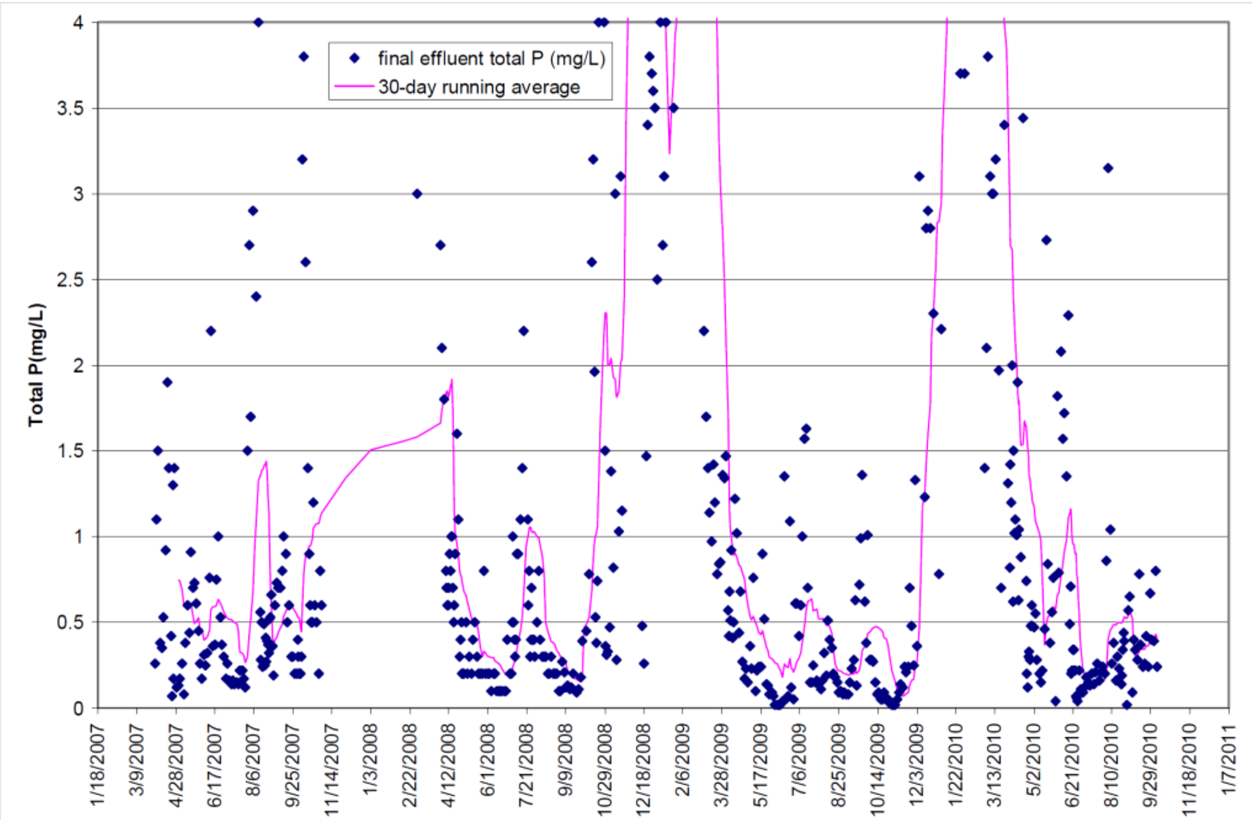


TABLE 4-3
AVERAGE FINAL EFFLUENT QUALITY
MAY 2007 – SEPTEMBER 2010

Parameter	Summer Operating Conditions (April -October)	Winter Operating Conditions (November – March)
TSS, lbs/day (mg/L)	319 (4.0)	760 (8.5)
BOD5, lbs/ day (mg/L)	389 (3.9)	698 (6.5)
Ammonia, lbs/day (mg/L)	61 (0.6)	113 (1.1)
Total Nitrogen, lbs/day (mg/L)	391 (6.2)	461 (6.4)
Total Phosphorus, lbs/day (mg/L)	44 (0.6)	360 (4.5)

4.2.3 Secondary Treatment Performance Conclusions

The following findings and conclusions can be drawn from the analysis of the existing activated sludge system:

1. The facility maintains high MLSS concentrations (an average concentration from 2007 to 2010 of 4,040 mg/L) and long solids residence times (15 to 30 days).
2. Theoretically, the SRT and/or MLSS should be able to be lowered while still maintaining nitrification. This will result in additional aeration tank treatment capacity.
3. Air flow control (distributing oxygen evenly and maintaining a DO setpoint) has been problematic, and should be remedied for optimal nutrient removal. For example, this would allow the use of the internal nitrate recycle line, which should greatly improve nitrogen removal.
4. The secondary clarifiers are close to their theoretical maximum capacity in terms of solids loading rate. Final effluent data indicate elevated TSS (>20 mg/L) during periods of high flow. Reducing the SRT (and correspondingly the MLSS) should alleviate this issue.
5. Lime is added upstream of the primary clarifiers in order to provide alkalinity and enhance pH for biological nitrogen and phosphorus removal as well as to promote solids removal in the primary clarifiers.
6. Enhanced biological phosphorus removal (EBPR) is occurring to some degree in the MLE process.
7. The facility does not use the internal nitrate recycle pumps, and this may be allowing the anaerobic conditions needed for EBPR to occur in the anoxic zone resulting in low secondary effluent phosphorus concentrations.

4.3 PROCESS MODELING

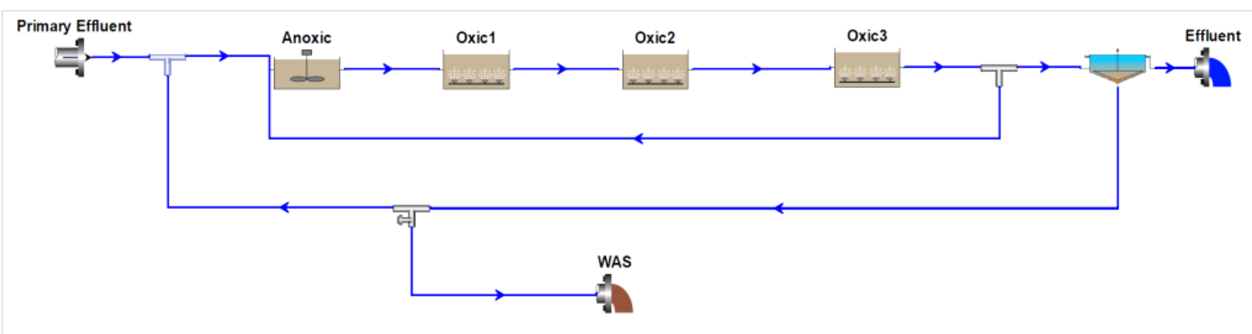
During preparation of this Facility Plan Amendment a biological process model was developed to aid in evaluating nitrogen and phosphorus removal capabilities of the Woonsocket WWTF. This section summarizes development of the model and qualifies the model's ability to predict future conditions.

4.3.1 Model Development

The BioWin® simulator (EnviroSim Associates) is a computer program that simulates biological processes commonly found in municipal wastewater treatment facilities. The program is based upon the *IA WPRC Activated Sludge Model No. 1* (ASM1) modified for biological phosphorus removal through the incorporation of the *Wentzel model.* This combined model is commonly referred to as the *General Model* as it can be applied to a wide range of process configurations, accurately predicting an appropriate balance between five organism masses: the poly-phosphate and non-poly-phosphate heterotrophs, autotrophs, propionic acetogens, and the methanogens. Wright-Pierce used the BioWin® Version 3.1 dynamic computer model to evaluate the current WWTF process.

The WWTF model was developed in steps. A process flow schematic was first generated, and the physical characteristics of the system such as aeration basins' volumes, clarifiers' dimensions, etc. were defined. The process flow schematic utilized for the facility is shown in Figure 4-14. Plant recycle flows including those from solids processing are combined with plant influent flows and loads and are introduced into the model as primary effluent flow.

FIGURE 4-14
BIOWIN PROCESS FLOW SCHEMATIC



Once the physical characteristics were defined, plant operating conditions were developed by specifying flow rates for the plant influent, return activated sludge, and waste activate sludge; defining the composition of the influent streams; and specifying the aeration to the basins. Proper definition of the influent waste characteristics is paramount in achieving realistic predictions from the model simulator.

Supplemental sampling during the three-week period of October 24 to November 11, 2010 resulted in a primary effluent characterization as shown in Table 4-4. The primary effluent contained a high level of volatile fatty acids and readily biodegradable COD. A mass balance on data from the supplemental sampling period indicates that substantial amounts (approximately 30% of volatile fatty acids) are generated within the primary clarifiers.

TABLE 4-4
BIOWIN PRIMARY EFFLUENT CHARACTERISTIC FRACTIONATION

Parameter	Model Default Value	Model Actual Value
Fbs – Readily biodegradable (including Acetate) [gCOD/g of total	0.27	0.414
COD]		
Fac – Acetate [gCOD/g of readily biodegradable COD]	0.15	0.22
Fxsp – Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0. 5	0.6
Fus – Unbiodegradable soluble [gCOD/g of total COD]	0.08	0.06
Fup – Unbiodegradable particulate [gCOD/g of total COD]	0.08	0.25
Fna – Ammonia [gNH3-N/gTKN]	0.75	0.428
Fnox – Particulate organic nitrogen [gN/g Organic N]	0.25	0.25
Fnus – Soluble unbiodegradable TKN [gN/gTKN]	0.02	0.02
FupN – N:COD ratio for unbiodegradable part. COD [gN/Gcod]	0.035	0.035
Fpo4 – Phosphate [gPO4-P/gTP]	0.75	0.6
FupP – P:COD ratio for unbiodegradable part. COD [gP/gCOD]	0.011	0.011

It is important to highlight several operational considerations, simplifications, and assumptions garnered from the operational review and incorporated into the model development:

- The biological process model was developed to evaluate secondary treatment process, using historic primary effluent data as the input to the system. This was done to avoid the necessity of modeling the impact of primary clarification, since actual historic primary effluent data was available.

- As previously noted, the plant currently operates in the Modified Ludzack-Ettinger configuration for biological nitrogen removal without internal recycle. Reportedly, this is due to problems in returning high dissolved oxygen to the anoxic zone which adversely impacts the denitrification process. Also, only one anoxic zone is used for both warm weather and cold weather conditions.

Mixed liquor suspended solids have been measured recently up to 5,000 mg/L, ranging from 78 to 87 percent volatile content.

It should be noted that phosphorus removal to low effluent concentrations (<1 mg/L) is dependent on physical/chemical removal of effluent solids, which depends on secondary clarifier performance. Although it can be simulated from historical performance, detailed fluid dynamic modeling of clarifier solids removal performance was not conducted. However, for each operational scenario modeled, historical solids removal performance combined with an appropriate safety factor was utilized to evaluate the impacts of the secondary clarifier suspended solids load to the future tertiary physical/chemical removal system.

4.3.2 Model Calibration

Actual plant operational data (including supplemental sampling performed for this study) during the three-week time period of October 24 to November 11, 2010 was used for the initial model calibration. This short calibration period was used to incorporate the additional data from the sampling period. However, it should be noted that a total of nine data points (each) were collected for the primary effluent characteristic COD, TKN, ammonia, orthophosphate, and total phosphorus. Although averaging the results over this three week period can be sufficient for a baseline model calibration if the system is relatively stable, the model output is susceptible to anomalous values measured during the period. Figure 4-15 presents the variability of primary clarifier effluent TKN concentrations during the supplemental sampling period. The ramifications of the high variability will be discussed below.

Several adjustments to the BioWin default kinetic and stoichiometric parameters were made in order to calibrate the process model. The key calibration parameter was to verify that the model accurately predicts the MLSS concentration and the amount of sludge produced. Once the model accurately reflects the mass generated by the process, nitrogen and phosphorus performance was analyzed. It should be noted that the plant has historically measured effluent nutrient data, but not influent nutrient data.

The model accurately simulated the MLSS concentrations and secondary sludge wasting rates observed during the calibration period. To accurately predict the MLSS and WAS values, the following parameter adjustments were made to the model as shown in Table 4-5:

- The stoichiometric aerobic yield of heterotrophs was increased;
- The fraction of ordinary heterotrophic organisms (OHO) to endogenous residue was increased;
- The kinetic parameters of heterotrophic anoxic and aerobic decay rates were decreased.

FIGURE 4-15
PRIMARY EFFLUENT TKN AND NH3 DURING SUPPLEMENTAL SAMPLING

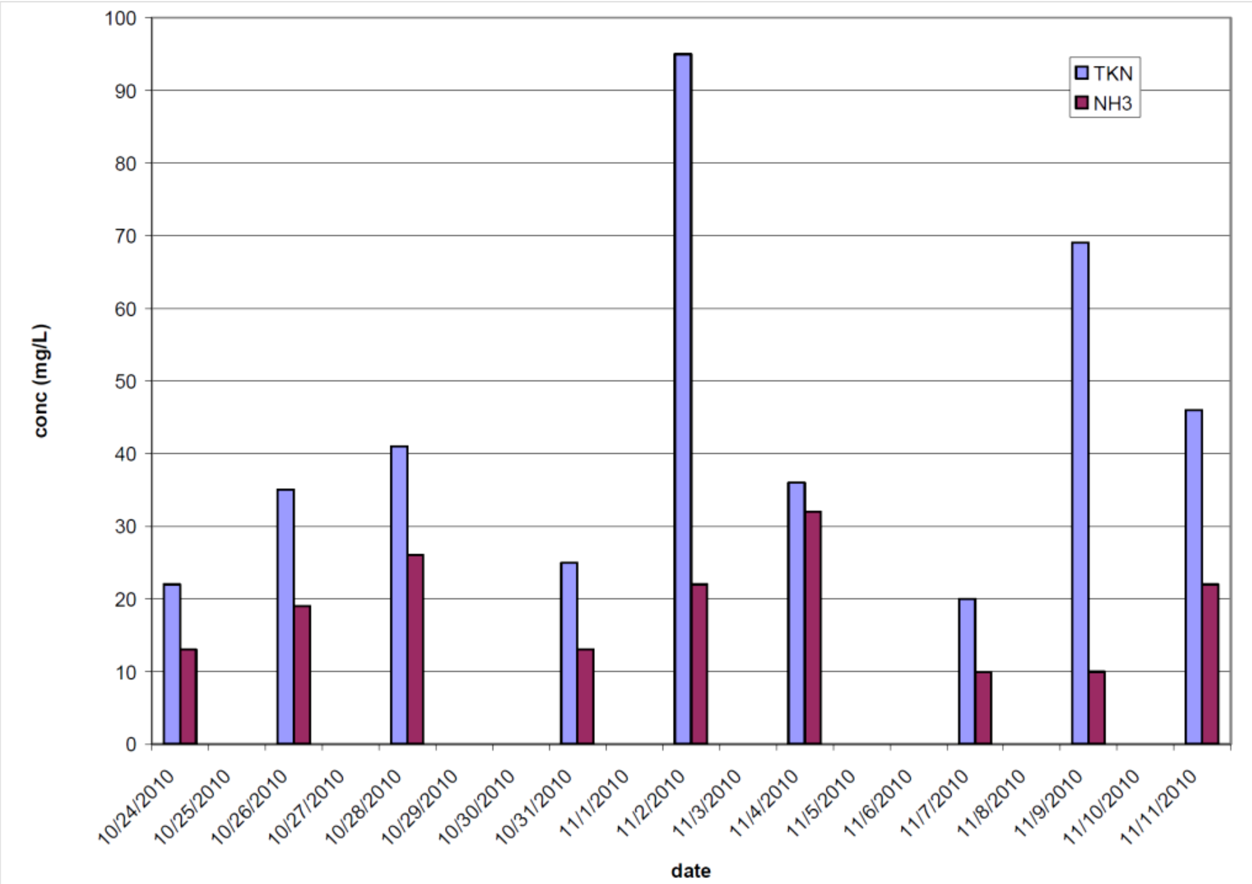


TABLE 4-5
ADJUSTED BIOWIN PARAMETERS USED IN THE BIOWIN MODEL

	Default	Adjusted
Stoichiometric Parameters		
Common		
Particulate Substrate COD:VSS Ration [mg COD/mg VSS]	1.6	1.8
Particulate Inert COD:VSS Ration [mg COD/mg VSS]	1.6	1.8
OHOs		
OHOs Yield (Aerobic) [-]	0.666	0.725
Fraction to Endogenous Residue [-]	0.08	0.12
Kinetic Parameters		
OHOs		
OHO Aerobic Decay[1/d]	0.62	0.5
OHO Anoxic/Anaerobic Decay [1/d]	0.3	0.25

While the calibrated model was able to accurately predict the MLSS concentrations and secondary sludge wasting rates, the initial model was unable to accurately predict the observed nitrogen removal performance of the process. This is potentially due to the limited available historic influent nitrogen data and the highly variable influent nitrogen data recorded during the supplemental sampling period, which may have resulted in a "weighted average" influent nitrogen value that is not truly reflective of the actual influent nitrogen level. Three alternative approaches were developed to update the "initial" model to simulate the nitrogen removal performance observed at the Woonsocket WWTF:

- **Approach 1:** This approach can be considered the "default" option. Calibration of the process model was conducted without adjustment to the primary effluent TKN values, nitrogen fractionation parameters, or the performance of the nitrogen removing bacteria.

- **Approach 2:** This approach consisted of eliminating the high influent TKN values from the model set. Limited TKN measurements were conducted during the supplemental sampling period. Several (3-4) samples could potentially inaccurately skew the average of the TKN measurements. Eliminating those measurements adjusted the primary effluent average TKN from 43.2 mg/L to 27.6 mg/L. It also should be noted that several of the primary effluent TKN values measured were greater than both the raw and plant drain TKN values, suggesting some anomalies in the data.

TABLE 4-6
BIOWIN MODEL CALIBRATION RESULTS USING 3 DIFFERENT APPROACHES

Condition	Plant Data	Model Output Approach 1	Model Output Approach 2	Model Output Approach 3
Primary Clarifiers Effluent				
BOD, mg/L	238	206	206	206
COD, mg/L	424	450	450	450
TSS, mg/L	116	120	120	120
VSS, mg/L	104	104	104	104
TKN, mg/L	43.2	43.2	27.6	43.2
NH3, mg/L	18.5	18.5	18.5	18.5
NOx, mg/L	1.4	1.4	1.4	1.4
P, mg/L	3.9	3.9	3.9	3.9
Temp, C	30.0	30.0	30.0	30.0
Flow Rate, mgd	10.21	10.21	10.21	10.21
Aeration Tanks				
No. of Tanks	6	6	6	6
Total Volume, Mgal	5.69	5.69	5.69	5.69
Total Volume, Anoxic, Mgal	1.42	1.42	1.42	1.42
Total Volume, Oxic, Mgal	4.27	4.27	4.27	4.27
MLVSS, Oxic Zone, mg/L	N/A	4,148	4,162	4,165
MLSS, OxicZone, mg/L	5,184	4,934	4,948	4,951
Internal Recycle Rate, mgd	0.0	0.0	0.0	0.0
SRT (oxic), days	14.0	14.3	14.3	14.3
SRT (total), day	18.0	19.1	19.1	19.1
Secondary Clarifier				
Effluent BOD5, mg/l	2	1	1	1
Effluent TKN, mg/l	2	2	2	3
Effluent NH3, mg/l	1	1	1	1
Effluent NOx, mg/l	6	17	8	7
Effluent TN, mg/l	9	20	10	10
Effluent P, mg/l	<1	<1	<1	<1
Effluent TSS, mg/l	5	7	7	7
Waste Activated Sludge				
Waste Activated Sludge, lb/d	13,544	12,075	12,111	12,119
Sludge Yield, lbs TSS/lb BOD5	0.67	0.69	0.69	0.69
Sludge Yield, lbs VSS/lb BOD5	N/A	0.58	0.58	0.58

- **Approach 3:** Because of the high volume of solids handling recycle and/or characteristics of the influent nitrogen, it can be assumed that a higher than normal level of the influent TKN may be particulate in nature and thus is removed through settling versus nitrification/denitrification process. The primary effluent nitrogen fractionation parameter can be adjusted in the model (fraction of non-biodegradable particulate nitrogen, or FupN) to increase the amount of influent organic nitrogen that is non-biodegradable and particulate; essentially calibrating the model such that all of the primary effluent nitrogen is organic. The supplemental sampling data supports the approach since the data does indicate that during periods of high influent TKN loading; the corresponding ammonia load did not increase proportionally, indicating that the additional nitrogen load was in the form of organic nitrogen. The supplemental sampling results also indicated that the nitrogen in the plant drain system is predominantly organic nitrogen. This likely because nitrogen released during cell decay (i.e. as would occur in a gravity thickener or similar sludge holding tank) would be in the form of non-biodegradable (or extremely slow degradation) particulate organic nitrogen.

Other possible mechanisms of "non-modeled" nitrogen removal include:

- Denitrification in the secondary clarifiers – High nitrate levels and anoxic conditions found in secondary clarifiers provide conditions for denitrification. Significant denitrification in the clarifiers, however, would require extensive hydraulic retention times or deep sludge blankets, and often is accompanied by foaming of secondary sludge, which has not been observed at Woonsocket.

- Volatilization of ammonia – Primary effluent pH levels at Woonsocket can approach 8.5 to 9 due to addition of lime. At high pH levels, a greater proportion of ammonia is found in the un-ionized form, which is considerably more volatile than the ammonium ion NH4 + predominant at neutral pH. The ammonia may be removed with the aeration air, resulting in less nitrification to nitrate occurring in the aeration tanks. This mechanism would be difficult to quantify without significant sampling.

- Simultaneous nitrification/denitrification – Numerous studies have shown the processes to occur simultaneously where bulk dissolved oxygen concentrations are low, as has been recorded in Aeration tanks 4 and 5 (Jimenez, et al, WEFTEC 2010).

At this time a definitive conclusion on which model is more accurate cannot be determined. However, it is likely that some combination of lower influent TKN and a higher fraction of non-biodegradable organic nitrogen should be included in the final process model. Data was not collected after the initial supplemental sampling efforts during October to November 2010 because of cold-weather (and therefore no nitrogen removal) operating conditions. However, during subsequent preliminary design, it is recommended that additional data collection be performed to further analyze current nitrogen removal performance. Refinement to the model will be used to further define design operating conditions to meet effluent limits. Refinement of the process model will have little impact on the proposed technology selection and associated capital cost.

4.3.3 Model Verification

The key calibration parameter was to verify that the model accurately predicts the MLSS concentration and the amount of sludge produced. In order to verify the calibrated model, a separate influent condition was modeled. Plant operational data from an 8-week period of July to August 2009 was used for the verification phase, representing warm weather conditions. If the original model is able to accurately predict the historical plant performance for a verification period, it is then assumed that the model is fully calibrated.

The results of the verification phase are presented in Table 4-7. As shown in the Table, the process model was able to accurately predict the mixed liquor concentrations and waste activated sludge quantities.

Regarding nitrogen removal, it was necessary to verify the various approaches used during calibration. In Table 4-7, Approach 1 presents verification results without any adjustments for nitrogen removal. As shown in Table 4-7, this approach did not sufficiently simulate nitrogen removal at the plant. Approach 2, eliminating anomalous values, was not employed because the primary effluent TKN was relatively consistent during the sampling period, ranging from 15 to 24 mg/L and an average of 18 mg/L TKN. Approach 3 presents the results of increasing the primary effluent nitrogen fractionation parameter (fraction of non-biodegradable particulate nitrogen, or FupN) to increase the amount of influent organic nitrogen that was non biodegradable and particulate, essentially calibrating the model such that all of the primary effluent nitrogen is organic. The verification model used for Approach 3 was able to match both biological yield and nitrogen removal observed for the period as shown in Table 4-7.

TABLE 4-7
BIOWIN MODEL VERIFICATION RESULTS
(JULY 28 TO AUGUST 26, 2009)

Condition	Verification Data (July-Aug 2009)		
	Plant Data	Approach 1	Approach 3
Primary Effluent			
BOD, mg/l	120	120	120
TSS, mg/l	45	45	45
VSS, mg/L	-	40	40
TKN, mg/L	18.3	18.3	18.3
NH3, mg/L	10.1	10.1	10.1
NOx, mg/l	0.0	0.00	0.00
P, mg/L	5.3	5.3	5.3
Ortho P, mg/l	1.5	1.5	1.5
pH	8.5	8.5	8.5
Temp, C	27.3	27.3	27.3
Flow Rate, mgd	11.1	11.1	11.1
Secondary System			
MLSS, OxicZone, mg/L	4,459	4,815	4,922
Effluent CBOD5, mg/l	2	1.62	1.62
Effluent COD, mg/l		22.6	23.1
Effluent TKN, mg/l	0.97	1.0	1.5
Effluent NH3, mg/l	0.1	0.1	0.1
Effluent NOx, mg/l	4.2	6.9	4.4
Effluent TN, mg/l	5.28	8.0	6.1
Effluent P, mg/l	0.36	1.5	0.9
Effluent TSS, mg/l	2	7.32	7.32
Waste Activated Sludge			
Flow rate, mgd	0.05	0.05	0.05
TSS, mg/L	10,452	10,224	10,451
Waste Activated Sludge, lb/d	4,358	4,263	4,358
Sludge Yield, lbs TSS/lb BOD5	0.39	0.38	0.39
Sludge Yield, lbs VSS/lb BOD5	-	0.23	0.23

*WAS concentration assumed as 130% of RAS concentration (mg/L)

4.3.4 Conclusions

The calibration and verification efforts suggest that a higher degree of nitrogen is removed at the Woonsocket WWTF than can be explained using the biological process model and existing readily available data. The disagreement may be due to a number of considerations outlined above, none of which can be confirmed at this point. However, during subsequent preliminary design, it is recommended that additional data collection modeling be performed to further analyze current nitrogen removal performance. Refinement to the model will be used to further define design operating conditions to meet effluent limits. Refinement of the process model will have little impact on the proposed technology selection and associated capital cost.

The biological process model can be used to determine the future MLSS concentration, subsequent loadings on the secondary clarifiers, nitrogen removal performance, and mass balance of the activated sludge system at the future design flows and loads. As a standard design practice, additional conservatism should be considered with respect to the effluent nitrogen levels leaving the secondary treatment process (for sizing of a tertiary denitrification process) due to conditions of process upset or equipment maintenance.

Additional conclusions from the preliminary biological process model development/calibration include:

- Effluent organic nitrogen ranged from 1.0 mg/L during the verification period of July-August 2009 to 1.41 mg/L during the calibration period of October-November 2010. Recalcitrant dissolved organic nitrogen fraction (soluble non-biodegradable TKN/total TKN) of 0.015 in the primary effluent was selected to simulate the observed effluent organic nitrogen numbers.

- Calibration indicated that the variation of bulk dissolved oxygen observed throughout the aeration tanks (specifically lower DO (1 mg/L) observed in tanks 4 and 5) does not affect nitrogen removal. However, this does not eliminate the possibility of simultaneous nitrification/denitrification occurring in local anoxic areas where the dissolved concentration may be lower than the bulk concentration measured, which would not be captured by the model.

- Total phosphorus effluent levels simulated for the calibration period was less than 1 mg/L, which is similar to observed results. The model simulation confirmed the establishment of an anaerobic zone within the anoxic zone, and enhanced biological phosphorus removal occurring within the subsequent aerobic zones. The model was not further calibrated to improve agreement of simulated versus observed effluent phosphorus, as the results simulated are considered sufficient for estimating loads for a tertiary physical/chemical removal system.

SECTION 4.A
PROCESS MODEL DEVELOPMENT FOR PLANNED CAPITAL IMPROVEMENTS

4.A.1 INTRODUCTION

This section 4.A, Process Model Development for Planned Capital Improvements, presents the planned capital improvement process configuration and modeling developed for preliminary design.

A two-stage activated sludge process was proposed for the planned capital improvements. Further collection and analysis of plant data resulted in revised influent load criteria and characterization. The proposal design was updated in response to the data collection and the two-stage activated sludge AB secondary treatment process was agreed upon.

4.A.2 MODEL CONTAMINANT LOAD BASIS

The design flows and loads are described in Chapter 3. A summary of those results for average and maximum month conditions are given in Table 4.A-1.

To optimize the planned capital improvements, data was collected for recent plant operations and loading. This data was used to verify and/or revise the March 2011 Draft Facility Plan loading criteria. It was also used to characterize the influent wastewater quality.

TABLE 4.A-1
DESIGN CONTAMINANT LOAD BASIS

Parameter	Units	Raw Influent	Recycle	Primary Influent (PI)
Flow				
Annual Average	MGD	9.0	3.9	12.9
Maximum 30-day Average	MGD	16.0	5.1	21.1
Five Day Biological Oxygen Demand (BOD_5)				
Annual Average	lbs/d	19,715	9,028	28,742
Maximum 30-day Average	lbs/d	31,170	19,162	38,895
Total Suspended Solids				
Annual Average	lbs/d	13,340	13,218	26,558
Maximum 30-day Average	lbs/d	23,158	19,740	40,985

TABLE 4.A-1
DESIGN CONTAMINANT LOAD BASIS

Parameter	Units	Raw Influent	Recycle	Primary Influent (PI)
Total Kjeldal Nitrogen (TKN)				
Annual Average	lbs N/d	2,070	1,230	3,300
Maximum 30-day Average	lbs N/d	2,612	1,649	3,768
Ammonia				
Annual Average	lbs N/d	1,320	360	1,680
Maximum 30-day Average	lbs N/d	1,642	429	1,846
Total Phosphorus (TP)				
Annual Average	lbs P/d	270	72	342
Maximum 30-day Average	lbs P/d	314	107	454

Notes: From Chapter 3

An examination of Table 4.A-1 shows that for the maximum month condition, the primary influent loads do not equal the sum of the maximum month raw and recycles numbers. This is a direct result of the fact that the peaks in the two separate streams do not normally happen at the same time. The plant operating data shows this same pattern.

For modeling purposes, the most important criteria are the primary influent flows and loads, at the maximum month loading conditions. Therefore, to be able to model the maximum month primary influent number, it is necessary to develop raw sewage and recycle loads that do add up to the design primary influent loads. This is done by proportioning the maximum month (MM) PI load according to the fractions seen at the average condition, i.e.:

- Model Basis Raw MM Load = Design PI MM Load * Design Raw Average Load / Design PI Average Load
- Model Basis Recycle MM Load = Design PI MM Load – Model Basis Raw MM Load

Table 4.A-2 compares the resulting model basis loads against the design loads for maximum month conditions. The modeling loads for average conditions are shown in Table 4.A-2.

TABLE 4.A-2
COMPARISON OF DESIGN VERSUS MODELING BASIS MAXIMUM MONTH LOADS

Parameter	Units	Raw Influent		Recycle		Primary Influent (PI)	
		Design	Model	Design	Model	Design	Model
BOD_5	lbs/d	31,170	26,678	19,162	12,216	38,895	
TSS	lbs/d	23,158	20,587	19,740	20,398	40,985	
TKN	lbs N/d	2,612	2,363	1,649	1,404	3,768	
Ammonia	lbs N/d	1,642	1,450	429	396	1,846	
TP	lbs P/d	314	358	107	95	454	

Notes:

Model PI = Model Raw + Model Recycle
Design loads from Chapter 3

4.A.2.1 Phosphorus Loads

Historical data on the recycle phosphorus loads is not applicable to the proposed process. This is a result of the impact of biological phosphorus removal (BPR) occurring in the plant and it's interaction with Synagro. Current operation allows BPR to remove phosphorous from the wastewater and store it in the biomass sent to Synagro. At the Synagro facility, this phosphorus laden sludge is stored/handled under anaerobic (i.e. unaerated) conditions. Under anaerobic conditions, the biomass that stored the phosphorus in the main liquids train releases that phosphorus back into solution, which is then returned to the plant after dewatering. This released phosphorus is then taken back up in the liquids process, returned to Synagro, and it cycles up once again.

This cycling up of phosphorus between the main plant and Synagro results in a very high phosphorus return load in the historical data set. The proposed process breaks this cycling-up loop through the addition of ferric chloride to both the primary clarifier and to the second stage sludge system. This ferric chloride has significant capacity to absorb additional phosphorus, and does so when biomass releases its stored phosphorus within the Synagro system. This essentially breaks the cycling up of phosphorus loop in the plant and therefore greatly reduces the amount of phosphorus in the recycle stream.

To deal with this issue in the process model, a whole plant (including Synagro) phosphorus mass balance was done to determine what the phosphorus content of the imported Synagro sludge must be. It was determined that the Synagro sludge contains approximately 0.007 lbs of phosphorus/lb VSS. This is very comparable to the industry standard value of 0.01 lbs phosphorus/lb VSS, and so is believed to be a reliable number. In the model, the 0.007 lbs P/lb VSS was used in characterizing the sludges, and the model predicted phosphorus levels were therefore used as the basis of design. The model results show the loads indicated in Table 4.A-2 for average conditions, but under maximum month conditions holding the 0.007 number constant results in higher recycle phosphorus loads than shown in Table 4.A-2. Rather than using a different method to determine maximum month loads, or adjust the 0.007 value downward in the maximum month runs, it was decided to just use the model values as predicted, which adds a level of conservatism in the maximum month models.

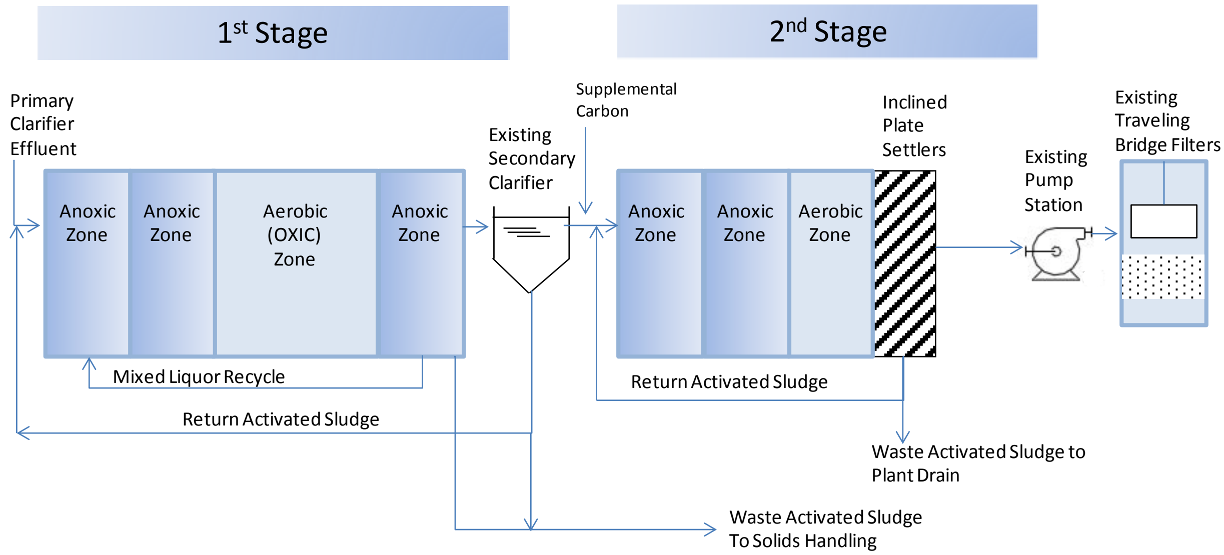
4.A.3 PROCESS DESCRIPTION

The planned capital improvements modify the existing plant configuration to meet the new effluent requirements at design PI flows and loads. A two-stage activated sludge process was originally proposed to meet this criteria, but after further data analysis the two-stage process was enhanced to include nitrification in the second stage sludge. This enhancement is a variation of what is known as the AB process. The proposal two-stage activated sludge process configuration and the two-stage activated sludge AB process configuration differ only in the presence of nitrification in the second stage sludge. Both designs use the existing primary treatment, tertiary filtration, and disinfection processes.

4.A.3.1 Proposal Basis of Design

The proposal design uses a two-stage biological process for secondary treatment. The first stage uses the existing aeration basins to provide nitrification, denitrification and BOD removal. This requires a long SRT to grow nitrifiers and a relatively high MLSS. The second stage has a separate biomass for denitrifying the mixed liquor with methanol as a carbon source. The existing secondary clarifiers are used for the first stage and new lamella plate settlers are used for the second stage. This process layout is shown in Figure 4A-1.

FIGURE 4.A-1
SECONDARY SYSTEM PROCESS FLOW DIAGRAM FOR ORIGINALLY PROPOSED TWO-STAGE ACTIVATED SLUDGE PROCESS



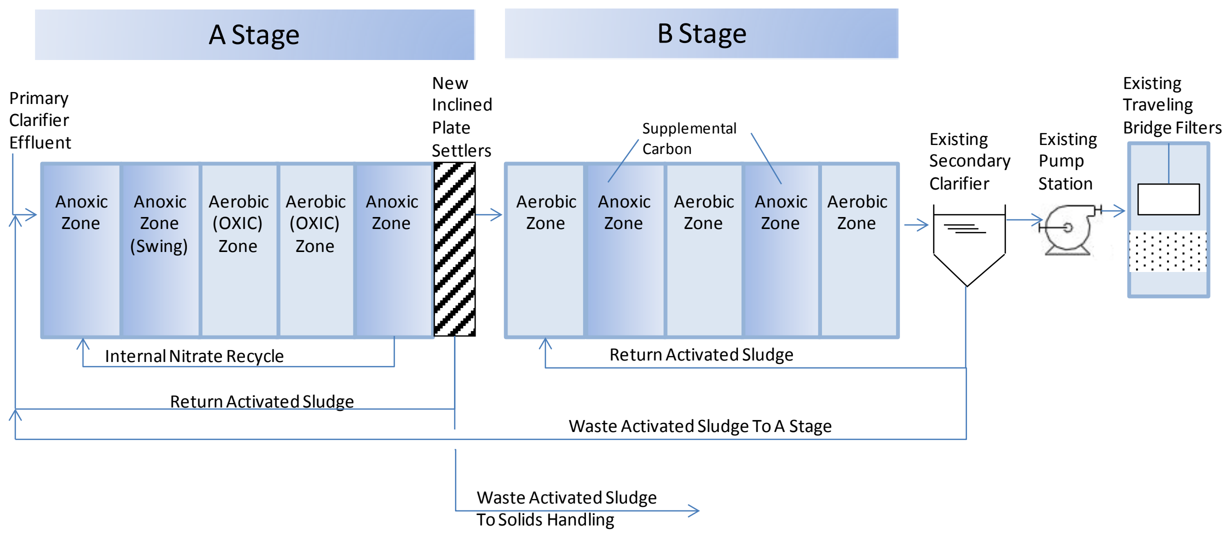
4.A.3.2 Two-Stage Activated Sludge AB Process Configuration Basis of Design

The AB process is also a secondary treatment configuration with two stages; the A Stage and the B Stage. The AB process differs from the proposal design in the strategy for achieving nitrification/denitrification in these stages. The A Stage is relatively small because it has a short aerobic SRT of about 4 days or less. Nitrifiers cannot be maintained at this SRT, so the A Stage is used primarily for BOD removal and some nitrification/denitrification as described below. The B stage has an aerobic SRT sufficiently long for nitrifier growth and it is configured for nitrification and denitrification with methanol addition. WAS, containing nitrifiers, from the B Stage is used to seed the A Stage, allowing some nitrification and denitrification despite the low SRT.

The AB process configuration requires a smaller bioreactor volume than the proposal configuration because the majority of the biomass (i.e. the heterotrophs grown on carbon) can be maintained at a much shorter SRT (<4 days versus 10 days). The AB process also provides more treatment capacity within a given bioreactor volume, which is of significant benefit to the City of Woonsocket. Lastly the inclusion of nitrification in the second stage (in addition to the first) provides another level of redundancy that improves overall process reliability.

The benefits of reduced bioreactor volume, increased capacity, and increased reliability resulted in the two-stage activated sludge AB process configuration being selected for the planned capital improvements. Figure 4A-2 shows the process flow diagram for the secondary system as revised with the AB process.

**FIGURE 4.A-2
SECONDARY SYSTEM PROCESS FLOW DIAGRAM FOR TWO-STAGE ACTIVATED SLUDGE AB PROCESS**



4.A.4 PROCESS MODELING

A process model was developed for the planned capital improvements to assist with the design of the process. The model uses Pro2D™, a wastewater plant simulation program that calculates complete mass balances using International Water Association based models including Activated Sludge Model 2d (ASM2d). The program calculates the mass balance for over 70 different wastewater components and performance of the various treatment flow streams, based on plant influent flows and loads, treatment plant processes and configuration, operational criteria, and chemical dosages.

The revised annual average loading conditions are used to predict typical performance and operational requirements. The revised maximum month loading conditions, summarized in Table 4.A-2, are used to determine the size and critical operating conditions for the new biological treatment process and other existing processes. Projected maximum week conditions are used as the basis for aeration requirements.

4.A.4.1 Overview of Model Configuration

The process model is a whole plant model (Figure 4.A-3) including chemically enhanced primary treatment (CEPT), secondary treatment (two-stage activated sludge AB process), and tertiary filtration. The model also includes Synagro solids treatment facilities to account for recycles to the WWTP. In addition, the model includes a degree of fermentation in the gravity thickener which matches reported values in plant recycle. Table 4.A-3 summarizes basic criteria of the liquids treatment processes.

TABLE 4.A-3
WHOLE PLANT – MODEL BASIS OF DESIGN

Parameter	Units	Value
Primary Treatment		
Clarifier, number	-	2
Diameter	ft	90
TSS Removal Efficiency	%	70%
Primary Sludge Concentration	mg/L	10,000
Secondary Treatment – Stage A		
Basins, number	-	3
Mixed Liquor Recycle Rate		700%
Secondary Treatment – Stage A Settlers		
Settlers, number		3
Settling area, each	sf	15,012
Return Activated Sludge Rate		50%
Effluent TSS	mg/L	20
Secondary Treatment – Stage B		
Basins, number		2
Mixed Liquor Recycle Rate		0%
Secondary Treatment – Stage B Clarifiers		
Clarifiers, number		3
Diameter, each	ft	110
Return Activated Sludge Rate		50%
Effluent TSS	mg/L	8

TABLE 4.A-3
WHOLE PLANT – MODEL BASIS OF DESIGN

Parameter	Units	Value
Tertiary Filtration		
Filters, number		4
Surface area, each	sf	1,384
TSS Removal Efficiency	%	60%

4.A.4.1.1. Primary Treatment

The model uses the existing primary clarifiers for chemically enhanced primary treatment (CEPT). Ferric chloride addition of at least 5 mg/L is used to control odor, remove phosphorus, and aid in settling. A TSS removal efficiency of 70% is assumed based on past performance.

4.A.4.1.2 Secondary Treatment – A Stage

Three A Stage basins are configured similar to an MLE process with a pre-anoxic zone followed by a series of aerobic zones. A mixed liquor recycle sends nitrate to the anoxic zone for denitrification. A small anoxic zone at the end of the basin limits the dissolve oxygen recycled to the anoxic zone. Overall basin sizes are summarized in Table 4.A-4.

TABLE 4.A-4
TWO-STAGE ACTIVATED SLUDGE AB PROCESS – BASIS OF DESIGN

	Zone (number)	Type	Stage Volume in all Tanks (gallons)	Depth (feet)
A Stage				
	1	Anoxic	768,849	16.5
	2	Swing (Anoxic or Aerobic)	768,849	16.5
	3	Aerobic	767,903	16.5
	4	Aerobic	767,903	16.5
	5	Anoxic	313,495	16.5

TABLE 4.A-4
TWO-STAGE ACTIVATED SLUDGE AB PROCESS – BASIS OF DESIGN

	Zone (number)	Type	Stage Volume in all Tanks (gallons)	Depth (feet)
B Stage				
	1	Aerobic	465,396	15
	2	Anoxic	465,396	15
	3	Aerobic	252,933	15
	4	Anoxic	536,217	15
	5	Aerobic	180,058	15

Note: All aerobic zones are design for up to 2 mg/L dissolved oxygen levels at loading rates up to maximum week loads.

4.A.4.1.3 A-Stage Secondary Plate Settlers

The A-Stage settlers are inclined plate settlers installed at the end of each A-Stage basin. From a modeling perspective the projected plate area is used as the basis for settling, and it is assumed that the effluent TSS would average approximately 20 mg/L. This higher than normal effluent TSS is chosen based upon slightly lower TSS removal performance expected from the plate settler system. RAS rates are set at 50% of the influent flow rate for purposes of modeling.

4.A.4.1.4 Secondary Treatment – B Stage

The B-Stage bioreactor system consists of two bioreactors in parallel. It does not include any recycle streams, other than recycled activated sludge (RAS) from the clarifier system. It is designed to allow the introduction of a portion of the plant recycles stream into the first anoxic zone, and for supplemental carbon addition in the second anoxic zone. Ferric Chloride is added to the end of these basins as needed to maintain the targeted effluent phosphorus goals.

4.A.4.1.5 B-Stage Secondary Clarifiers

The existing secondary clarifiers are used for the B-Stage clarification. Effluent TSS from these clarifiers is assumed to average 8 mg/L, and they were operated at 50% RAS rate.

4.A.4.1.6 Tertiary Filters

The existing filters are modeled assuming a 60% TSS removal, and backwash production of approximately 10% of the influent volume.

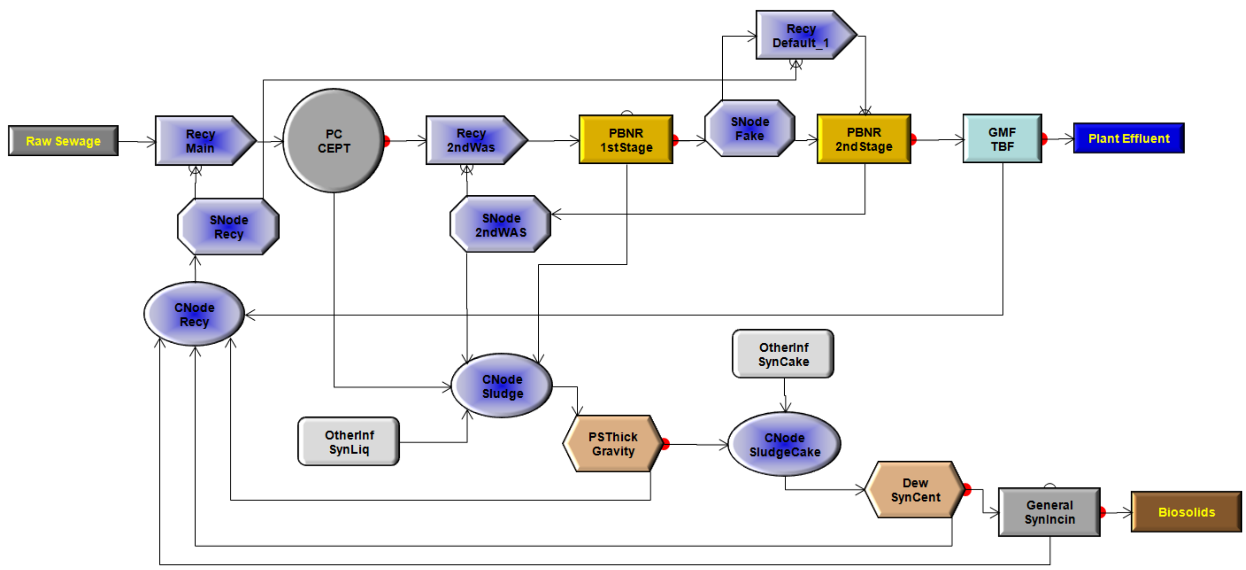
4.A.4.1.7 Solids Treatment

Waste solids from primary settling and waste activated sludge are directed to the Synagro system for treatment. In the model, the primary sludge and WAS is directed to a thickening and fermentation module, where it is combined with the Synagro imported liquid sludge. Thickened sludge from this unit is then combined with the Synagro imported cake for dewatering prior to incineration.

The Synagro influent sludges were characterized as if they were equivalent to primary sludge, then adjusted in character to give the recycle characteristics described in Chapter 3. The one difference in the modeling was that the phosphorus content of the Synagro sludges was kept constant in all runs at 0.007 lbs phosphorus/lb VSS. This resulted in higher returned recycle phosphorus loads than described in Chapter 3, This conservatism was not felt to be detrimental to the overall modeling effort.

The solids stream process flow diagram does not match the exact setup of the actual plant. This was done to capture the fermentation aspects of both the imported liquid sludge and the fermentation in the gravity thickener in a single module. Since performance was adjusted in the solids system to match the design recycle values, this difference in the setup has no impact upon the overall design of the main plant.

FIGURE 4.A-3
WOONSOCKEET WHOLE PLANT PROCESS FLOW DIAGRAM AS MODELED



4.A.4.2 Calibration

Model calibration was done in two parts. The first was to use the information provided in the RFP and from subsequent wastewater characterization studies to fractionate the wastewater. The wastewater characterization studies included special sampling of the raw sewage, recycles and PE streams to fractionate the bulk parameters (such as BOD and TSS) to useful modeling fractions such as COD, VSS, biodegradable/nonbiodegradable, and nutrients. Once the wastewater was fractionated, the basic performance characteristics of the existing plant equipment was either determined from field data or estimated based on best engineering practice. These later items included such parameters as primary clarifier TSS removal and solids thickening/dewatering solids removals.

This analysis resulted in the wastewater fractionation parameters given in Table 4.A-5. The Unit process performance parameters are given in Appendix F.

TABLE 4.A-5
RAW WASTEWATER FRACTIONATION

Raw Wastewater Characteristics			
Item	**Value**	**Typical**	**Default**
BOD_U/BOD_5 Ratio	1.40	1.4 - 1.65	1.55
Non-Biodegradable VSS, % of Total VSS	20%	20%-40%	25%
Fraction of Non-Biodegradable VSS that is Equivalent to Decay Products	40%		0%
Volatile Content of Particulate Organic Matter, % of TSS	95%	85-90%	90%
COD of VSS,mg COD/mg VSS	1.59	1.42 - 2.00	1.42
COD of the Particulate Non-Biodegradable VSS, mg COD/mg VSS	1.18		0.00
Filtrate Non-Biodegradable COD, % of Total COD	5.00%	5%-10%	5%
Volatile Fatty Acid (VFA) Content of Truly Soluble Biodegradable Material	30%	5% - 50%	20%
Portion of Filtrate COD that is Colloidal, % of Filtrate COD	12%		40%
Soluble, Non-Biodegradable Organic Nitrogen, % of Filtrate Non-Bio COD	6.00%	4% -8%	6.00%
Nitrogen Content of VSS, N/VSS (%)	3.13%	3.13%	2.28%
Phosphorus Content of VSS, P/VSS (%)	0.90%	0.01	1.00%
Colloidal Non-Biodegradable Fraction (% of Colloidal COD)	10%	10%	100%

4.A.4.3 Scenarios

The scenarios summarized in Table 4.A-6 were modeled for the two-stage activated sludge AB treatment process.

TABLE 4.A-6
PROCESS MODEL SCENARIOS

Condition	**Loading Condition**	**Raw Wastewater Temperature, C**	**Stage 1 Bioreactor Water Temperature, C**
Annual Average Summer	Annual Average	13.9	22.9
Annual Average Winter	Annual Average	12.0	19.1
Maximum Month Summer	Maximum Month	16.2	22.9
Maximum Month Winter	Maximum Month	12.0	19.1
Annual Average Winter, Incinerator OOS	Annual Average	12.0	13.6
Annual Average April, Incinerator OOS	Annual Average	13.6	13.6

The treatment processes are generally sized to meet the Maximum Month Summer condition because effluent criteria is most stringent in the summer. Winter scenarios are used to evaluate operational requirements. Two additional scenarios verify that the plant can meet effluent criteria with the Synagro incinerator out of service. Suspension of incineration impacts the recycle sludge quality and lowers the temperature of the wastewater in secondary treatment.

4.A.4.4 Results

The process models are used to estimate many aspects of future performance. The model results for Annual Average and Maximum Month Summer scenarios are summarized in Table 4.A-7.

TABLE 4.A-7
TWO-STAGE ACTIVATED SLUDGE AB PROCESS MODEL RESULTS

Parameter	**Units**	**Annual Average Summer**	**Maximum Month Summer**
Primary Clarifier			
Hydraulic Loading	gpd/sf	957	1,580
Ferric Chloride Addition	lbs/d	517	853

TABLE 4.A-7
TWO-STAGE ACTIVATED SLUDGE AB PROCESS MODEL RESULTS

Parameter	Units	Annual Average Summer	Maximum Month Summer
BOD Removal	%	35%	41%
Secondary A-Stage			
SRT	days	4	3
MLSS	mg/L	1,875	1,872
MLVSS	mg/L	1,481	1,472
Food to Mass	lbs-COD/lbs-VSS	0.57	0.69
AOR	lbs-O2/d	19,892	22,196
Air Required	scfm	7,749	9,767
Phosphorus Addition	lbs/d	71	100
Secondary Settler			
Hydraulic Loading	gpd/sf	271	447
Solids Loading	lbs/d/sf	6	10
Secondary B-Stage			
SRT	days	10	10
MLSS	mg/L	1,885	2,870
MLVSS	mg/L	1,088	1,710
Food to Mass	lbs-COD/lbs-VSS	0.05	0.06
AOR	lbs-O2/d	2,594	5,096
Air Required	scfm	961	1,728
Methanol Addition	gpd	45	40
Secondary Clarifier			
Hydraulic Loading	gpd/sf	436	717
Solids Loading	lbs/d/sf	10	25
Ferric Chloride Addition	lbs/d	400	700
Tertiary Filtration			
Hydraulic Loading Rate	gpm/sf	1.41	2.42
Sludge Production			
Primary Sludge	lbs/d	18,257	28,013
Waste Activated Sludge	lbs/d	11,262	14,321
Net Sludge Discharge	tons/d	8.1	11.0

TABLE 4.A-7
TWO-STAGE ACTIVATED SLUDGE AB PROCESS MODEL RESULTS

Parameter	Units	Annual Average Summer	Maximum Month Summer
Effluent Quality			
Total Nitrogen	mg-N/L	2.5	2.5
Total Phosphorus	mg-P/L	<0.1	<0.1
Ammonia	mg-N/L	0.1	0.1

4.A.4.4.1 Mixed Liquor Suspended Solids Inventory

The upgraded secondary treatment stages are sized for a maximum month MLSS concentration of approximately 2,300 mg/L in Stage A and 3,000 mg/L in Stage B. These concentrations are substantially lower than the 4,000 – 5,000 mg/L that the existing basins are operated at, as described in Chapter 4, and should improve the performance of the secondary clarifiers.

The lower MLSS concentrations are achieved by operating the A Stage at 3 – 4 days aerobic SRT. The low SRT decreases the aeration requirements while increasing sludge production. The smaller B Stage is maintained at a long enough SRT to promote nitrification.

4.A.4.4.2 Sludge Production Rates

Primary Sludge is generated at approximately 18,000 – 28,000 lbs/d. This includes sludge generated by ferric chloride addition for CEPT operation.

The Waste Activated Sludge (WAS) sent to Synagro is calculated to be 11,000 to 15,000 lbs/d. The WAS from Stage B is discharged into stage A. The Stage A WAS removes the biomass produced by both stages.

The net production of sludge, after subtracting Synagro recycle loads, is 8 tons/d at design annual average and 11 tons/d at maximum month conditions.

4.A.4.4.3 Aeration

The net air requirement is 8,710 scfm at annual average design and 11,495 scfm at maximum month. This is a decrease from previous plant operation described in section 4.2.2.3. The reduction in air requirements can be attributed to better DO control and shorter aerobic SRT.

4.A.4.4.4 pH and Alkalinity

The AB configuration includes some denitrification upstream of nitrification. This maintains alkalinity throughout the aeration basins at a concentration high enough to maintain an adequate pH. No lime will be added at the primary clarifiers, so the pH is expected to be close to 7. The Pro2D model used a pH input of 7.2 in the aeration basins.

4.A.4.4.5 Secondary Settler and Clarifier Overflow Loading Rates

The new lamella inclined plate settlers are sized for the hydraulic loading and operate at 271 – 447 gpd/sf of projected plate area. The solids loading on the settlers is only 6 – 10 lbs/d/sf of projected plate area for average and maximum month flows. The performance of these settlers is not critical because any solids that are not captured continue to Stage B.

The AB process results in lower solids loading on the existing secondary clarifiers. This is because the MLSS concentration in Stage B is maintained at or below approximately 3,000 mg/L. This results in an average solids loading rate of 10 lbs/d/sf and a maximum month rate of 25 lbs/d/sf. These rates are within typical clarifier design parameters.

4.A.4.4.6 Nitrification and Nitrogen Removal

Full nitrification is achieved in Stage B. Dissolved oxygen is continuously monitored in each aerobic zone and process air flow to each zone is independently controlled. Reliable dissolved oxygen concentrations, a 10 day aerobic SRT, and warm wastewater should provide robust nitrification. Additionally, WAS from stage B is discharged in Stage A. This transfer of nitrifiers allows a limited amount of nitrification to occur in the Stage A.

Denitrification requires nitrate and BOD and it occurs in both stages. Most nitrate produced in or recycled back to Stage A can be consumed in the pre-anoxic zone using influent BOD. Anoxic zones in Stage B allow additional denitrification but rely primarily on carbon addition. A portion of the Synagro recycles may be routed to Stage B as a carbon source.

4.A.4.4.7 Phosphorus Removal

Ferric addition at the primary and secondary clarifiers chemically binds orthophosphate. This removes raw influent phosphorus in the primary clarifiers and it reduces the phosphorus load from the Synagro recycle. The model predicts that no BPR occurs in the aeration basins under these conditions. The phosphorus load of the Synagro recycle is dramatically reduced because without BPR there is no phosphorus release in the gravity thickener.

4.A.4.4.8 Chemical Requirements

Ferric chloride addition at the primary clarifiers reduces odors, precipitates phosphorus, and has a small impact on BOD removal. Ferric chloride is also added at the secondary clarifiers to precipitate phosphorus and enhance settling. An average of 917 lbs/d ferric chloride was required and 1,553 lbs/d at maximum month.

With ferric addition, secondary treatment may become phosphorus limited. The process model required a supplement of 71 lbs-P/d for annual average and 100 lbs-P/d for maximum month conditions. Full scale optimization of ferric dosing and phosphorus addition may reduce these chemical requirements.

Denitrification in Stage B requires methanol addition. A dose of approximately 45 gpd was required to maintain an effluent total nitrogen concentration below 3 mg-N/L. However, during winter months methanol addition can be reduced or suspended because of less stringent effluent criteria.

SECTION 5
SCREENING OF NUTRIENT REMOVAL ALTERNATIVES

5.1 INTRODUCTION

Potential alternatives to enhance the nitrogen and phosphorus removal capabilities of the Woonsocket WWTF are presented in this section. As discussed in Section 1, Woonsocket is under a Consent Agreement to achieve new total nitrogen (TN) and total phosphorus (TP) limits. The proposed RIPDES effluent limits, as it relates to the selection of a nitrogen and phosphorus removal technology, are defined in Table 5-1

TABLE 5-1
REVISED RIPDES EFFLUENT NUTRIENT LIMITS

Effluent Characteristic	Discharge Limitations		Monitoring Requirements	
	Average Monthly	Max Daily	Measurement Frequency	Sample Type
Flow	16.0 MGD			
Phosphorus, Total:				
(Nov1 – Mar 31)	1 mg/L		3/ Week	24-hr comp.
(April 1 – Oct 31)	0.1 mg/L		3/ Week	24-hr comp.
Nitrogen, Total:				
(May 1 – Oct 31)	3 mg/L		3/ Week	24-hr comp.
(April)	10 mg/L		3/ Week	24-hr comp.
(May 1 – Oct 31)	400 lb/d		3/ Week	24-hr comp.
Ammonia, Total:				
(June – Oct 31)	2 mg/L	49.4 mg/L	3/ Week	24-hr comp.
(Nov. 1 – Apr. 30)	15 mg/L	53.8 mg/L	1/Week	24-hr comp.
(May)	12 mg/L	53.8 mg/L	1/Week	24-hr comp.

5.2 NUTRIENT REMOVAL TECHNOLOGIES

5.2.1 Nitrogen Removal

Total nitrogen removal is accomplished through the use of two main biological processes; nitrification and denitrification. When coupled together, influent nitrogen is reduced through either converting the influent nitrogen to nitrogen gas or capturing it as a solid and "wasting" it out of the system. Currently, the Woonsocket plant provides both nitrification and denitrification within the existing secondary treatment system using the MLE process configuration. However, to improve the nitrogen removal to meet the new permit limits, the existing denitrification removal process will need to be enhanced.

In general, biological nitrogen removal (denitrification) processes can be grouped into two categories: substrate level (exogenous) denitrification and endogenous level denitrification. Substrate level denitrification processes can achieve effluent total nitrogen levels in the 6.5 to 7.0 mg/l range. These processes are characterized as having an initial anoxic zone (i.e., an unaerated zone within the aeration tanks) in which the influent BOD5 is the substrate that drives the denitrification process.

When substrate level denitrification is combined with endogenous level denitrification, effluent total nitrogen levels below 3.0 mg/l are possible, depending on residual non-biodegradable nitrogen fractions. Endogenous level denitrification processes are characterized by having anoxic zones at or near the effluent end of the aeration tanks. Historically, carbon released from cell decay was used to drive the denitrification process. The use of an external carbon source (i.e., methanol) is typically used to reduce the required post anoxic volume and produce reliable process performance. The anoxic zone can be included in the activated sludge tanks (single sludge suspended growth processes) or as a stand-alone process downstream of the secondary clarifiers (separate 2nd stage sludge tertiary processes).

Typically, endogenous level denitrification processes alone (i.e., tertiary denitrification systems) are not cost-effective for the treatment of nitrogen (i.e. versus combined processes) due to the amount of supplemental carbon required. If effluent TN levels less than 3.0 mg/l are required then more advanced non-biological processes may be required.

5.2.1.1 Nitrogen Characterization and the Ability to Achieve 3.0 mg/l TN

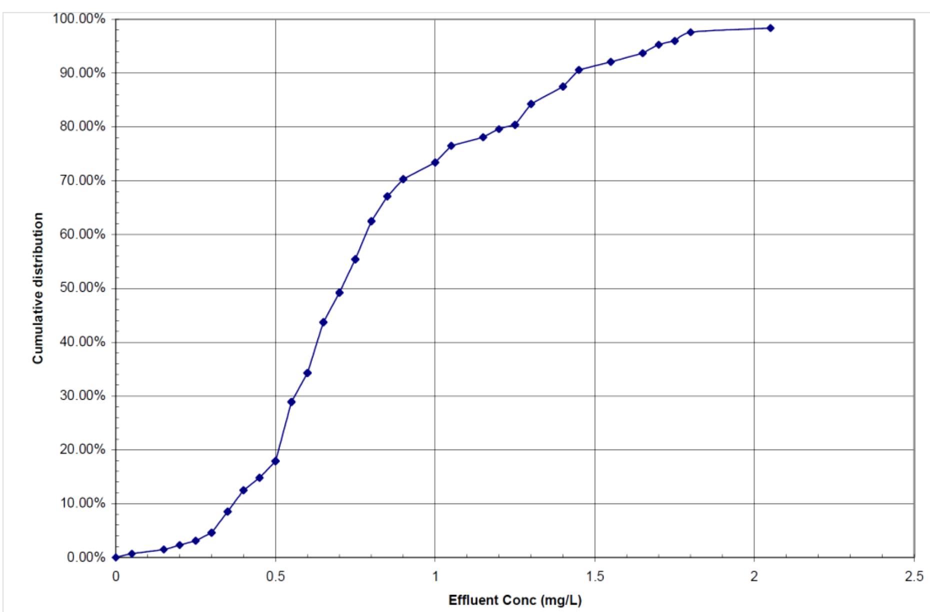
Total nitrogen is the sum of several nitrogen components including ammonia, organic nitrogen, nitrate and nitrite. The ability to consistently meet effluent total nitrogen levels of 3.0 mg/l is predicated on the robustness of the selected nitrogen removal technology (i.e., its ability to achieve a high level of nitrogen removal performance under varying influent conditions) and the inherent characteristics of the nitrogen species found in the wastewater.

Of particular concern is the dissolved organic nitrogen (DON) fraction. Effluent DON is primarily due to recalcitrant or hard-to-degrade forms of the influent nitrogen which can pass through the treatment plant unchanged. Typical municipal recalcitrant DON (rDON) levels range from 0.5 – 2.0 mg/l. The effluent rDON value is a function of the influent wastewater characteristics, not the specific process employed at the facility.

The effluent rDON will impact the facilities ability to achieve an effluent TN of 3.0 mg/l regardless of the biological nitrogen removal technology selected. Figure 5-1 shows the frequency distribution for the historical (2008-2010) effluent organic nitrogen values at the Woonsocket facility. These values could be particulate or dissolved in nature. Hence, additional nitrogen removal can be achieved through a further reduction in the effluent solids levels; therefore, removals are enhanced with the existing effluent filtration. Thus, the true rDON value at this plant will be below the values presented in Figure 5-1.

A review of the historic operating data indicates that ninety percent of the time the effluent organic nitrogen has been below 1.5 mg/l. The effluent organic nitrogen presented in Figure 5- 1 is final effluent (i.e., after sand filtration). While the effluent suspended solids level is low at the Woonsocket WWTF, additional solid reduction is expected through the new two-stage activated sludge AB process, further reducing the effluent organic nitrogen.

FIGURE 5-1
FREQUENCY DISTRIBUTION FOR EFFLUENT ORGANIC NITROGEN



The remaining nitrogen compounds of the effluent total nitrogen are ammonia and nitrate/nitrite and are affected by the operation of the biological process. Ammonia reduction is achieved via nitrification which occurs in the existing secondary treatment process. The effluent ammonia level is a function of the sludge retention time and the process operating characteristics (i.e., temperature, pH, dissolved oxygen level, etc.). Effluent ammonia levels of less than 1.0 mg/l are typical of systems that fully nitrify. The historical plant data suggests periods of low ammonia levels followed by small daily ammonia spikes (i.e., 1.5 to 2.0 mg/l of ammonia). A more detailed review of the operating data will need to be conducted to determine the cause of the "spikes". For 2009, the monthly average effluent ammonia level was below 0.2 mg/l (except for the months of November and December) which indicates that full nitrification is achievable at current flows and loads.

Nitrate/nitrite levels consistently below 0.5 mg/l should be achievable with a process that fully denitrifies and is affected by variations in dissolved oxygen levels, mixed liquor concentrations and readily biodegradable carbon in the anoxic zones. Supplemental carbon should be included in the design for periods when wastewater characteristics are less than optimum.

The combined nitrogen concentrations of ammonia (<1.0 mg/l), organic nitrogen (1.5 mg/l) and nitrate (0.5 mg/l) should result in a facility that can meet the proposed TN limit of 3.0 mg/l. Given that the existing secondary process has been able to achieve a monthly average ammonia limit well below 1.0 mg/l, compliance with a total nitrogen limit of 3.0 mg/l is possible.

5.2.1.2 Single Sludge Versus Separate Tertiary Processes

There are several excellent methods of nutrient removal for designs of wastewater treatment processes to 3.0 mg/l total effluent nitrogen. Either single-stage or two-stage activated sludge systems are viable. These processes can take various physical configurations within the treatment plant. A common method to achieve an effluent total nitrogen concentration of 3.0 mg/l is usually a combined denitrification system that utilizes both pre and post anoxic zones (exogenous and endogenous processes). The endogenous stage can be included into the main activated sludge process (single sludge suspended growth processes) or as a standalone process downstream of the secondary clarifiers (separate sludge tertiary processes). Examples of each include:

Single sludge suspended growth processes:

- 4-stage Bardenpho process
- Sequencing Batch Reactor

Two-Stage Activated Sludge

- First Stage nitrification followed by Second Stage denitrification
- Two-Stage (AB Process) with nitrification and denitrification in both stages

Separate sludge tertiary processes:

- Denitrification Filter
- Biological Anoxic Filter
- Tertiary Moving Bed Biofilm Reactor (MBBR)

Each of these processes has shown the ability to achieve a total nitrogen effluent concentration of 3.0 mg/l. A number of facilities in Western States and several facilities in the Southeast have a total nitrogen limit of 3.0 mg/l on a monthly average basis. Advanced nitrogen removal processes require operational flexibility, a robust design, and adequate wastewater characterization to meet performance requirements. Safety factors are needed to adequately cover variations in wastewater influent and operations to meet permit compliance goals.

While there is data to support each of these processes as having the ability to meet the proposed nitrogen limit, the two-stage activated sludge AB process has been proven effective in several facilities, including Strass Austria and DCWASA. A two-stage activated sludge process followed by effluent filtration provides several benefits, as described in the following paragraphs.

The AB process allows operational flexibility since it includes aeration basins with two distinct and separate solids retention times (SRT). The first stage is operated at a lower SRT to achieve nitrification and denitrification by seeding nitrifiers from the second stage. During wet weather the first stage protects the second stage from washout and can also be operated in a step feed mode. The second stage, which operates at a higher SRT (typically 9-10 days) returns a portion of the well nitrified mixed liquor to the first stage for seeding.

5.2.2 Phosphorus Removal

Phosphorus removal can be accomplished either through biological and/or chemical removal processes. A portion of the influent phosphorus is used by the bacteria present in the secondary treatment process for growth. The remaining phosphorus can be removed by enhancing the biological uptake of the phosphorus or by chemical removal. In each case, the phosphorus is converted to a solid and wasted out of the system. Therefore, the only way that phosphorus is removed from the system is through the solids removal process. The phosphorus is ultimately removed from the WWTF in the incinerator ash.

To achieve the proposed total phosphorus (TP) limit of 0.1 mg/l, a chemical phosphorus removal step will be required as biological phosphorus removal cannot easily meet the proposed limits. Additionally, a very high level of solids removal will be required (to remove the particulate phosphorus). Processes that can achieve a total phosphorus limit less than 0.1 mg/l routinely have effluent total suspended solids concentrations less than 2.0 mg/l.

Chemical phosphorus removal consists of adding a metal salt to the waste stream in an effort to convert the soluble phosphorus to particulate form. Then the particulate phosphorus is removed in a solids separation step. The chemical dose is a function of the total amount of phosphorus to be removed and the effluent TP goal (the amount of chemical addition increases as the effluent TP limit is reduced).

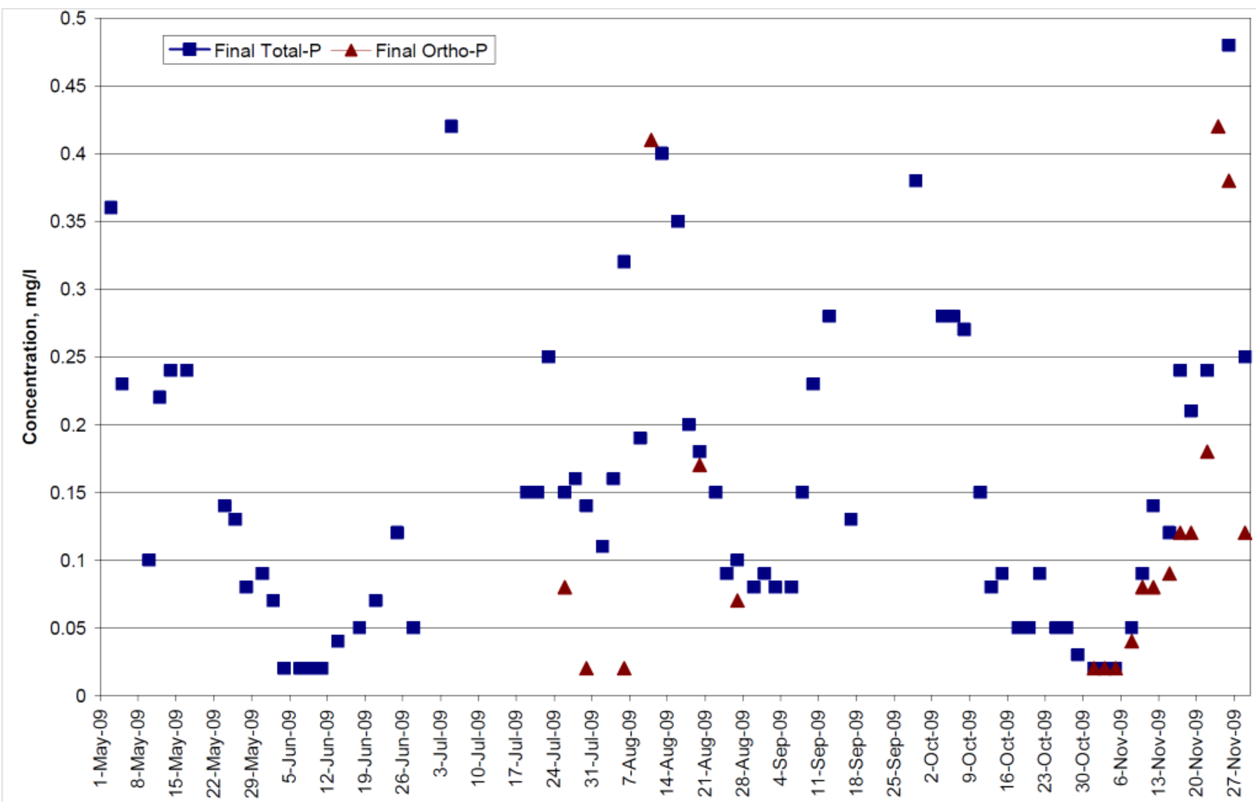
5.2.2.1 Phosphorus Characterization and the Ability to Achieve 0.1 mg/l TP

Total phosphorus is the sum of several phosphorus components. These components can be classified as either soluble or particulate. Particulate phosphorus can be removed to varying degrees based on the solids removal performance of a selected technology. Soluble phosphorus can be classified as either reactive or non-reactive. The term reactive is the portion of the phosphorus that will react with reagents eventually allowing for its removal as a particulate. Inorganic orthophosphate (PO4) is the largest component of this group.

The chemical species that make up non-reactive soluble phosphorus is unknown. What is known is that the majority of this phosphorus component cannot be removed via current tertiary phosphorus removal processes and will pass through the facility into the effluent untreated.

The historical data at the Woonsocket WWTF does not lend itself well to determine the value of non-reactive soluble phosphorus. However, as shown in Figure 5-2, the plant has measured effluent total and ortho-phosphorus concentrations well below the proposed limit of 0.1 mg/l at times, indicating that during those periods non-reactive soluble phosphorus was not at a level high enough to create a concern with the facility's ability to achieve a total phosphorus limit of 0.1 mg/l. However, there are also periods where the total and ortho-phosphorus levels may include elevated non-reactive soluble phosphorus.

FIGURE 5-2
TOTAL AND ORTHO PHOSPHORUS IN FINAL EFFLUENT
(MAY – NOVEMBER 2009)



5.2.3 Combined Biological Nitrogen and Phosphorus Removal Processes

There are biological nutrient removal processes that combine nitrogen removal processes with biological phosphorus removal (i.e., 5-stage Bardenpho). Enhanced Biological Phosphorus Removal (EBPR) processes have been demonstrated to be capable of reducing effluent total phosphorus to as low as 0.1 mg/L. However, when combined with nitrogen removal their performance is typically reduced.

EBPR should lower, but not eliminate, operational costs due to chemical addition, because the low total phosphorus effluent limit of 0.1 mg/l will require chemical precipitation and tertiary solids removal (to be discussed later in this section). Biological phosphorus removal and nitrogen removal processes both rely on carbon as the driving force behind their performance. Hence they are always in competition with each other, potentially resulting in reduced performance of both processes.

Following the selection of the nutrient removal processes, consideration will be given to incorporating/maintaining biological phosphorus removal within the existing aeration basins.

5.2.4 Nitrogen and Phosphorus Removal Process Alternatives

A broad array of technologies has been used successfully for nitrogen and phosphorus removal at municipal wastewater treatment facilities, as summarized in Table 5-2. The performance exhibited by each technology will vary from facility to facility based on site specific wastewater characteristics, process loadings (i.e., percent of design capacity), secondary clarifier performance and filtration performance (for particulate nitrogen removal).

The technologies presented in Table 5-2 can be categorized as single-stage activated sludge, two-stage activated sludge, or hybrid sludge processes with or without enhanced biological phosphorus removal, separate sludge fixed-film systems, and physical-chemical phosphorus treatment processes.

Several of the biological process configurations can be enhanced in an effort to reduce the tank volume required by the processes. Typically, these enhancements increase the amount of bacteria present in a given volume. Examples include two-stage activated sludge processes, membrane bioreactors, integrated fixed film activated sludge (IFAS) and BioMag. The proposed total nitrogen (TN) limit of 3.0 mg/l limits the processes that were considered for the Woonsocket

WWTF. Each of the processes in Table 5-2 are capable of achieving 3.0 mg/L and were reviewed in more detail, except the Sequencing Batch Reactor process. This process requires significantly different tankage than the current process utilized at the Woonsocket facility and is not compatible with current facilities infrastructure. Furthermore, this process is not well suited for larger plants as is the case at Woonsocket. Given the significant changes required to install this process it was eliminated from further consideration.

TABLE 5-2
NITROGEN AND PHOSPHORUS PROCESS ALTERNATIVES

<table>
<tr><th></th><th>Bio-N Removal Processes</th><th>Bio-N and P Removal Processes</th><th>P Removal Processes</th><th></th></tr>
<tr><td>TN 6-10 mg/L</td><td>Modified Lutzack-Ettinger (MLE)</td><td></td><td></td><td>TP > 1.0 mg/L</td></tr>
<tr><td>Tn 5-8 mg/L</td><td>Cyclic Aeration
Simultaneous Nit-Denit
Step-Feed</td><td>Anaerobic-Anoxic-Oxic (A2O)
Modified UCT
Virginia Initiative Process (VIP)</td><td rowspan="2">Anaerobic-Oxic (AO)</td><td rowspan="2">TP 0.5 – 1.0 mg/L</td></tr>
<tr><td rowspan="3">TN 3-5 mg/L</td><td rowspan="3">Sequencing Batch Reactor
4-Stage Bardenpho
Tertiary denitrification sand filter
Tertiary biological anoxic filter
Tertiary Anoxic Moving Bed Biofilm Reactor
Two-Stage Activated Sludge Process
Two-Stage Activated Sludge AB Process</td><td>5-Stage Bardenpho</td></tr>
<tr><td rowspan="2"></td><td>Cloth Filtration
Sand Filtration</td><td>TP 0.1 to 0.5 mg/L</td></tr>
<tr><td>Two-stage activated sludge process followed by Sand Filtration
"Activated" Filtration
Dissolved Air Flotation
Ballasted Flocculation
Membrane Filtration</td><td>TP < 0.1 mg/L</td></tr>
</table>

Of the identified technologies in Table 5-2, the membrane filtration process will be significantly more costly to build and operate than the other processes. A tertiary membrane filtration process, while capable of meeting the 0.1 mg/l total phosphorus limit, is better suited where other project drivers (i.e., water re-use) requires the use of membranes versus just low TP levels. Thus, given the site specific issues at the Woonsocket facility, this process alternative was eliminated from further consideration.

5.3 PRELIMINARY SCREENING OF NITROGEN REMOVAL ALTERNATIVES

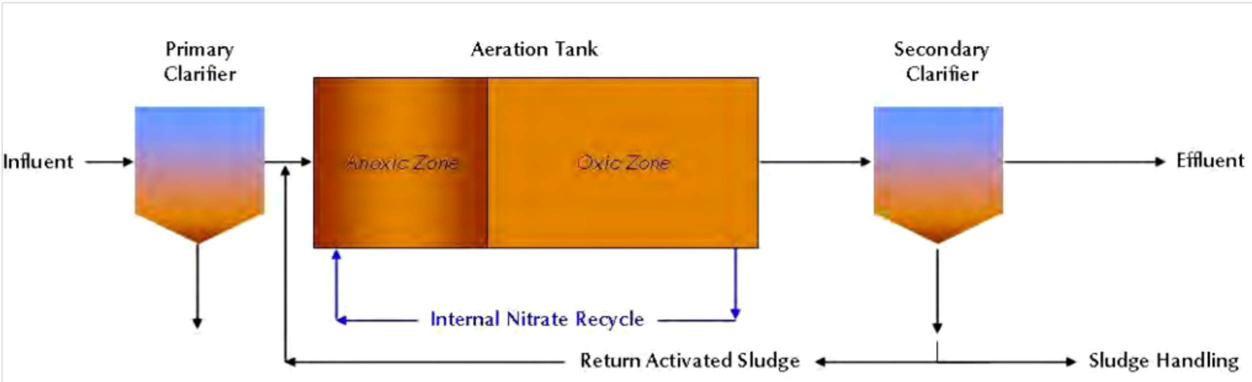
5.3.1 Single Sludge Suspended Growth Processes

5.3.1.1 Modified Ludzack-Ettinger Process

The Woonsocket WWTF currently utilizes the Modified Ludzack-Ettinger (MLE) process to remove nitrogen, BOD5 and TSS. The MLE process is configured to have one or more anoxic reactors precede the aerated reactor(s) of an activated sludge system. Primary effluent and return activated sludge (RAS) is fed into the anoxic reactor. The configuration of the reactors uses the organic carbon present in the influent wastewater for denitrification. This process is shown in Figure 5-3.

To achieve biological nitrogen removal, ammonia must first be completely transformed to nitrate (nitrification) in the oxic zone of the activated sludge system. Nitrates produced in the aerobic zones are recycled back to the anoxic zone through a pumped internal recycle system where they come in contact with the raw soluble BOD5, thus creating anoxic conditions within the zone conducive for denitrification. Biological phosphorus removal, while not specifically designed into the MLE process, can be achieved through a reduction in the internal recycle rate. As the amount of nitrates decrease, the anoxic zone can become anaerobic resulting in some enhanced biological phosphorus removal. Thus, while not designed for, the MLE process can exhibit some enhanced reduction of phosphorus. To achieve additional phosphorus removal, chemical precipitation of the phosphorus would be necessary.

FIGURE 5-3
MLE PROCESS FLOW DIAGRAM



The limit of technology for the MLE process is typically considered between 6 to 10 mg/l of total nitrogen. The effluent total nitrogen level achieved is highly dependent on the amount of influent substrate carbon available for the denitrification process. Increasing the influent carbon-to-nitrogen ratio typically results in improved performance.

This process configuration cannot achieve the proposed RIPDES TN limit of 3.0 mg/l. The installation of additional components (aeration tanks, internal recycle pumps, secondary clarifiers, etc) would result in additional capacity but would not have a meaningful impact on the effluent total nitrogen. Therefore, this process alternative was eliminated from further consideration.

5.3.1.2 Bardenpho Process

The Bardenpho process (either 4 or 5-stage) has been used successfully to meet a total nitrogen limit of 3.0 mg/l. New England installations in Connecticut include Glastonbury; Fairfield; Stratford; and Waterbury. The 5-stage Bardenpho process includes an initial anaerobic reactor followed by a primary anoxic zone, primary oxic zone, secondary anoxic zone and reaeration zone in series through the aeration tank. The first-stage anaerobic zone is used for biological phosphorus removal while the remaining anoxic and oxic zones are primarily for nitrogen removal. The first anoxic zone and oxic zone are essentially the same process as the MLE process. Nitrates are recycled from the effluent end of the first oxic stage to the first anoxic stage. However, a secondary anoxic zone is also provided for additional denitrification to further reduce the effluent total nitrogen from this process as shown in Figure 5-4. The reaeration zone at the end is provided to add dissolved oxygen to the mixed liquor prior to the secondary clarifiers. To provide sufficient food (carbon) to complete the denitrification reactions, a supplemental carbon source is typically utilized in the secondary anoxic zone. As previously noted, this reduces the necessary size of the second anoxic zone compared to relying on endogenous decay.

The 4-Stage Bardenpho process operates in the same configuration as the 5-Stage Bardenpho process without the initial anaerobic zone at the influent end of the aeration tank. The 4-Stage Bardenpho process provides for biological nitrogen removal but does not provide for enhanced biological phosphorus removal. Because the microorganisms responsible for denitrification are not competing with phosphorus accumulating organisms, better nitrogen reduction can be achieved in the 4-Stage Bardenpho process than in the 5-Stage Bardenpho process if influent carbon is a limiting factor (due to a lack of sufficient carbon in the first anoxic zone following carbon utilization in the anaerobic zone). As shown in Figure 5-4 it is also possible to add supplemental carbon to the secondary influent to drive higher phosphorus and nitrogen removal rates.

As for all biological process, the final sizing criteria would be determined using the BioWin process model. However, for this level of evaluation, preliminary sizing can be developed using "rule-of-thumb" calculations. Typical hydraulic residence times for each zone of the 5- Stage Bardenpho process and the 4-Stage process are shown in Table 5-3.

FIGURE 5-4
FIVE-STAGE BARDENPHO PROCESS FLOW DIAGRAM

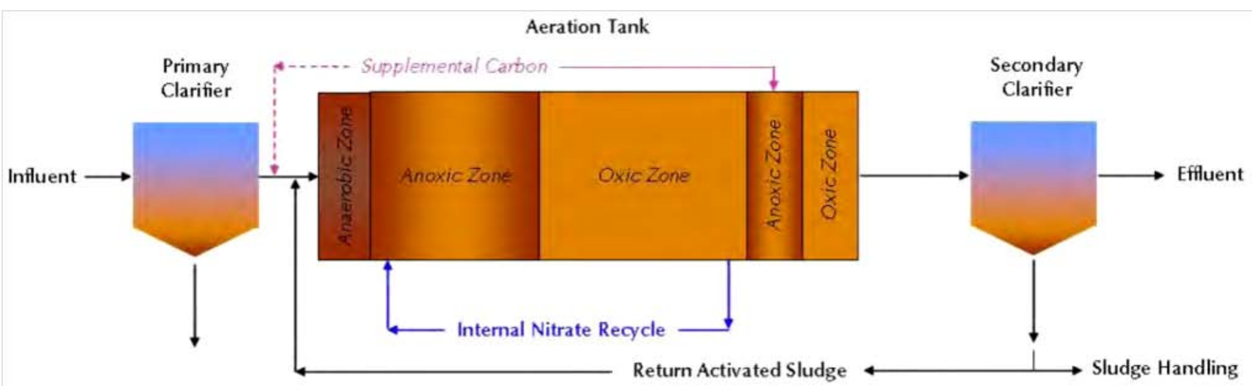


TABLE 5-3
PRELIMINARY SIZING CRITERIA FOR 4- AND 5- STAGE BARDENPHO

	Hydraulic Residence Time (hrs)	**Typical Hydraulic Residence Time (hrs)**	
	Existing MLE Process (at 18.6 mgd[1])	**5-stage Bardenpho**	**4-stage Bardenpho**
Anaerobic Zone	N/A	0.5 – 1.0	N/A
Anoxic Zone	1.84	2.5 – 4.0	2.5 – 4.0
Aerobic Zone	5.51	6.0 – 10.0	6.0 – 10.0
Post-Anoxic Zone	N/A	2.0 – 4.0	2.0 – 4.0
Post-Aerobic Zone	N/A	0.25 – 0.5	0.25 – 0.5
Total	**7.35**	**11.25 – 19.5**	**10.75 – 18.5**

Note:
[1] Current design maximum month flows for secondary influent

It should be noted that the typical residence times shown in Table 5-3 assumed mixed liquor concentrations between 2,000 and 4,000 mg/l. The residence times presented for the current MLE process are based on current design maximum month flow conditions. In order to provide the hydraulic residence time needed for the Bardenpho process, additional aeration tankage would be needed. The current MLE process is operating slightly below the desired range of 8.5 to 14 hrs total at the maximum month flow rate.

The facility currently has six 0.95 mgal aeration tanks with a total volume of 5.7 mgal. Installing two new 0.95 mgal tank would increase the total aeration volume to 7.6 mgal for a total HRT of 9.8 hrs. Three new 0.95 mgal tanks would increase the total aeration volume to 8.55 mgal for a total HRT of 11.0 hrs. Given the site constraints, the construction of additional aeration tanks is not feasible. Therefore, the 4-Stage and 5-Stage Bardenpho alternatives were removed from further consideration.

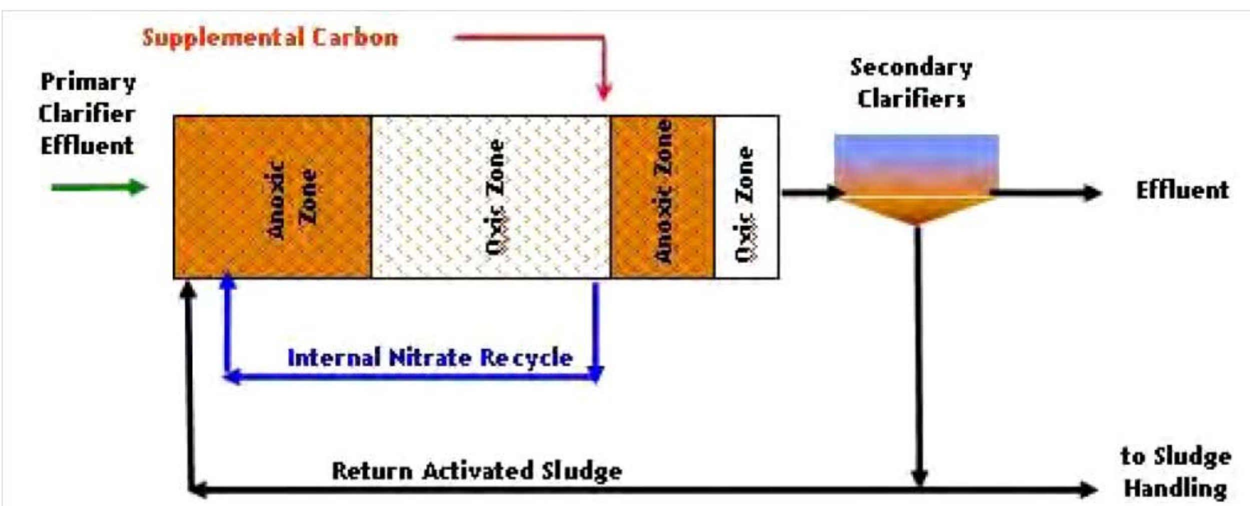
5.3.1.3 Bardenpho with IFAS, BioMag, or Membrane Bioreactor Process

These three alternatives would seek to meet the proposed effluent limits using the Bardenpho process by retaining increased biomass concentrations (and therefore biological treatment capacity) within the existing reactor tank volumes. Potentially, these enhancements to the Bardenpho process could eliminate the need to construct additional aeration tanks and/or secondary clarifiers.

IFAS (integrated fixed film activated sludge) is a hybrid process that uses a combination of fixed-film media and suspended growth activated sludge within the aeration tanks to retain higher levels of biomass than conventional activated sludge systems. The IFAS technology requires proprietary media and retaining screens. In some cases coarse-bubble diffusers are necessary to suspend the media. The IFAS process schematic is shown with the 4-Stage Bardenpho configuration in Figure 5-5 (the technology may be used with the 5-Stage configuration as well). The media has traditionally been used in the main oxic zone to aid in reducing the volume required for nitrification. Recent facilities have incorporated the media in the anoxic zones as well. The location and amount of media used is dictated by the additional capacity required, and thus can vary from facility to facility. In the future, additional media can be added to increase the performance or capacity of the process.

The IFAS system has been used successfully at several locations for the removal of nitrogen including Broomfield, CO; Groton, CT; and West Haven, CT. The majority of the current IFAS installations are in combination with the MLE process. Various piloting work has been conducted documenting the ability of the IFAS media in combination with a 4-stage Bardenpho process to achieve a total nitrogen limit of 3.0 mg/l. There are two IFAS facilities under construction to meet an effluent TN less than 3.0 mg/l (Fields Point WWTF, Providence, RI; and Mamaroneck, NY). The IFAS process was included for further analysis.

FIGURE 5-5
BARDENPHO CONFIGURATION WITH IFAS FOR NITROGEN REMOVAL

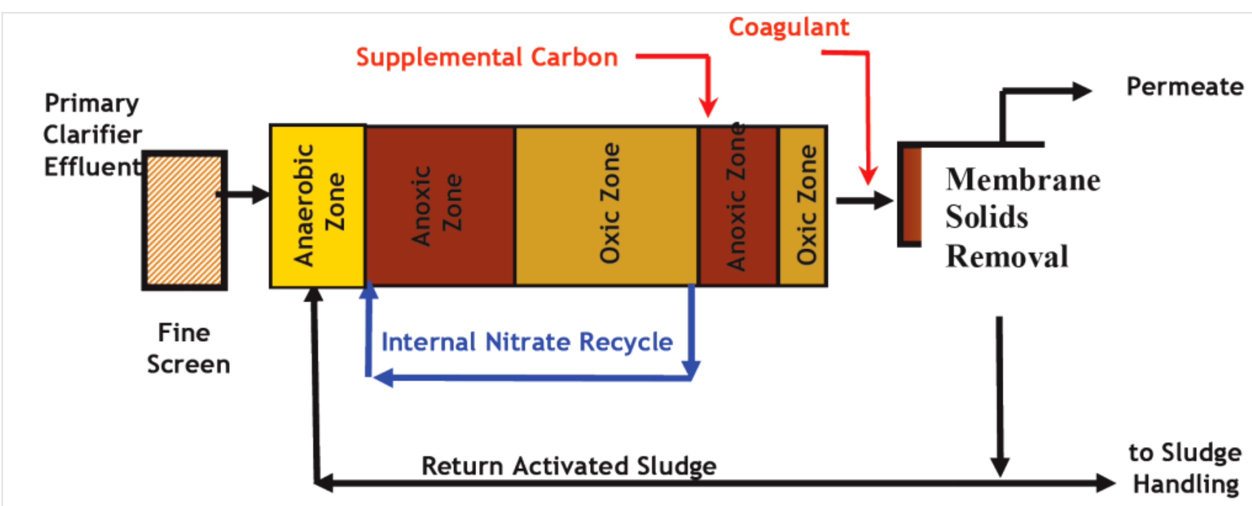


Alternatively, additional biomass within the aeration tanks could be achieved through the use of a membrane solids removal system as shown in Figure 5-6. Essentially, the membrane system replaces the secondary clarifiers as the MLSS removal process. The membrane cassettes and ancillary components could be installed within the existing secondary clarifiers. Ancillary equipment includes permeate pumps, scour blowers, chemical cleaning system, and large return activated sludge pumps (4 to 6 times influent flow). The membrane bioreactor (MBR) system requires a fine screening facility (2-3 mm band screen) upstream to protect the membrane cassettes.

By utilizing membranes in-lieu of the existing secondary clarifiers, the mixed liquor concentration can be increased to 7,000 to 10,000 mg/l, significantly reducing the amount of volume required by the Bardenpho process. The installation of a Bardenpho process and membrane system could be achievable within the existing infrastructure already onsite. The membrane system also has the inherent advantage of producing a very clean effluent potentially eliminating the need for a tertiary phosphorus removal process (assuming metal salt addition prior to the membrane cassettes).

However, one of the major disadvantages of the membrane system is the very high capital cost for the membrane equipment as well as a very high operational cost due to extensive amount of aeration and return flow rates utilized by the membrane process. The membrane cassettes also have a finite life and need to be replaced every 8 to 10 years at substantial cost.

FIGURE 5-6
BARDENPHO CONFIGURATION WITH MBR FOR NITROGEN (AND PHOSPHORUS) REMOVAL



The combined membrane-Bardenpho process has been used at the Broad Run WRF (Ashland, VA) and is being designed for the Norwalk, CT WPCF. It should be noted that at Broad Run, the process achieves an annual average TN concentration of 3.2 mg/l (without the use of supplemental carbon) as that facility's limit is 4.0 mg/l. The Norwalk design is currently on-hold because of cost and affordability concerns (capital, operational and maintenance costs).

Membrane bioreactors have proven capable of achieving low nutrient limits given sufficient biomass and reactor volume. In the case of Woonsocket, the technology could be installed without additional tank construction and potentially could eliminate the necessity of a tertiary phosphorus removal step. Reservations regarding this technology include high capital costs of the membranes, along with high energy and maintenance costs associated with permeate pumping and membrane cleaning and replacement. While intriguing due to the elimination of the tertiary phosphorus removal step, the capital and operational costs of this process are significant.

BioMag technology uses a proprietary magnetic ballasted flocculent that improves biomass settling properties, allowing the secondary system to potentially operate with higher concentrations of mixed liquor solids. The ballasted mixed liquor solids would be removed in the existing secondary clarifiers and recycled back to the aeration tanks. A ballast removal step is utilized to separate the ballast material from the waste sludge. However, it is not clear exactly how much additional active biomass can be allowed with this process. At this time, there are no full scale installations of this process. There are two pilot tests currently being conducted at smaller facilities in New England. Thus, this technology has not been applied at a scale similar to the Woonsocket WWTF, and we would not recommend further review of this technology for this application.

5.3.2 Separate Sludge Tertiary Processes

Tertiary denitrification alternatives are used in conjunction with and downstream of the biological secondary treatment processes discussed in the previous section. In each scenario, it is assumed that the upstream process would remain as an MLE process. It should be noted that a number of technologies are available that can treat both nitrogen and phosphorus in the same unit. However, denitrification is a biological process, and denitrifying microorganisms require residual phosphorus for growth (up to 1 mg/L). At the low effluent nutrient limits for Woonsocket, none of the combined tertiary processes have been able to successfully demonstrate the ability to achieve the proposed limits in a single process. Therefore, only separate denitrification tertiary systems are recommended and discussed in this section.

Each of the tertiary alternatives rely on the same biological principles to remove nitrogen. Secondary clarifier effluent (source of nitrates) along with a supplemental carbon source is introduced into the process. A specific media surface (i.e., sand, small plastic beads, etc.) is provided to enable the formation of an attached biological growth. The attached bacteria in the presence of nitrate and carbon and without oxygen will convert the nitrate to nitrogen gas ultimately reducing the effluent total nitrogen. The tertiary process does not provide ammonia removal (this needs to be accomplished in the secondary activated sludge process), but does provide for some particulate nitrogen removal through enhanced solids separation.

The major differences between the alternative tertiary nitrogen removal processes include the solids removal aspect of each process, the type of media and the methods utilized to remove the excess biological growth.

5.3.2.1 Tertiary Denitrification Sand Filter

Of the tertiary alternatives, deep-bed sand filters have by far the most experience in the United States. One example is the Tetra Denite system which is being used successfully at the Littleton-Englewood WWTF (Colorado) to achieve an effluent TN less than 3.0 mg/l. Similar to the existing Woonsocket filters, this alternative would consist of multiple filter cells in parallel operation. The number and size of the cells would be dictated by the specific manufacturers' sand filter system. Ancillary components vary from manufacturer to manufacturer. A preliminary review of the existing sand filters was conducted. However, given their shallow bed depth (12" to 18") they are not suitable for denitrification.

The denitrification sand filter process has the inherent benefit of removing nitrogen and suspended solids in a single step, eliminating the need for additional downstream nitrogen removal processes. However, it cannot achieve concurrent phosphorus removal to the levels required by the RIPDES permit. Thus a tertiary phosphorus removal process would still be required.

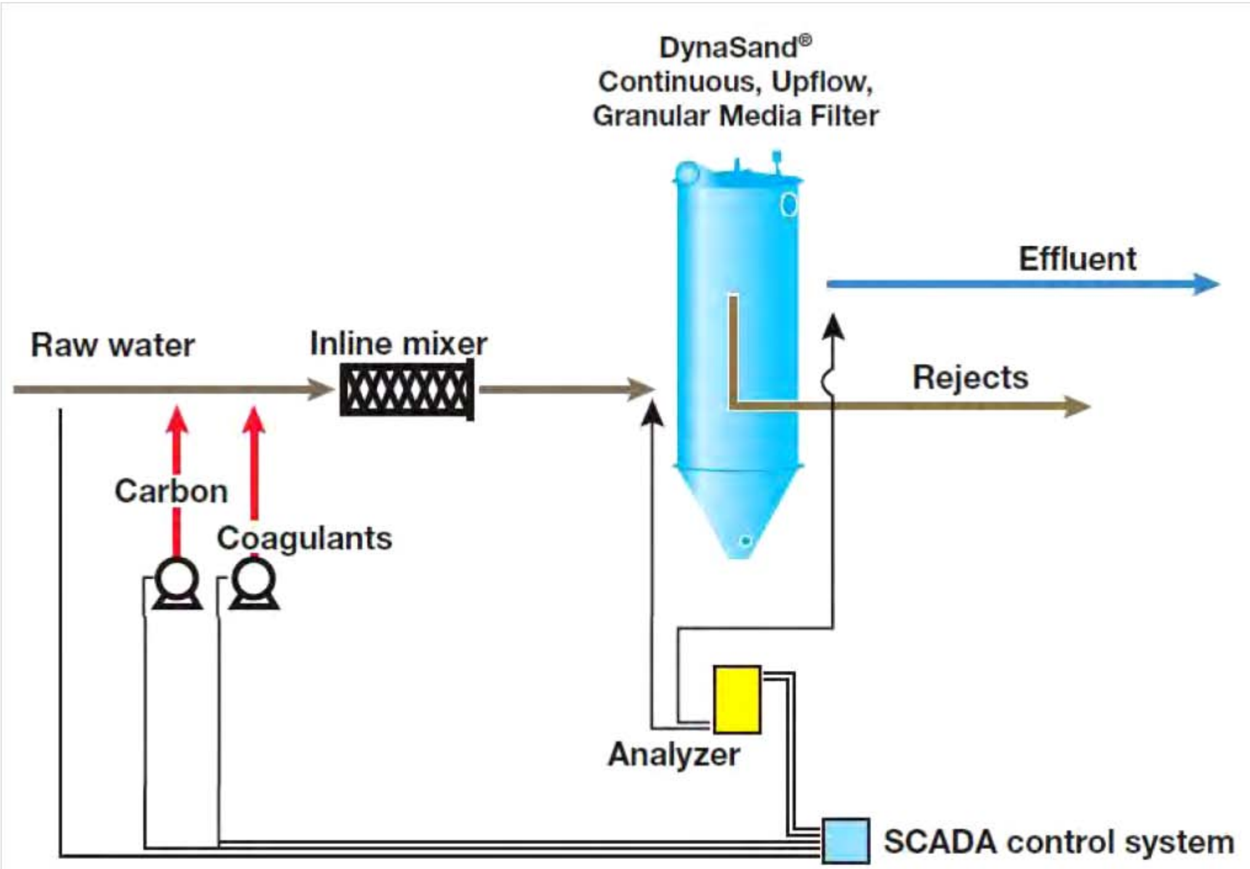
The Dynasand filter is another example of a deep bed (approximate 6-ft deep bed) sand filter used for denitrification. The Dynasand filter is an up-flow, deep-bed granular media filtration technology that does not require backwash pumps or storage tanks with its continuously self-cleaning filter and modular design. The process flow diagram of this filtration technology is shown in Figure 5-7.

This system can stand alone as a package unit with steel tanks for the filters or the filters can be installed in a poured-in-place concrete tank installation. Influent feed water is distributed at the bottom of the sand bed and flows upward through the filter to an overflow weir at the top. An air lift pipe runs down through the center of the bed. The waste sand from the bottom of the bed is lifted up using the air lift and drops down through a cleaning zone where it is scoured and washed by the filtered effluent. The cleaned sand drops onto the top of the bed. The bed is continuously backwashed while the filter is in service.

The Tetra Denite system is different in that it is not a continuously backwashed filter. Thus, periodically each of the filters cells is taken offline and completely backwashed to remove the excess biology.

Preliminary sizing indicates that for either system a total filter area of 5,600 ft2 would be required (2 gpm/ft2, all filters in service at 16 mgd). In addition, denitrification sand filters can be very deep on the order 20 to 25 feet depending on each manufacturers system. In general, the Dynasand process is simpler to operate than the Tetra system, but the Tetra system is a more "tried and true" tertiary denitrification process.

FIGURE 5-7
DYNASAND PROCESS FLOW DIAGRAM

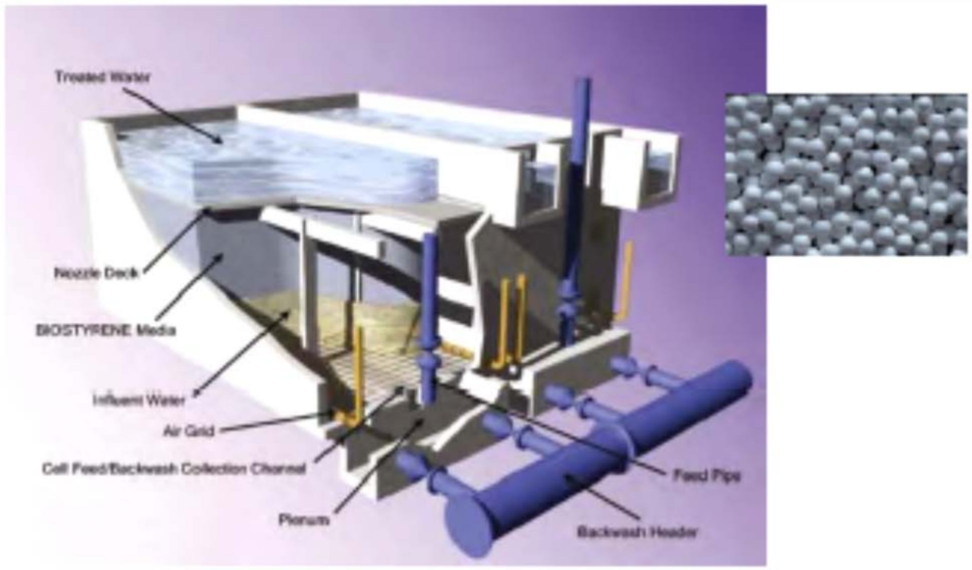


5.3.2.2 Biological Aerated (Anoxic) Filter

Another filtration technology is a biological anoxic filter (BAF) which is shown in Figure 5-8. BAFs use polystyrene balls or other plastic media and can be loaded at higher peak hydraulic rates (8 to 9 gpm/ft2) than the sand filtration system, ultimately reducing the process footprint. The BAF process has proven to be capable of meeting total nitrogen limits of 3 mg/L in full-scale applications. The BAF process has been used successfully at the Cheshire, CT; Southington, CT; and West Warwick, RI wastewater facilities.

The BAF packing is less dense and therefore imposes less head loss than the deep-bed sand filter, which reduces potential pumping energy costs and decreases footprint. The deep bed sand filter, however, captures solids more effectively.

FIGURE 5-8
BAF SCHEMATIC (COURTESY OF KRUGER)



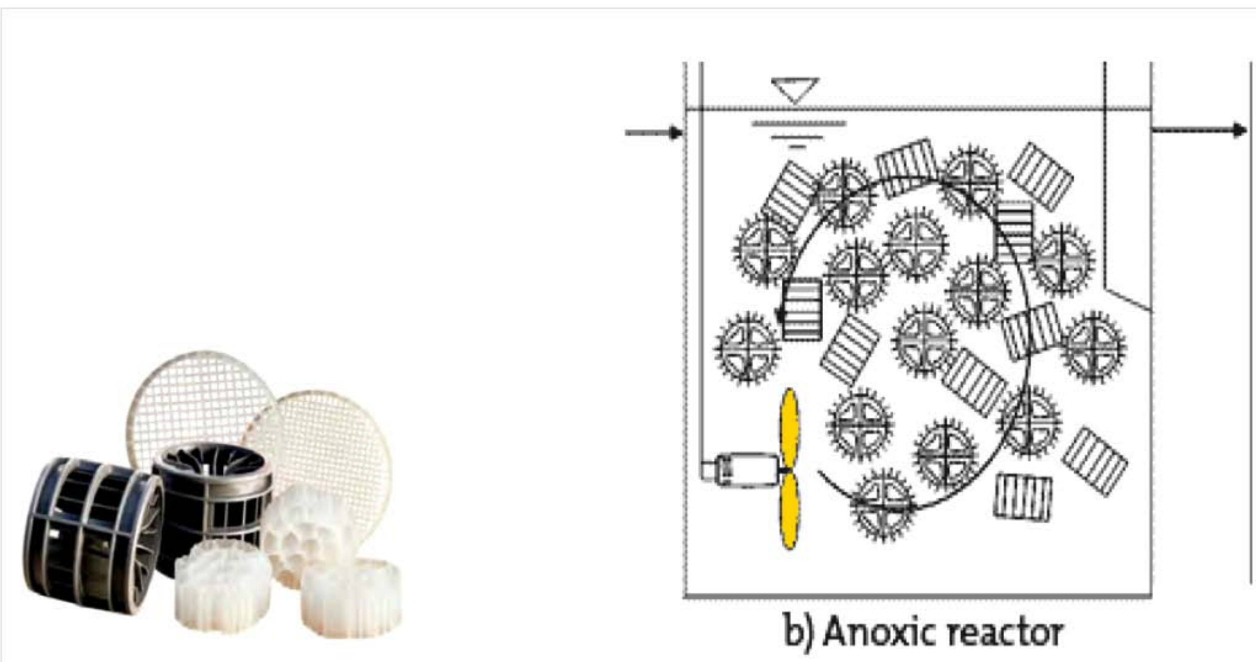
The BAF process operates similar to the Tetra Denite in which different cells are periodically isolated and backwashed to remove excess biological growth. Typically, the BAF process requires fewer filter cells than the sand filter. The BAF process will also occupy roughly half of the area of the denitrification sand filter (2,600 ft2). However, this process is also very deep at approximately 25 to 30 ft. While the BAF process does have some advantages of the smaller footprint including reduced concrete costs, it can be more complicated to operate than the sand filter process. Specifically, this system will require more ancillary equipment and has a more complicated automatic control system.

5.3.2.3 Moving Bed Biofilm Reactor (MBBR)

The Moving Bed Biofilm Reactor (MBBR) consists of small plastic media suspended in a reactor as shown in Figure 5-9. The media are generally coarser and more irregular than that for a BAF. Slow mixing using a propeller mixer is used to suspend the media. An MBBR is a true biofilm process that does not require solids recycle. MBBR media is not intended to filter solids. Therefore, "bumping" and backwashing that are required for a filter are not required with an MBBR. However, a downstream solids removal process is required to complement the MBBR process. At Woonsocket, where a tertiary solids removal mechanism for removal of phosphorus will be required, that process limitation is eliminated (assuming the tertiary phosphorus removal process can handle the expected solids loading levels from the MBBR process). The MBBR process has been used successfully at several locations for the removal of nitrogen including 10 years of successful operation at the South Caboolture WRF (Australia); South Adams County, CO; and Crow Creek, WY. Since the MBBR does not provide for solids removal (via filtration of the wastewater) the headloss across this system will be significantly less than other tertiary denitrification technologies.

The MBBR process typically is installed as three MBBR units in series (i.e., a single train). For redundancy, a complete system usually consists of two or more MBBR trains. The first two units are larger anoxic tanks. The final smaller tank is aerated to eliminate supplemental carbon breakthrough and help "drive off" nitrogen gas. Each tank consists of the small plastic media suspended in the tank. Ancillary components include media retaining screens, mixers, small blowers and supporting instrumentation. A downstream solids removal step is required to remove the effluent TSS and provide for wastage of the excess biomass. As previously discussed, it is assumed that the phosphorus removal step would provide this function, eliminating the need to consider that unit process as part of the screening criteria.

FIGURE 5-9
MBBR REACTOR AND MEDIA ILLUSTRATION (COURTESY OF KALDNES)

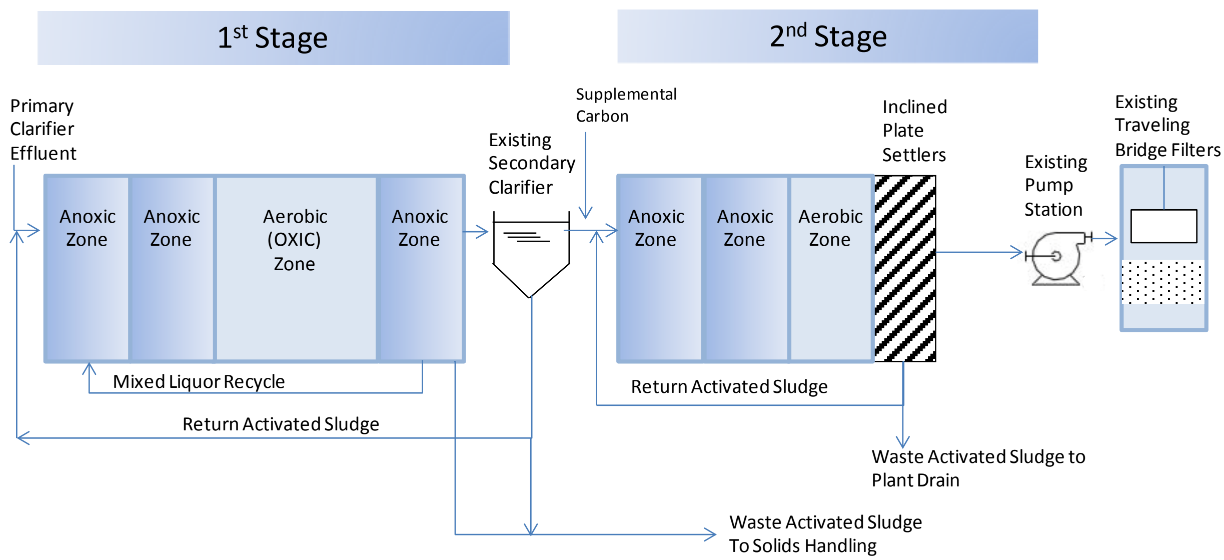


Based on initial sizing criteria, it is assumed that a total MBBR volume of 0.85 mgal would be required. Typically, these systems are designed with a sidewater depth of 20-ft resulting in similar depths to the denitrification sand filter. However, the headloss through the MBBR is significantly lower than the other tertiary denitrification process (approx 1 ft versus 4 ft). At a sidewater depth of 20-ft, the MBBR footprint is approximately 5,700 ft2 or roughly equal to the area of the denitrification sand filter. Unlike the other tertiary denitrification processes, this process does not have a lot of ancillary equipment or complicated automatic backwashing or control schemes. In general the operation of this process (from a sophistication standpoint) is similar to a typical stage of a nitrogen removal secondary process.

5.3.3 Two-Staged Activated Sludge Process

The two stage activated sludge process includes a first stage activated sludge process, which can be achieved by modifying the existing aeration basins, followed by a separate second stage activated sludge process with inclined plate lamella clarifiers for sedimentation. The second stage system would be constructed adjacent to the traveling bridge sand filter building. Figure 5-10, Two-Stage Activated Sludge Process, depicts the proposed configuration.

FIGURE 5-10
TWO-STAGE ACTIVATED SLUDGE PROCESS



Modeling indicates that a second stage suspended growth system can achieve the needed total nitrogen removal in a relatively small volume. Less than an hour of detention time is needed at the design loading rates, but the solids/liquids separation system must be able to handle the higher solids loads from what is essentially a second stage sludge system, similar to the DC Water Blue Plains Facility.

Various high-rate clarification technologies were considered to achieve the removal of solids.

- The bio-Actiflo process was considered, but evaluation of this technology resulted in significant concerns about using it full time and the associated maintenance issues.
- DensaDeg is not an option for this application because it cannot handle high biological solids loads.
- Bio- Mag is another option for this plant, and it was given a detailed evaluation, but it was not cost effective when compared against the last option, inclined plate settling.
- Inclined plate settlers provide a large setting area in a relatively small footprint. For this application, the plate settlers have to be able to handle a solids load with a significant organic fraction.

A two-stage activated sludge process with use of inclined plate settling was evaluated. In the first stage there are no plates; settling is achieved solely by conventional gravity settling. After the conventional gravity settling zone, a large fraction of the solids have dropped out, and then the downstream plate settling section polishes the remaining solids. The plate settlers are designed to handle larger solids loads by using wide plate spacing to improve solids separation. The real benefit to the City of Woonsocket comes from how this approach integrates with the phosphorus removal goals.

A two-stage activated sludge process with plate settlers for solids separation meets the needs for TN removal. This same system is also suitable for phosphorus removal. Chemical, in the form of ferric chloride and polymer, needs to be added to the bioreactor system to achieve the phosphorus removal goals, followed by filtration as a final solids removal step in the process. Not only is adding the ferric chloride to the bioreactor a convenient place, is also provides the following:

- Gives a long contact time between the ferric chloride and the phosphorus, which minimizes chemical usage.

- Builds up an inventory of ferric hydroxide in the bioreactor system that buffers the natural variations in secondary effluent phosphorus, and make the system more stable as compared to a high rate clarifier system.
- Improves settling of the solids entering the clarifier system.

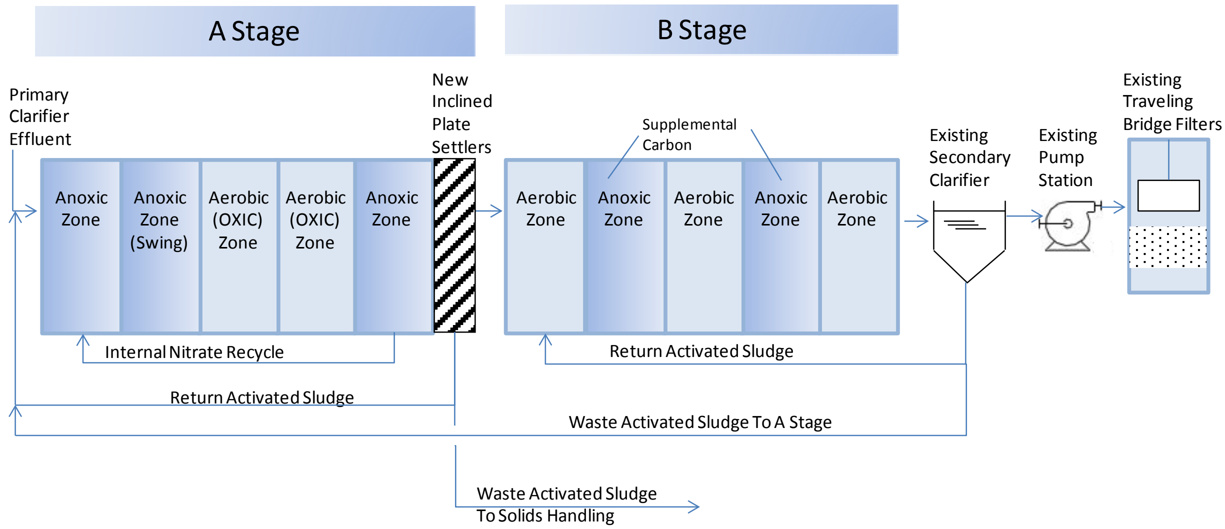
This two-stage activated sludge process was the preferred nitrogen removal process until raw wastewater characterization data, determined through additional sampling campaigns over the past several months, led to modifications of the process configuration. The new, preferred configuration is discussed in Section 5.3.4 below.

5.3.4 Two-Stage Activated Sludge AB Process

The AB Process is a two-stage activated sludge process consisting of a high load “A” stage and a second more conventional “B” stage. The A stage promotes sludge settling and dewatering while the B stage results in BOD removal and nitrification. The A stage operates with a low solid retention time (SRT) of about 3-4 days while stage B has a longer (10 day) SRT to allow the nitrifiers in the sludge to reproduce. The two-stage process is a continuous, self-seeding process since the nitrifiers are continuously added to the first stage from the second stage.

The physical configuration of this process is depicted in Figure 5-11, Two-stage Activated Sludge AB Process. The existing aeration basins needs to be modified into three “A” stage basins, followed by inclined plate lamella clarifiers within each basin and two “B” stage basins, then followed by the existing secondary clarifiers and filters.

Figure 5-11
Two-Stage Activated Sludge AB Process



This process was included for further analysis because there is experience with this process at other full scale facilities and process modeling indicated it was viable for the Woonsocket WWTF.

5.4 PRELIMINARY SCREENING OF PHOSPHORUS REMOVAL ALTERNATIVES

As discussed previously, a two-stage activated sludge system followed by gravity filtration or a tertiary phosphorus removal process will be required to meet the 0.1 mg/l total phosphorus limit. It is important to note that enhanced biological phosphorus removal can be included in the secondary system to the extent possible to reduce chemical requirements and loading on a tertiary system.

As discussed in Section 4, it appears that some biological phosphorus removal is currently occurring in the MLE activated sludge process, facilitated by the limited internal recycle utilized by the operators. Depending on the final nutrient removal process selected, the existing MLE process could be enhanced to maximize the nitrogen removal performance of the process but some level of biological phosphorus removal would be maintained. These enhancements will be discussed in subsequent sections of this report.

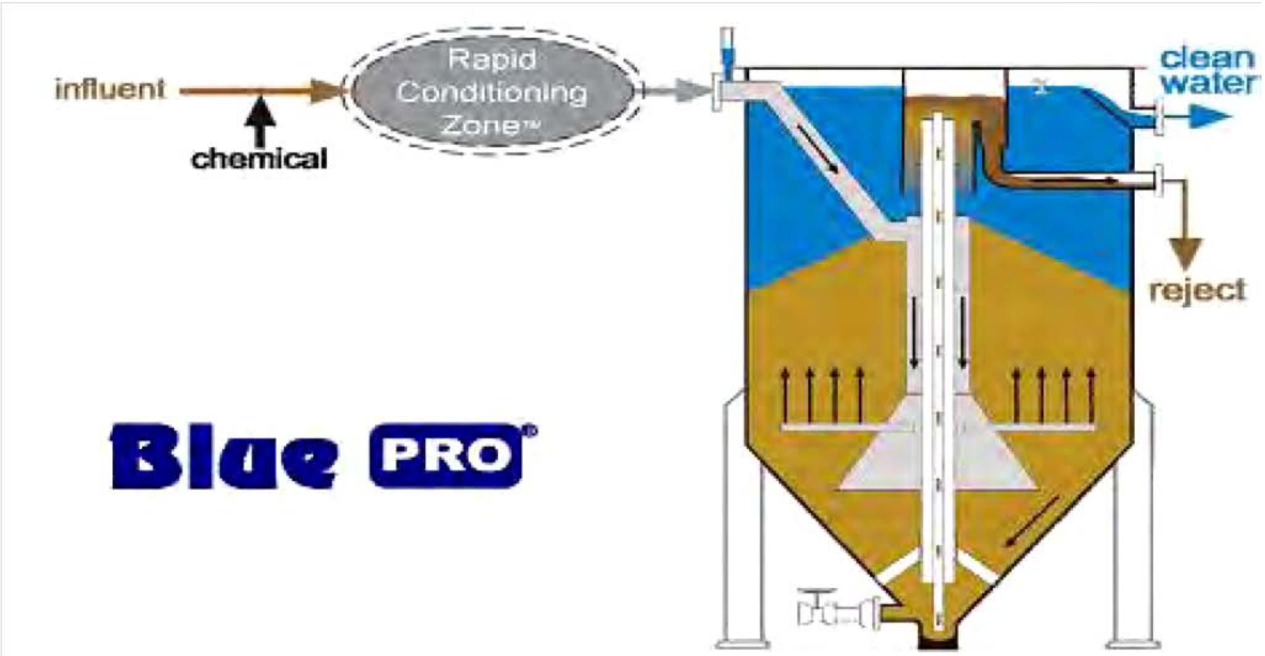
5.4.1 Tertiary Filtration

This category encompasses a broad variety of filters, including conventional gravity filters, moving-bed filters, pulsed-bed filters, travelling-bridge filters, and cloth filters. All of these technologies impose moderate to high head loss, which will require additional effluent pumping. Various manufacturers have made claims that their filtration process is capable of achieving a total phosphorus limit of 0.1 mg/l. Some technologies have been able to demonstrate that ability during limited pilot testing. However, only one of these technologies ("activated" filtration (i.e., Blue-PRO)) has shown repeatable success at these levels.

The BluePRO system is a deep bed, continuous backwashing activated sand filter that operates similarly to the Dynasand filter previously discussed. The significant difference between the BluePRO system and the Dynasand filter is that BluePRO uses a hydrous ferric oxide (HFO) coating on the granular media surface which will adsorb the dissolved phosphorus in the secondary effluent. This removal mechanism is more efficient than coagulation-filtration processes for removing phosphorus and may allow this technology to consistently achieve an effluent total phosphorus concentration of 0.1 mg/l. The HFO coating regenerates continuously on the surface of sand grains in the media. A basic representation of the BluePRO process is shown in Figure 5-12.

This "Activated" Filtration process would not be compatible with the MBBR denitrification process alternative due to the high effluent TSS from the MBBR process (200 to 300 mg/l). From a sizing perspective, the size of this process would be roughly 5,600 sf and approximately 20 to 25 feet deep. At this time, we are not aware of any full scale installations greater than 5 mgd.

FIGURE 5-12
BLUEPRO PROCESS FLOW DIAGRAM

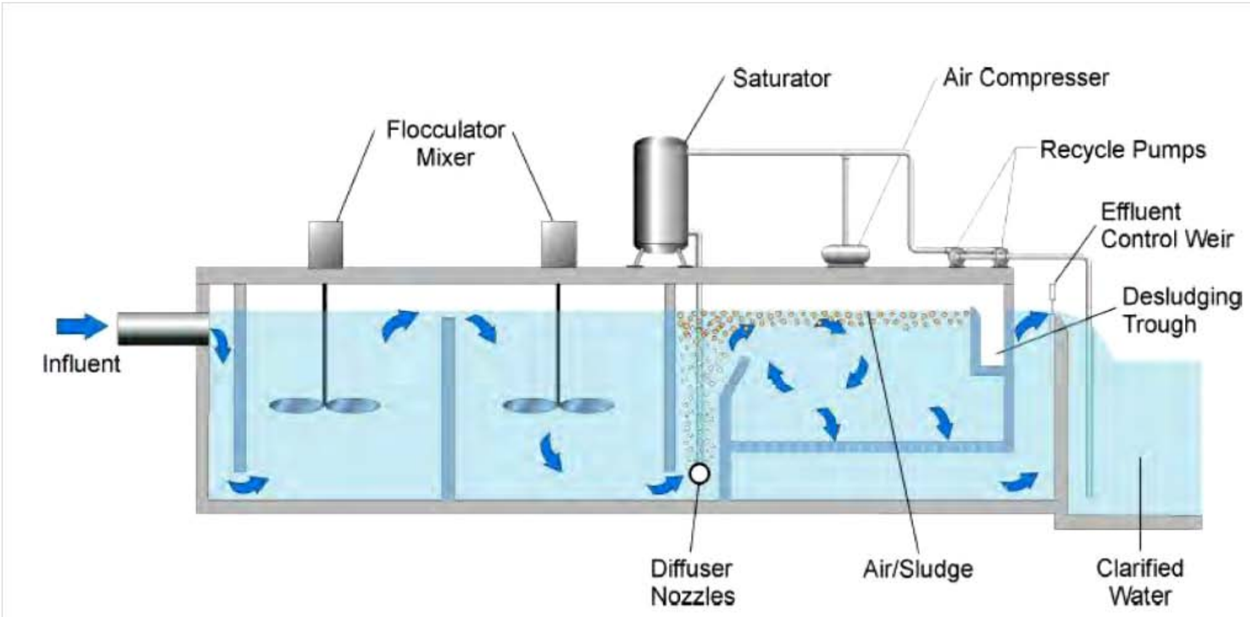


5.4.2 Buoyant Flocculation Process (Dissolved Air Flotation)

The AquaDAF technology is a high rate clarifier system that utilizes Dissolved Air Flotation (DAF) methods. AquaDAF is the trade name of the unit manufactured by IDI, and is utilized as an example for this study; there are other manufacturers of similar DAF style technologies.

In the first step, process influent enters a channel prior to the flocculation zone where a coagulant is added. Next, the coagulated influent undergoes flocculation with variable speed mixers. The AquaDAF flotation process allows for the flocculation step to make larger floc particles that are light, and polymer usage is not required in this step. Flocculated influent is next injected into a saturated air-water recycle stream; the recycled water stream is then depressurized, creating micro-bubbles which disperse into the final flotation zone. The micro-bubbles created in this third step form an air covering and the flocculated particles rise, separating in a reverse method than conventional clarifiers. Solids containing the precipitated phosphorus then accumulate on the surface of the flotation zone where they are removed either hydraulically or mechanically. Figure 5-13 presents a process flow diagram for the AquaDAF system.

FIGURE 5-13
AQUADAF PROCESS FLOW DIAGRAM



The AquaDAF process is currently operating successfully at the Hudson, MA WWTF achieving an effluent total phosphorus concentration less than 0.1 mg/l. At this time, we are not aware of any other full scale installations. Pilot testing has been carried out at other facilities utilizing this process as a solids removal step downstream of an MBBR. Thus, while additional investigation is required, it should be compatible with the MBBR alternative.

While having a slightly larger footprint, the overall capital and operational cost of the DAF technology will be comparable with ballasted flocculation. This process is slightly more complicated to operate than the other processes and can be difficult to "dial-it-in" to the site specific conditions as demonstrated during the start-up period at Hudson and the recent piloting work at the Smithfield, RI facility. This process is also not as robust as the other tertiary phosphorus removal processes in addressing varying influent flow rates (i.e., high flow events). therefore, this process is not recommended for the Woonsocket WWTF.

5.4.3 Ballasted Flocculation

There are several competing ballasted flocculation processes available for use as a tertiary phosphorus removal process. In general, each uses a ballast material (i.e., sand, magnetite) to enhance the settling rate of a conventional chemical floc (i.e., ferric phosphate). The resulting floc displays enhanced settling characteristics, which greatly reduces the area needed for settling the particles. The increased settling rate results in a system footprint that is between 5 and 20 times smaller than conventional clarification systems of similar capacity. For the purposes of the study, Kruger's Actiflo process and Cambridge Water Technology's (CWT) CoMag process were utilized for comparison purposes. However, all of the ballasted flocculation processes in the marketplace should be considered in the future if ballasted flocculation is the selected phosphorus removal technology.

Each of these processes would be compatible with the MBBR denitrification process. The ballasted flocculation process has been used successfully at the Syracuse Metro WWTF (NY) and is currently being constructed/designed for the Jaffrey, NH; Leominster, MA; Manchester, CT; and Smithfield, RI wastewater facilities.

5.4.3.1 Actiflo

This tertiary treatment system is a clarification process (chemical injection, coagulation tank, injection tank, maturation tank, and settling tank) which combines microsand enhanced flocculation and lamella settling (ballasted flocculation). These individual steps are shown in the process flow diagram presented in Figure 5-14.

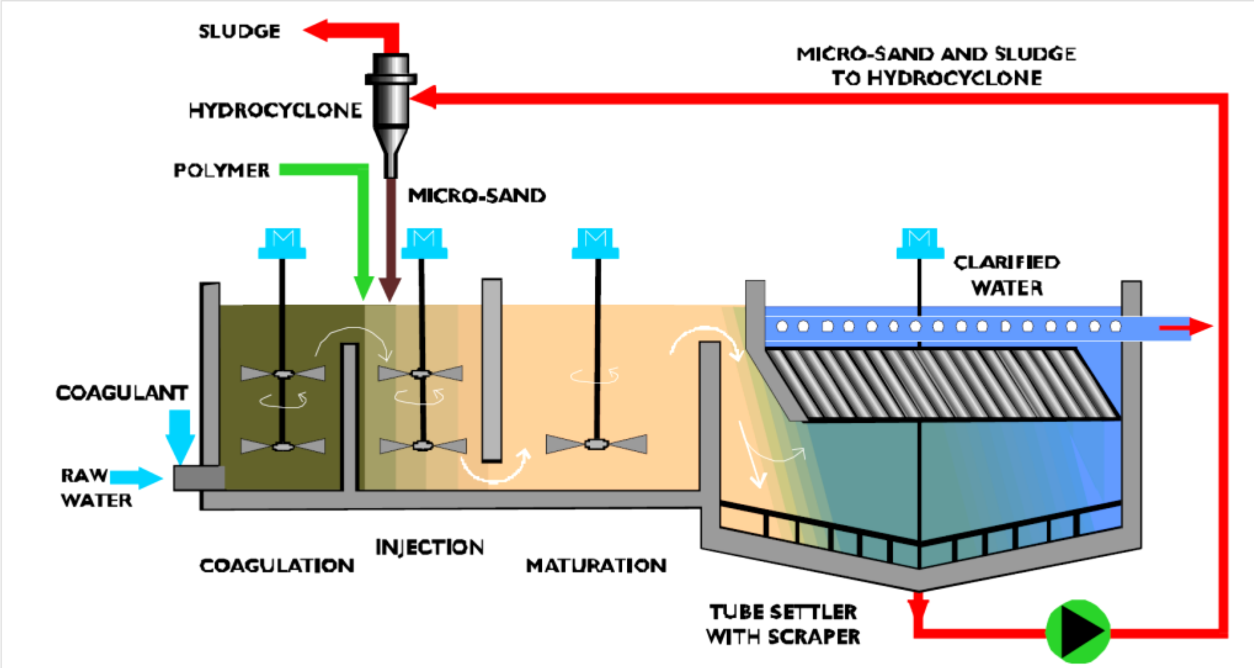
The ACTIFLO tertiary alternative provides the following advantages:

- Effluent TP ~ 0.05 – 0.15 mg/L
- No downtime for backwashing
- Multiple installations
- Small footprint
- Low hydraulic head requirements

The ACTIFLO tertiary alternative has the following disadvantages:

- Power consumption
- Multiple pieces of equipment

FIGURE 5-14
ACTIFLO PROCESS FLOW DIAGRAM



5.4.3.2 CoMag

The CoMag process by CWT is also a ballasted flocculation process; however, it utilizes magnetite instead of microsand as the ballast material. The magnetite mixes with the flocs created by flocculation and coagulation. The flocs containing magnetite are then allowed to settle. The magnetite has a greater specific gravity than sand and results in a higher settling rate than processes that use other ballast material. Therefore, CoMag uses a more conventional clarifier design without the use of lamella settling plates. The settled sludge containing magnetite is then pumped through a shear mixer which removes the particles from the magnetite, and finally, the magnetite is then separated from the slurry composition by using a magnetic drum and returned to the process. A process flow diagram of the CoMag process is presented in Figure 5-15.

The CoMag tertiary alternative provides the following advantages:

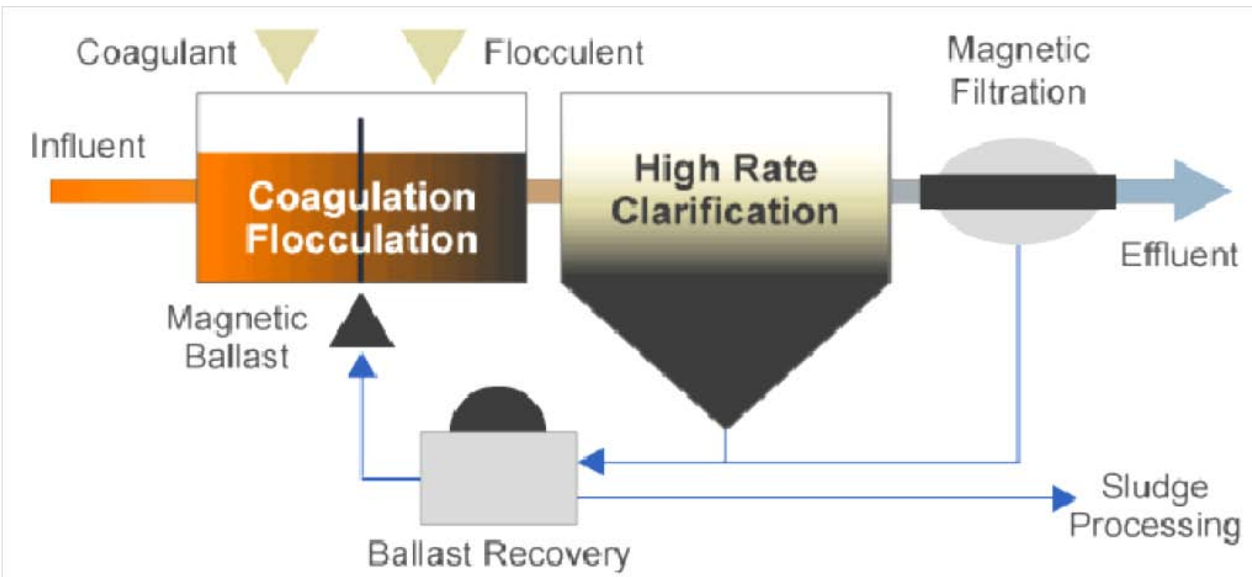
- Effluent TP ~ 0.05 – 0.15 mg/L
- No downtime for backwashing

- Small footprint
- Low hydraulic head requirements

The COMAG tertiary alternative has the following disadvantages:

- Limited Installations
- Medium power consumption

FIGURE 5-15
COMAG PROCESS FLOW DIAGRAM



5.5 PROCESS SCREENING ANALYSIS

The proposed RIPDES effluent limits can be achieved through several alternative process modifications. Specifically, the following alternative processes could be incorporated into the Woonsocket facility to meet an effluent total nitrogen limit of 3.0 mg/l:

1. 4-Stage Bardenpho with Integrated Fixed Film Activated Sludge (IFAS)
2. 4-Stage Bardenpho with Membranes
3. MLE Process with Tertiary Denitrification Filter
4. MLE Process with Tertiary Biological Anoxic Filter

5. MLE Process with Tertiary Moving Bed Biological Reactor

6. Two-stage Activated Sludge Process

7. Two-stage Activated Sludge AB Process

The Woonsocket facility is also required to meet an effluent total phosphorus limit of 0.1 mg/l.

5.5.1 Preliminary Cost Analysis

A preliminary cost analysis for each alternative was conducted based on recent projects and/or proposals for other similarly sized facilities. The estimated costs are not reflective of detailed analysis of site specific conditions at the Woonsocket facility, but provide a preliminary cost comparison between alternative processes for comparative screening purposes. The costs presented do not include all of the project cost items that would be common between the alternatives (e.g., required scope items such as influent screening, methanol feed, electrical, etc.), and hence they are not a complete cost estimate. Table 5-4 summarizes the expected capital cost for each of the five process alternatives. (More detailed overall program costs are presented in Section 9 – Plan Selection.)

TABLE 5-4
PRELIMINARY CAPITAL COST ANALYSIS

		Nitrogen Removal Facilities			Intermediate Pump Station	Ballasted Flocculation Facilities	Total Estimated Cost
	Alternative	Screening Facility	Aeration Tanks	New Facilities			
1	4-Stage Bardenpho w/ IFAS	N/A	$5.0 M	$10.0 M	$5.0 M	$15M	$35.0 M
2	4-Stage Bardenpho w/ Membranes	$7.0 M	$5.0 M	$32.0 M	N/A	N/A	$44.0 M
3	MLE w/ Denitrification Filter	N/A	$1.0 M	$18.0 M	$5.0 M	$15M	$39.0 M
4	MLE w/ Biological Anoxic Filter	N/A	$1.0 M	$12.0 M	$5.0 M	$15M	$33.0 M
5	MLE w/ Moving Bed Biological Reactor	N/A	$1.0 M	$10.0 M	$5.0 M	$15M	$31.0 M
6	Two-stage Activated Sludge Process	-	-	-	-	-	$36.9M[a]
7	Two stage Activated Sludge AB Process	-	-	-	-	-	$36.9M[b]

Notes:

[a] Design-build contracted cost

[b] Cost savings are being evaluated

The preliminary cost analysis was prepared to determine the order of magnitude capital cost for the different alternatives for screening purposes. One of the key assumptions used to develop the capital cost of the Screening Facility is as follows:

- An influent screening upgrade will be required for each of the alternatives due to process consideration as well as the age and condition of the existing headworks facility.
- This upgrade is anticipated to result in an influent screening facility capable of screening all of the influent wastewater via a 3/8" or 1/4" mechanical screen.
  - For this screening analysis, it is assumed that this would be accomplished by replacement of the existing influent screening facility with a new influent screening facility.
  - The above costs and analysis assumes that Synagro would provide separate screening of all of the solids handling recycles.
- The membrane process will require additional screening (typically with a 3 or 4 mm band screen) facility. Thus, a second stage influent screening facility will be required downstream of the first stage screen facility (a single 3 or 4 mm band screen facility in-lieu of 2 separate facilities is not recommended). The estimated construction cost of $7.0M was assumed for this additional second stage facility based on a recent cost estimate for the proposed Norwalk WPCF membrane upgrade.

A preliminary review of the operational costs for each process was also conducted. This operational cost review focused on large operational cost components that would provide additional credibility to eliminate one or more of the five short-listed processes. A more detailed O&M cost analysis was conducted as part of the evaluation of the selected nutrient removal technologies following the screening analysis. In general, each process alternative will require similar levels of supplemental carbon, coagulant for the precipitation of phosphorus, sampling requirements, etc. Therefore, it was determined that the major differentiating operational cost was electricity. Table 5-5 summarizes the preliminary additional electrical consumption for each of the five process alternatives versus the current activated sludge MLE process. The estimated annual operating expense is based on $0.098/kW-hr for electrical demand and $3.86/kW-hr for the demand charge.

TABLE 5-5
PRELIMINARY ADDITIONAL ELECTRICAL CONSUMPTION ANALYSIS

	Alternative	Additional Connected Horsepower (HP)	Annual Power Consumption (kWh/yr)	Annual Operating Expense ($)
1	4-Stage Bardenpho w/ IFAS	450	1,460,000	$143,000
2	4-Stage Bardenpho w/ Membranes	1,200	3,850,000	$500,000
3	MLE w/ Denitrification Filter	300	400,000	$40,000
4	MLE w/ Biological Anoxic Filter	1,300	300,000	$50,000
5	MLE w/ Moving Bed Biological Reactor	200	540,000	$60,000
6	Two-stage Activated Sludge Process	650	- [a]	- [a]
7	Two stage Activated Sludge AB Process	550	- [a]	- [a]

Notes:

[a] See Section 9.5 for Operation contract costs

Key operational issues are identified as follows:

- It has been assumed that each process would require the addition of one full-time employee to assist in operation and maintenance of each process alternative.

- Of the seven alternatives, the 4-stage Bardenpho with membranes will have a significantly higher operational cost than the other alternatives. The additional operational costs are a result of the high RAS flow rates and aeration requirements (for membrane scouring). This alternative will also require significant operational oversight and maintenance activities as well as periodic membrane replacement. Potentially, while not accounted for in this analysis, a second full time employee may be required for this alternative.

- The three tertiary nitrogen removal processes (Alternatives 3, 4 and 5) will have similar electrical operational costs. However, unlike the other tertiary nitrogen removal processes, Alternative 4 (MLE Process with Biological Anoxic Filters) will have a higher connected horsepower requirement due to the size of the air scour blower and backwash pumps.

- The two-stage activated sludge processes has lower power consumption than the other five processes.

- While there are other operational cost differences between the seven process alternatives, none of the remaining items will result in significant difference between the various processes.

5.5.2 Conclusions and Recommendations

The Woonsocket WWTF has several options with respect to achieving the new total nitrogen and total phosphorus limits. Given the stringent nature of the limits and the size and complexity of the facility, only those processes with a proven track record (i.e., other full-scale similar installations) where considered for further evaluation. To achieve the proposed limits, several unit processes in series will be required. A preliminary cost analysis was prepared to determine the order-of-magnitude capital cost for the different alternatives. However, only the two-stage activated sludge AB process was found to have the best operational flexibility and capacity to achieve the long term goals for the facility.

Only one process is recommended for further analysis as follows:

- Two-stage Activated Sludge AB Process

Section 6, Detailed Evaluation of Nutrient Removal Alternatives, presents a more detailed evaluation of the previously described processes and the AB Process.

SECTION 6
DETAILED EVALUATION OF NUTRIENT REMOVAL ALTERNATIVES

6.1 INTRODUCTION

The City of Woonsocket is under a Consent Agreement to achieve more stringent water quality discharge requirements, specifically stricter effluent limits for total nitrogen (TN) and total phosphorus (TP) as defined in Table 5-1. An initial preliminary screening of potential nutrient removal alternatives was conducted in Section 5 in an effort to identify probable treatment concepts and technologies that would be appropriate for long-term use at the WWTF. Given the nature of the WWTF's new effluent limits and the size and complexity of the facility, only four nutrient removal alternatives were retained for further investigation and more detailed analysis. These nutrient removal alternatives are:

- 4-Stage Bardenpho with integrated fixed film activated sludge (IFAS), followed by a ballasted flocculation process.
- MLE Process with a tertiary biological anoxic filter (BAF), followed by a ballasted flocculation process.
- MLE Process with a tertiary moving bed biofilm reactor (MBBR), followed by a ballasted flocculation process.
- Two-Stage Activated Sludge AB Process

In this section, conceptual layouts of the four selected alternatives are presented along with a summary of their basis of design information, advantages/disadvantages, site modification requirements, economic analysis, and associated process recommendations based on design flows and loads to the WWTF. Additional details pertaining to the recommended alternative are presented in Section 9 -Plan Selection.

As part of the recommended nitrogen and phosphorus removal alternatives, there are also several other critical system improvements which need to be included (i.e., influent mechanical screens, upgrade to electrical services, etc). These common items and their associated costs are addressed in subsequent sections of this facility plan amendment.

6.2 TWO-STAGE ACTIVATED SLUDGE PROCESSES FOR NUTRIENT REMOVAL

CH2M HILL's approach to nitrogen and phosphorus removal is to combine the processes and modify existing aeration tankage to meet the new permit requirements, thereby eliminating the need for ballasted flocculation.

A two-stage activated sludge AB process is the recommended nitrogen and phosphorus removal alternative to meet the new effluent total nitrogen limit of 3.0 mg/l and phosphorus limit of 0.1 mg/l. All nitrogen removal alternatives, evaluated in this Section, require the addition of a new tertiary phosphorus removal process following the nitrogen removal process, with the exception of the two-stage activated sludge AB process that is being recommended to be used for both nitrogen and phosphorus removal. It is important to note that enhanced biological phosphorus removal can be employed in the existing secondary system to the extent possible to help reduce chemical requirements. The two-stage activated sludge AB process requires several ancillary components for successful operation including:

1. Chemical Coagulant and Feed System – A small amount of metal salts (i.e., ferric chloride) for the precipitation of phosphorus will be required. It is recommended that the existing chemical facility (for metal salt storage) be used. It is expected that the existing unused storage tanks in the existing Chemical Building would be used. Currently this building houses a sodium hydroxide tank, two ferric chloride tanks, and one polymer storage tank.

2. Polymer System – The two-stage activated sludge AB process will require a small amount of polymer to aid in floc development. A polymer system will be located within the chemical building.

3. Second Stage Sludge Pumping – The second stage (i.e., Stage "B") sludge pumping is required to be located near the existing aeration basins. A portion of the return activated sludge is pumped back from the secondary clarifiers to the second stage basins; the remaining portion of the second stage sludge is recycled to the first stage (i.e., Stage "A") for seeding nitrification.

Table 6-1 summarizes the basis of design/operating characteristics of the two stage activated sludge AB process and compares the original two stage activated sludge process from the original proposal to the revised two stage activated sludge AB process.

TABLE 6-1
BASIS OF DESIGN FOR THE
TWO-STAGE ACTIVATED SLUDGE (AS) PROCESSES

Configuration	**Two-Stage Activated Sludge Process**	**Two-Stage Activated Sludge AB Process**
Aeration Tanks - Stage	**1st Stage**	**A Stage**
No. of Tanks	5	3
Capacity of each Tank (cf) (gal)	152,000 (1,140,000 gallons)	152,534 (1,141,000 gallons)
Average Water Depth (ft)	15	16.5
Unit Volume	cf (gal)	cf (gal)
Zone 1	190,508 (1,425,000) Anoxic	102,780 (768,849) Anoxic
Zone 2	190,508 (1,425,000) Anoxic	102,780 (768,849)Anoxic or Aerobic
Zone 3	190,508 (1,425,000) Aerobic	102,654 (767,903) Aerobic
Zone 4	128,386 (960,326) Aerobic	102,654 (767,903) Aerobic
Zone 5	62,122 (464,674) Anoxic	41,909 (313,495) Anoxic
Total Volume Per Stage Train (cf) (gal)	152,396 (1,140,000)	152,396 (1,141,000)
Total Volume for Stage (cf) (gal)	761,979 (5,700,000)	457,588 (3,423,000)
SRT (days) – annual avg summer	10	4
MLSS (mg/l) – annual avg summer	2000	1,875
Recycle Rate (%)	1000	700
Air Demand (scfm)	8134	7,749
Secondary Treatment – Stage A Settlers		
Settlers, number		3
Settling area, each (sf)		15,012
Return Activated Sludge Rate		50%
Effluent TSS (mg/L)		20

TABLE 6-1
BASIS OF DESIGN FOR THE
TWO-STAGE ACTIVATED SLUDGE (AS) PROCESSES

Configuration	**Two-Stage Activated Sludge Process**	**Two-Stage Activated Sludge AB Process**
Aeration Tanks – Stage	**2nd Stage**	**B Stage**
No. of Tanks	4	2
Capacity of each Tank (cf) (gal)	30,145 (225,485)	126,970 (950,000)
Average Water Depth (ft)	24	15
Unit Volume	cf (gal)	cf (gal)
Zone 1	10,801 (80,795)	62,214.4 (465,396) Aerobic
Zone 2	10,801 (80,795)	62,214 (465,396)Anoxic
Zone 3	10,801 (80,795)	33,812 (252,933) Aerobic
Zone 4	35,376 (264,612)	71,681 (536,217) Anoxic
Zone 5	26,400 (197,472)	24,070 (180,058) Aerobic
Zone 6	26,400 (197,472)	NA
Total Volume Per Stage Train (cf) (gal)	30,143 (225,485)	126,997 (950,000)
Total Volume For Stage (cf) (gal)	120,571 (901, 940)	253,993 (1,900,000)
Total Volume for Both Stages (cf) (gal)	882,425 (6,601,000)	711,582 (5,323,000)
SRT (days)	4	10
MLSS (mg/l) – annual avg summer	1050	1,885
Recycle Rate (%)	NA	NA
Air Demand (scfm)	132	961
Tertiary Filtration		
Filters, number		4
Surface area, each (sf)		1,384
TSS Removal Efficiency (%)		60%

6.3 NITROGEN REMOVAL ALTERNATIVES

Nitrogen removal alternatives were initially evaluated under design average and maximum month loadings conditions as discussed in Section 5. The sizing of each investigated alternative was based on existing plant performance, BioWin process modeling and Pro2D Process Modeling results are presented in Section 4, Performance of the Secondary Treatment Facilities and Process Model Development and/or recommendations from various equipment system manufacturers and vendors. For the development of the detailed evaluations presented in this Section, more detailed process design and secondary effluent data was provided to each system manufacturer in order to obtain site specific process sizing and further validation of preliminary system design requirements. In all cases actual plant data was appropriately modified to reflect the anticipated future loading projections and account for probable temporary process upset conditions which could occur within the secondary treatment system (i.e., an effluent nitrate value of 8.0 mg/l was provided versus the actual plant values which are closer to 6.0 mg/l).

Once the performance of each alternative was assessed, the capital improvements and operation and maintenance costs associated with each alternative were identified and utilized to develop a more detailed evaluation of the four different alternatives previously discussed.

6.3.1 MLE Process Analysis

The process model described in Section 4 was used to evaluate the performance of the existing MLE nutrient removal process and to predict loading conditions to downstream treatment processes. The process modeling generally indicates that due to the relatively high wastewater temperature entering the facility's aeration tanks (caused by the on-site Synagro Merchant Sludge Incineration operations) the existing MLE process is adequately sized and thus no significant reduction in process performance (i.e., total nitrogen removal efficiency through the existing secondary system) is expected given the projected modest increase in the wastewater flows and loads.

6.3.1.1 MLE Process Analysis – Capacity

The existing activated sludge MLE process (aeration tanks and secondary clarifiers) have sufficient capacity to treat the future design flows and loads under all conditions expect for periods of high influent loading and low temperature (during periods of incinerator shut-down).

Unplanned shutdowns of the incinerator operation for more than 2 to 3 days could cause nitrogen removal performance issues during cold weather periods such as April and May. During average influent loading conditions, process modeling indicates that the existing MLE process can perform successfully during these "shutdown" periods if adequate steps are taken within the activated sludge process (i.e., increase in the aerobic sludge retention time) without a significant reduction in the effluent total nitrogen levels from the activated sludge process.

If high influent loading and low temperatures conditions occur simultaneously, the aerobic volume of the aeration tanks will be in-sufficient to maintain nitrification. To address this issue, plant operations staff will have to aerate the anoxic zone essentially eliminating denitrification (i.e., total nitrogen removal) during this period (as well as increase the aerobic SRT). Once the incinerator comes back on-line or the influent loads are reduced back to normal levels, denitrification could be resumed. During these periods effluent total nitrogen removal levels will be impacted and thus will need be addressed through the selected nitrogen removal process upgrade.

6.3.1.2 MLE Process Analysis – Nutrient Removal Performance

As described in Section 4, modifications to the existing aeration system are recommended. Currently, the internal recycle is not utilized by plant operations staff. Discussion with plant staff indicated that this is due to the high levels of dissolved oxygen within the aeration tanks. Utilization of the internal recycle will be required as the flows and loads increase in order to achieve effective nitrogen removal.

Process modeling indicated that at the design flows and loads, the MLE process can achieve an effluent nitrate/nitrite concentration between 4.0 and 6.0 mg/l. Thus, the conditions evaluated for the three nitrogen process alternatives described below and the resulting infrastructure improvements are conservative.

Operating the internal recycle pump will reduce the potential for biological phosphorus removal and will ultimately increase the amount of phosphorus leaving the secondary system. Process modeling indicated that with the internal recycle fully operational, the secondary effluent total phosphorus may be as high as 2.5 mg/l, which is a significant increase over current levels. Ultimately, the increased secondary effluent phosphorus level may be necessary for the successful operation of a tertiary denitrification process.

Alternatively, the existing MLE process could be reconfigured to achieve both biological nitrogen and phosphorus removal. In all cases, the new process would produce slightly greater total nitrogen values than the MLE process. However, the new process would reduce the effluent total phosphorus. Process modeling was conducted and it was determined that the existing process could be retrofitted to an A2O process to achieve biological nitrogen and phosphorus removal. The A2O process would maintain similar effluent total nitrogen levels as the existing MLE Process, but the effluent total phosphorus concentration would be approximately 1.0 mg/l lower. It should be noted that the reduced phosphorus level would not result in changes to the capital cost of the proposed tertiary phosphorus removal process (the tertiary phosphorus removal processes size is dictated by the design flow rate and not the amount of phosphorus that needs to be removed). Reducing the secondary effluent total phosphorus will reduce the amount of coagulant required for the tertiary phosphorus removal process.

Preliminary estimates indicate that a 1.0 mg/l reduction would save approximately $25K/yr in coagulant cost. At a 4% interest rate over 20 years, this results in an equivalent capital project of $337,000. Preliminary, it is not anticipated that the chemical savings achieved by the A2O process would offset the debt retirement to upgrade the existing MLE process.

6.3.2 Four-Stage Bardenpho Process w/ IFAS followed by Ballasted Flocculation Process

The four-stage Bardenpho process (a combined denitrification system) has been used successfully to meet a total nitrogen limit of 3.0 mg/l by incorporating two distinct denitrification conditions (exogenous and endogenous denitrification) to achieve nitrogen removal. The four-stage Bardenpho process provides for biological nitrogen removal but does not provide for enhanced biological phosphorus removal thereby allowing for improved nitrogen removal as the denitrifiers are not competing with the phosphorus accumulating organisms. As appropriate and based on site specific operating conditions, supplemental carbon is often added to the endogenous anoxic zone, if needed, to drive higher denitrification rates.

One of the drawbacks of the four-stage Bardenpho process is the large aeration volume requirements. To overcome this issue, the Integrated Fixed Film Activated Sludge (IFAS) technology can be used to enhance the system's process performance. The IFAS technology has the potential to effectively increase the capacity of the four-stage Bardenpho process without the need for a significant amount of additional aeration tank volume by adding media to the mixed liquor to combine suspended growth (i.e., MLSS) and attached growth (i.e., bacteria attached to the media surface) in the same tank.

As shown in Figure 6-1, the four-stage Bardenpho with the IFAS process consists of an initial anoxic zone followed by subsequent two oxic (i.e., aerated) zones. IFAS media would is added to the second oxic zone to effectively increase the sludge retention time (SRT) of the process. This effectively reduces the required aeration tank volume. Similar to the plant's existing MLE process, an internal recycle pump is used to bring nitrate formed in the aerobic zone back to the initial anoxic zone where it is allowed to come in contact with the influent carbon and is subsequently removed (exogenous denitrification).

Following the aerated zones, a secondary endogenous anoxic zone is provided to remove additional nitrate that was not recycled back to the front anoxic zone in an effort to further reduce the effluent total nitrogen level. Denitrification in the secondary anoxic zone is achieved either through carbon release from cell decay (endogenous denitrification) or through the addition of a supplemental carbon source.

Under this alternative, a small aerated zone is provided at the end of the process to reduce the potential for possible septic or anaerobic conditions forming within the secondary clarifiers, which could result in rising sludge, elevated suspended solids excursions or loss and/or odor concerns.

Based on existing system hydraulics at the treatment plant (i.e. hydraulic profile), it will be necessary to pump the effluent wastewater exiting the Bardenpho process to the new ballasted flocculation process as shown in Figure 6-1.

Advantages:

The four-stage Bardenpho process with IFAS provides the following advantages:

- Enhances the capacity of the existing secondary system while eliminating the need for additional aeration tanks and/or tertiary nitrogen removal tanks.

- Operationally, very similar to the existing MLE process.

- Least amount of new equipment of the three alternative processes.

- Would meet the proposed RIPDES effluent nitrogen limits for the City of Woonsocket.

Disadvantages:

The four-stage Bardenpho process with IFAS has the following disadvantages:

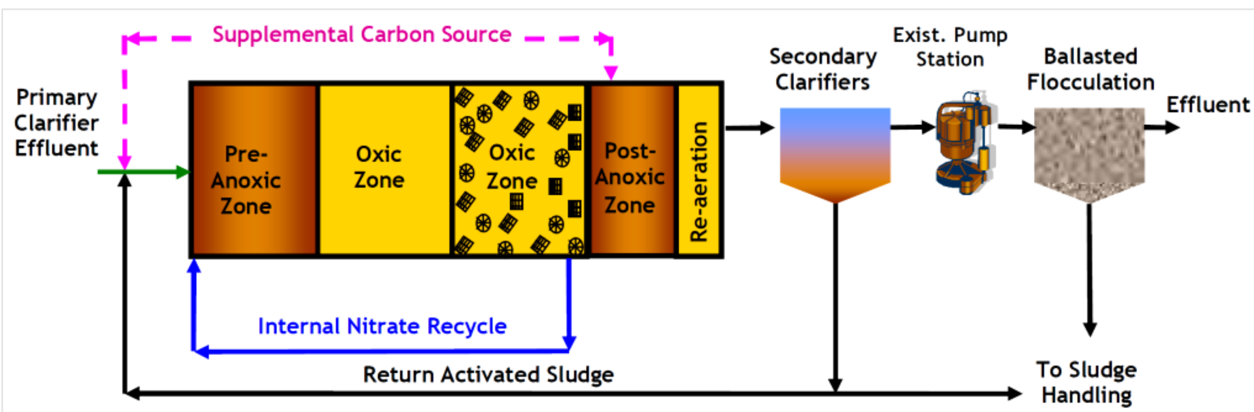
- Additional energy requirements. The IFAS vendors recommend the use of medium-bubble diffused aeration system due to the difficulty of accessing the diffusers.

- Less efficient oxygen transfer rate due to medium-bubble aeration.

- Single stage process. Thus, this process is not as robust as the tertiary processes in achieving the total nitrogen effluent concentration of 3.0 mg/l (that is the other evaluated nutrient removal alternatives).

- Limited operations and process flexibility.

FIGURE 6-1
FOUR-STAGE BARDENPHO WITH IFAS FOLLOWED BY BALLASTED FLOCCULATION PROCESS FLOW DIAGRAM



The four-stage Bardenpho process with IFAS would require several ancillary components for successful operation including:

1. Media.

2. Retaining Screens.

3. Internal recycle pumps.

4. Modify the existing aeration system with medium-bubble air diffuser system.

5. Influent mechanical screening.

6. Submersible mixers.

Table 6-3 summarizes the basis of design/operating characteristics of the four-stage Bardenpho process with IFAS.

TABLE 6-3
BASIS OF DESIGN: FOUR-STAGE BARDENPHO PROCESS WITH IFAS

	Design Operating Conditions
Aeration Tanks	
Number of Trains	6
Reactor Volume	
Pre-Anoxic Reactor, mgal	1.7
IFAS Reactor, mgal	2.3
Post-Anoxic Reactor, mgal	1.2
Re-aeration Reactor, mgal	0.5
Media Information	
Total Bulk Volume, ft^3	96,833
Total Effective Surface Area, ft^2	27,600,000
% Fill of Bio-film Carriers	~55%
Aeration	
Diffuser System	Medium Bubble (4.0 mm orifice)
Residual D.O. Level in Oxic Zone, mg/L	2 – 3
Residual D.O. Level in IFAS Zone, mg/L	4 – 6
Total Air Requirements	27,000 SCFM
Discharge Pressure, psig	6.8

Note: The basis-of-design data information presented in the table is reflective of actual design data provided by a select system manufacturer during the preparation of this Facility Plan Amendment.

The Woonsocket WWTF currently has six (6) 0.95 million gallon (mgal) aeration tanks with a total volume of 5.7 mgal. Under this treatment alternative, the existing aeration tanks would be reconfigured into a four-stage Bardenpho process. Process evaluations to-date conclude that there is adequate aeration tank volume at the WWTF and no additional aeration tanks would be needed to retrofit the secondary treatment process to a Four-Stage Bardenpho Process with IFAS. Under this alternative, each of the existing aeration tanks would be internally reconfigured with the following physical modifications:

- Installation of new intermediate walls to create the anoxic zones.
- The addition of a dedicated mixing system within the anoxic zones.
- An internal recycle system consisting of a submersible propeller pump and associated piping per aeration tank.
- A new medium bubble aeration system. Need to modify the existing fine-bubble aeration system in all aeration tanks to incorporate medium-bubble aeration.

A supplemental carbon facility would be required for the Bardenpho process. Table 6-4 provides a brief summary of the required system components for the four-stage Bardenpho process with IFAS process.

TABLE 6-4
FOUR-STAGE BARDENPHO PROCESS WITH IFAS – REQUIRED COMPONENTS

Location	Equipment Provided Within
Anoxic Zones	1. Submersible mixers
Aerobic Zones	1 Biomass carrier media (i.e. plastic or HDPE carrier elements). 2. 304L stainless steel sieve assemblies. 3. 304L stainless steel medium bubble diffused aeration system. 4. Internal mixed liquor recycle pumps.
Common	1. Instrumentation and Control 2. Field Instruments (level switches, probes, analyzers, etc.)

6.3.3 Modified Ludzack-Ettinger (MLE) Process with Biological Anoxic Filter (BAF) **followed by Ballasted Flocculation Process**

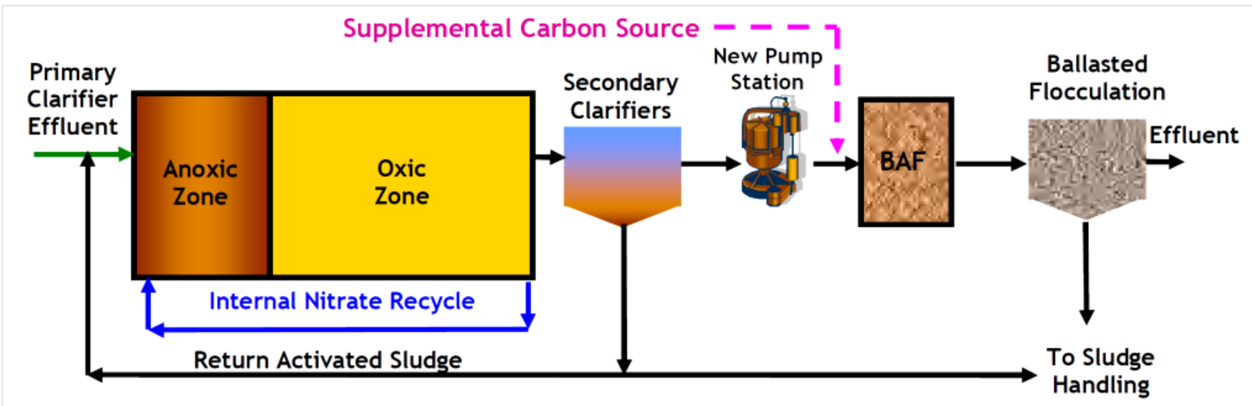
Under this Nitrogen Removal Alternative, it is proposed that the WWTF's existing MLE process be re-used to achieve full nitrification. However, while the existing MLE process is effective in reducing the effluent total nitrogen, the MLE treatment process alone cannot reduce the total nitrogen concentration to the required limits. Generally speaking, the MLE process can reliably reduce the plant's total nitrogen concentration to approximately 6 – 10 mg/L, therefore an additional treatment step is required to reduce the total nitrogen concentration to less than or equal to 3.0 mg/L. In order to meet the facility's new more stringent water quality limits, under this alternative, a new biological anoxic filter (BAF) would be installed downstream of the secondary clarifiers for additional nitrogen reduction (i.e., denitrification), followed by ballasted flocculation for tertiary phosphorus and solids removal.

The biological anoxic filter (BAF) is a biological, submerged filter containing a fixed, dense granular bed with influent wastewater flowing in the upward or downward direction (depending on the manufacturer selected). Nitrified effluent from the MLE process would be pumped to the influent channel of the BAF units and evenly distributed to the cells through a series of weirs. As the wastewater flows through the filter media, attached heterotrophic biomass reduce nitrate in the wastewater to nitrogen gas, which will be released to atmosphere. Like the previously nitrogen removal alternative, the BAF process also requires an external carbon source, such as methanol, to remove the nitrogen. The BAF process also has a relatively large hydraulic headloss requirement. Therefore, a new intermediate pump station would be required following the secondary clarifiers to integrate the proposed BAF unit process into the plant's hydraulic flow path.

Routine backwashing of the BAF filter units is necessary to remove retained solids and maintain a thin, active biofilm. Backwashing is initiated either manually or automatically based upon elapsed time or on reaching a pre-set terminal headloss. The media would be periodically washed by a sequence of air scour, combination air scour/backwash water, and water only rinse steps. Backwashing will typically pump filter effluent water from a separate storage tank using a centrifugal water pump. The air scour blower capacity is based on a maximum air scour rate of 5.9 scfm/ft2 of filter media. The backwash pump would be sized for a maximum backwash water rate of 12.3 gpm/ft2 of filter media. Given the frequency of backwashing, this could result in upwards of a million gallons per day in recycle water that would need to be treated by the existing wastewater facility. Furthermore, given the periodic nature of the backwash, it is almost certain that a flow equalization tank will be required to dampen the effects of the recycle on the main treatment facility.

The proposed BAF system would consist of four (4) new filter units (3 on-duty + 1 standby). Each filter unit would have a surface area of approximately 985 ft2 (32.67 ft x 30.17 ft). Each unit would consist of a concrete tank with monolithic underdrain system, bottom influent and air/water backwash distribution system, granular expanded clay media, gravel support bed, influent pipe, effluent pipe, backwash waste channel with stilling baffle, and all automatic valves.

FIGURE 6-2
MLE PROCESS FOLLOWED BY BAF AND BALLASTED FLOCCULATION PROCESS FLOW DIAGRAM



Advantages:

The Modified Ludzack-Ettinger Process with the BAF Process has the following advantages:

- BAF has smaller footprint than other media or sand filters alternatives.
- Denitrification is separated from other biological processes allowing for enhanced process optimization and operational flexibility.
- Well-proven technology for denitrification with full-scale application achieving TN of 3.0 mg/L.
- Two-stage process, thereby providing enhance operator flexibility and a robust treatment approach.

Disadvantages:

The Modified Ludzack-Ettinger Process with BAF Process has the following disadvantages:

- BAF process requires a relatively deep excavation for installation (25 – 30 ft).
- A new intermediate pump station is needed upstream of the BAF units to overcome the high headloss through the filters (8 – 10 ft).
- The BAF process requires significant instrumentation and controls and other ancillary equipment requirements.
- Requires enhanced operator attention and O&M due to more extensive process equipment and control systems associated with the BAF process.
- The BAF process has multiple electrical equipment systems and equipment thereby requiring an increased connected (electrical) equipment load and electrical power demand.
- Higher backwash volume requirements (i.e., recycle flows) and possible need for a new flow equalization facility. Depending on the quantity of peak backwash volumes it may be necessary to re-size the plant's influent and primary effluent pumping systems.

The Modified Ludzack-Ettinger Process with BAF Process would require several ancillary components for successful operation including:

1. Media.
2. Backwash pumps.
3. Air scour blowers.
4. Air/water backwashing distribution system.
5. Intermediate pump station.
6. Instrumentations and control.
7. Automatic and manual valves.

8. Compressed air system for automatic valves.

9. Flow screens and strainers.

10. Backwash storage and pumping facilities.

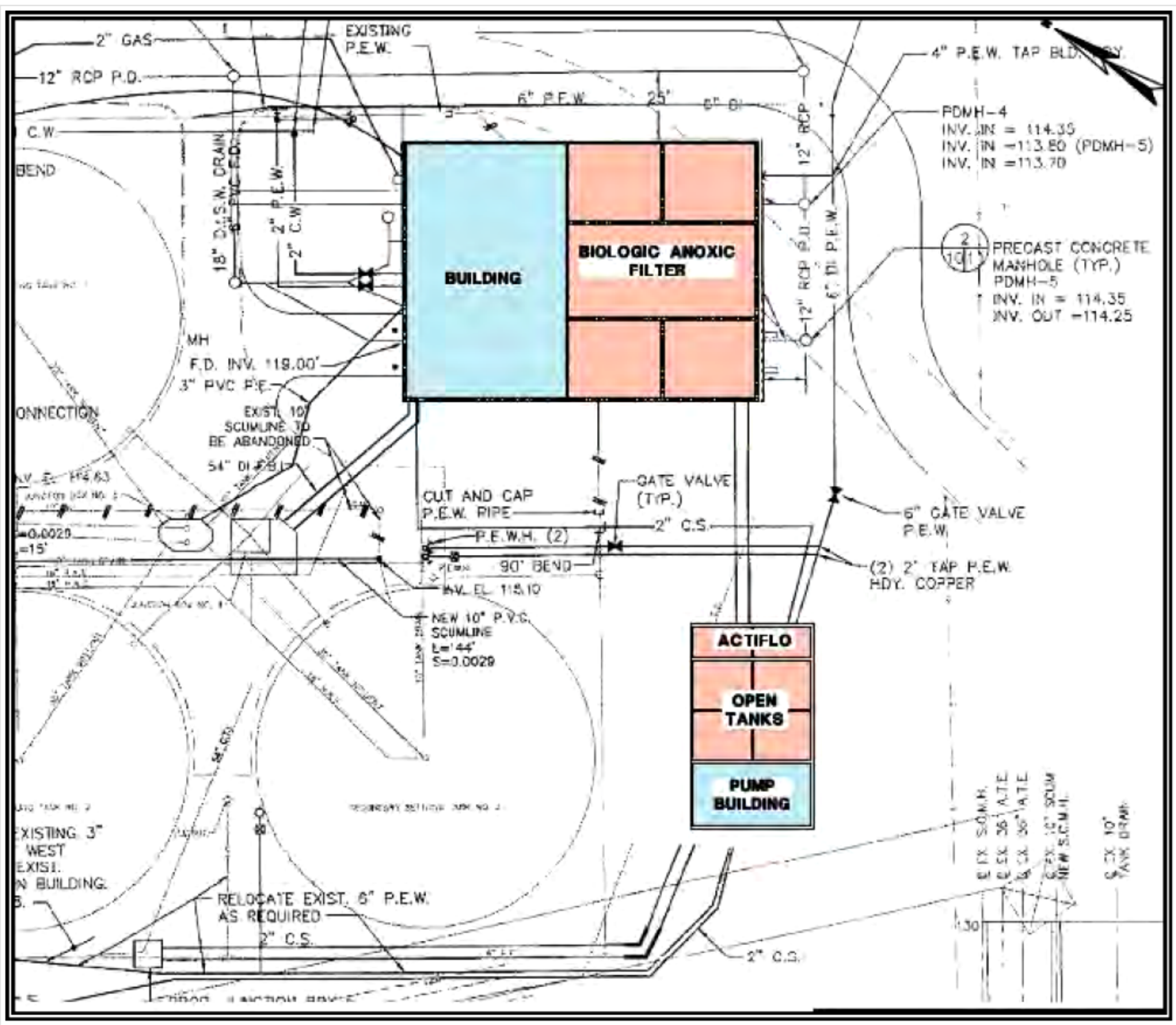
Table 6-5 provides a summary of the basis of design/operating characteristics for the biological anoxic filters (BAF) nitrogen removal alternative.

TABLE 6-5
BASIS OF DESIGN: BIOLOGICAL ANOXIC FILTER

	Design Operating Conditions
Number of Filters	
Total	4
On-Line	3
Module Dimensions, each	
Length, ft	32.67
Width, ft	30.17
Area, ft^2	985
Total Area, ft^2	2,956
BIOLITE™ Media and Support Gravel	
Gravel Volume, ft^3	4,140
Gravel Depth, ft	1
BIOLITE™ Media Volume, ft^3	41,200
BIOLITE™ Media Depth, ft	9.5
Backwash Pumps	
Number of Pumps	2
Type	Centrifugal
TDH, ft	60
Backwash Rate, gpm/ft^2 of Filter Media	12.3
Backwash Pump Capacity, gpm	12,908
Air Scour Blowers	
Number of Blowers	2
Type	Rotary Lube
Air Scour Blower Capacity, SCFM	5,785
Air Scour Rate, SCFM/ft^2 of Filter Media	5.9
Discharge Pressure, psig	11
Compressed Air System	
Number of Compressors	2 (1 On-Line + 1 Standby)
Type	Dual-Head Reciprocating
Compressor Capacity, SCFM	15
Discharge Pressure, psig	100

As previously summarized in section 6.3.1, the facility currently has six (6) 0.95 million gallon (mgal) aeration tanks with a total volume of approximately 5.7 mgal. Under this nitrogen removal alternative it is proposed that the existing aeration tanks would remain unchanged and continue to be used as an MLE process. Process evaluations to-date conclude that there is adequate volume in the existing aeration tanks and no additional aeration tanks would be needed to retrofit the secondary treatment process (i.e. MLE Process) to incorporate use of the BAF Process. Under this treatment alternative, it is envisioned that the existing Effluent Filter Building would be demolished and a new Denitrification Building would be installed as shown in Figure 6-3.

FIGURE 6-3
BAF CONCEPTUAL LAYOUT



Under this alternative a new intermediate pump station would also be required upstream of the proposed BAF unit. In addition, a supplemental carbon facility would be required for the BAF process. Table 6-6 provides a brief summary of the required system components for the BAF process.

TABLE 6-6
BIOLOGICAL ANOXIC FILTER (BAF) PROCESS – REQUIRED COMPONENTS

Location	Equipment Provided Within
Chemical Storage Room	1. Supplemental Carbon System (Pumps)
Blower Room	1. Air Scour Blowers 2. Air Compressor System
Pump Room 1	1. Backwash Pumps 2. Flow meters 3. Instrumentation (analyzers)
Pump Room 2	1. Intermediate lift pumps 2. Flow meters 3. Instrumentation
Common	1. Instrumentation and Control 2. Field Instruments (level switches, probes, analyzers, etc.)

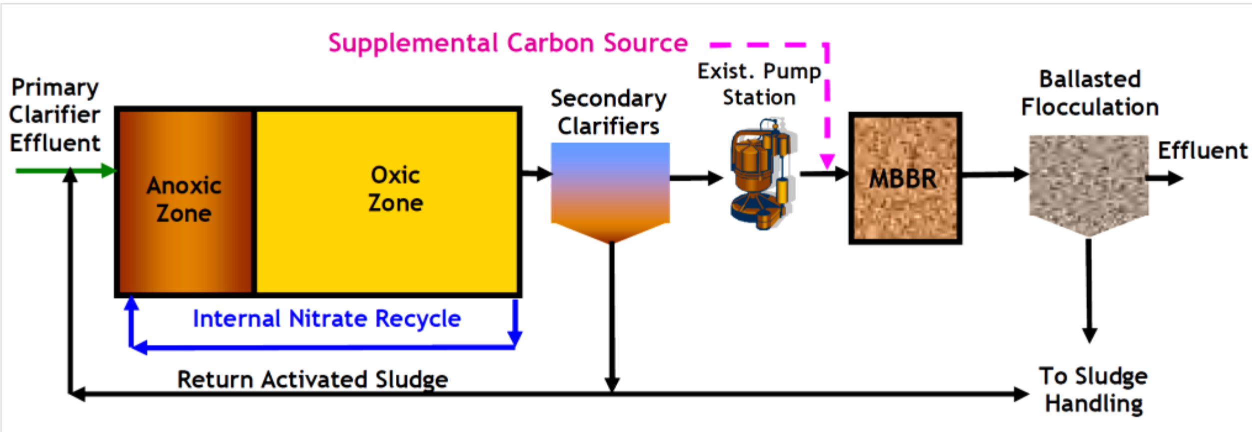
6.3.4 Modified Ludzack-Ettinger (MLE) Process with Moving Bed Biofilm Reactor (MBBR) followed by Ballasted Flocculation Process

Achieving compliance with the new effluent nitrogen limits could also be achieved by the installation of a tertiary MBBR process instead of the BAF units. The MBBR process is very similar to the previously discussed IFAS alternative in that it relies upon the addition of supplemental media to provide a site for the growth of fixed film bacteria. The key difference between the two technologies is that the MBBR process does not rely on suspended growth (i.e., MLSS) to provide additional treatment.

The MBBR process is a continuous flow through process consisting of an initial anoxic zone followed by a small aerobic zone. Media is added to each zone to support the bacterial growth. Supplemental carbon would be required in the anoxic zone to promote denitrification. Mixers would be used to keep the media in suspension in the anoxic zone. Like the other previously explored nitrogen removal alternatives the MBBR process would also require the installation of a downstream tertiary clarification process as shown in Figure 6-4. For this alternative, the ballasted flocculation process would be used for both tertiary phosphorus removal and tertiary solids clarification.

Under this alternative, the MBBR system would consists of three (3) process trains (2 Duty and 1 Standby) with each train having three (3) tertiary denitrification reactors in series and one (1) post-aeration reactor.

FIGURE 6-4
MLE PROCESS FOLLOWED BY MBBR AND BALLASTED FLOCCULATION PROCESS FLOW DIAGRAM



Advantages:

The MLE process followed by the MBBR process has the following advantages:

- Complete flow through process – no backwashing.
- Two-stage process, thereby providing a robust multi-barrier type treatment approach to low level nitrogen removal
- Very little ancillary equipment.
- Simple operation (no complicated control sequence).
- Low head-loss (~2 ft). Under this alternative treatment approach, the existing influent pump station within Effluent Filter Building could be used to lift wastewater into the new MBBR process tanks.
- Denitrification is separated from other biological processes allowing for process optimization and enhanced operational flexibility.

Disadvantages:

The MLE Process followed by the MBBR process has the following disadvantages:

- The ballasted flocculation process has to be operational as long as the MBBR is online.

The following ancillary components would be needed for this alternative for successful operation:

1. Media.
2. Retaining screens.
3. Submersible mixers.
4. Medium-bubble diffused aeration system.

Table 6-7 provides a summary of the basis of design/operating characteristics of the MBBR nitrogen removal alternative.

As previously summarized in Sections 6.3.1 and 6.3.2, the facility currently has six (6) 0.95 million gallon (mgal) aeration tanks, thereby providing a total volume of approximately 5.7 mgal. Under this nitrogen removal alternative it is proposed that the existing aeration tanks remain unchanged and continue to operate using the MLE process. As stated previously, process evaluations to-date conclude that there is ample volume in the existing aeration tanks and no additional aeration tanks would be needed to retrofit the secondary treatment process to incorporate use of the MBBR process. One of the significant advantages of the MBBR process is the limited hydraulic headloss imparted by the media. As such, it was determined that the existing intermediate pump station within the plant's Effluent Filter Building could be retrofitted and re-used for the alternative.

Similar to the BAF treatment alternative explored in Section 6.3.2, under this alternative the existing filter building would be demolished (expect for the building's influent pumping station components) and a series of three (3) new MBBR tanks would be constructed in the vicinity of the current filter building complex (see Figure 6-5 for general layout details). The existing pumping station would be rehabilitated and re-used to lift secondary effluent to the new MBBR system. Table 6-8 summarizes the required components for the MBBR process.

TABLE 6-7
BASIS OF DESIGN: MOVING BED BIOFILM REACTOR (MBBR)

	Design Operating Conditions
Number of Trains	3
Number of Anoxic Zones/Train	3
Number of Oxic Zones/Train	1
Tank Dimensions	
Anoxic Zone	
Length, ft	27
Width, ft	30
SWD, ft	20
Volume, ft^2 (each)	16,200
Total Volume, ft^2	145,800
Oxic Zone	
Length, ft	13
Width, ft	30
SWD, ft	20
Volume, ft^2 (each)	7,800
Total Volume, ft^2	23,400
Media Requirements	
Anoxic Media (K1) Volume, ft^3	76,022
Anoxic Media (K1) Effective Surface Area, ft^2	11,587,350
% Fill of Anoxic Bio film Carriers	~ 45%
Oxic Media (K3) Volume, ft^3	11,617
Oxic Media (K3) Effective Surface Area, ft^2	1,770,431
% Fill of Oxic Biofilm Carriers	~ 48%
Aeration	
Diffuser System	Medium Bubble (4.0 mm orifice)
Residual D.O. Level in Oxic Zone, mg/L	~ 3
Total Air Requirements	483 SCFM
Discharge Pressure, psig	9.0

FIGURE 6-5
MBBR CONCEPTUAL LAYOUT

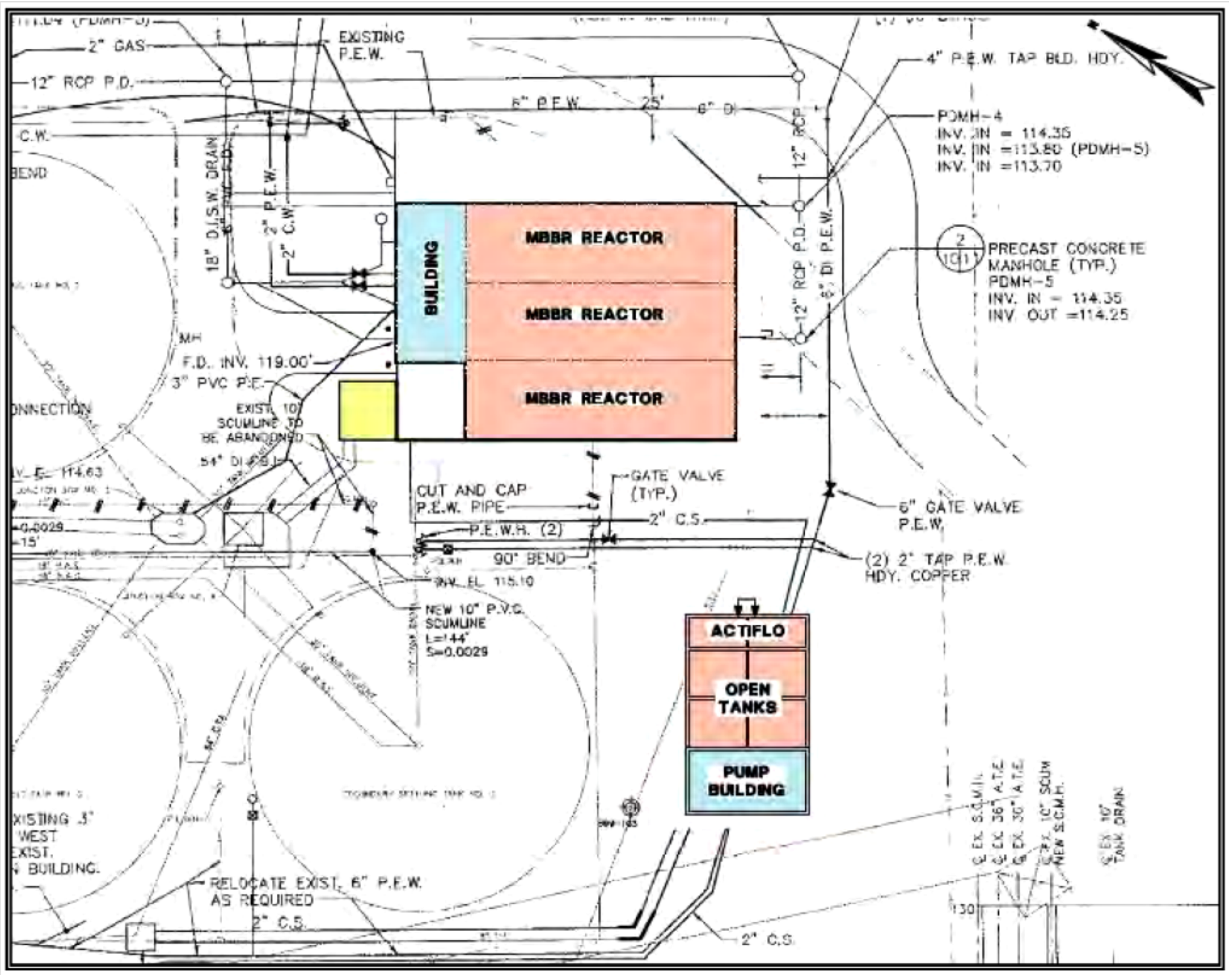


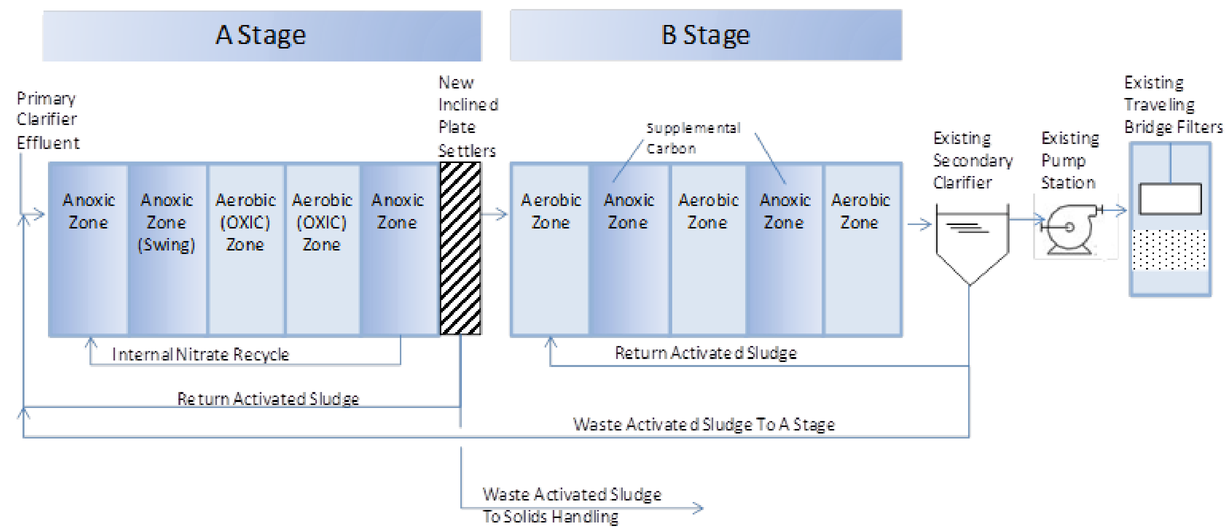
TABLE 6-8
MOVING BED BIOFILM REACTOR (MBBR) PROCESS
REQUIRED COMPONENTS

Location	Equipment Provided Within
Anoxic Zones	1. Twenty seven (27) submersible mixers 2. Biomass carrier media (i.e. plastic or HDPE carrier elements.
Aerobic Zones	1. Biomass carrier media (i.e. plastic or HDPE carrier elements. 2. 304L stainless steel sieve assemblies. 3. 304L stainless steel medium bubble diffused aeration system.
Blower Room	1. Two (2) positive displacement blowers (1 duty + 1 standby)
Supplemental Carbon Room	1. Flow meters 2. Instrumentation (analyzers) 3. Pumps
Common	1. Instrumentation and Control 2. Field Instruments (level switches, probes, analyzers, etc.)

6.3.5 Two-Stage Activated Sludge AB Process followed by Traveling Bridge Sand Filtration

Achieving compliance with the new effluent nitrogen limits could also be achieved by the installation of a two stage activated sludge AB Process followed by the existing traveling bridge sand filters. As discussed in Section 5 this process has several advantages and when qualitatively compared to the two-stage activated sludge process from the original proposal, was found to be more flexible. Figure 6-6 shows the configuration for this process.

FIGURE 6-6
TWO-STAGE ACTIVATED SLUDGE AB PROCESS



The AB process, followed by the traveling bridge filters, has the following advantages:

- Complete flow through process
- Two-stage process, thereby providing a robust multi-barrier type treatment approach to low level nitrogen removal
- Nitrification is seeded in the first stage and protected from washout
- Simple operation (no complicated control sequence).

- Nitrification and denitrification are incorporated into both stages of the AB biological treatment process. This gives greater operational flexibility and the optimization of the process size allows both stages to be located in the same area of the plant.

The AB process, followed by the traveling bridge filters, has the following disadvantages:

- Requires a medium amount of ancillary equipment for second stage RAS/WAS pumping, scum and foam removal, and aeration upgrades.
- More complex construction sequencing due to reusing of existing aeration basins.
- Replacement of the primary effluent pump station is required.

The following ancillary components needed for this alternative for successful operation include:

1. RAS/WAS pumps
2. Primary effluent pumps
3. Submersible mixers
4. Medium-bubble diffused aeration system
5. Aeration blowers

Table 6-9 provides a summary of the basis of design/operating characteristics of the two-stage activated sludge AB process alternative.

TABLE 6-9
BASIS OF DESIGN: TWO-STAGE ACTIVATED SLUDGE AB PROCESS

CONFIGURATION	DESIGN OPERATING CONDITIONS
Aeration Tanks – Stage A	**A Stage**
Number of Tanks	3
Capacity of each Tank (cf) (gal)	152,534 (1,141,000 gallons)
Average Water Depth (ft)	16.5
Unit Volume	cf (gal)
Zone 1	102,780 (768,849) Anoxic
Zone 2	102,780 (768,849)Anoxic or Aerobic
Zone 3	102,654 (767,903) Aerobic
Zone 4	102,654 (767,903) Aerobic
Zone 5	41,908 (313,495) Anoxic
Total Volume per A Stage Train	152,530 (1,141,000)
Total Volume - A Stage	457,589 (3,423,000)
SRT– annual avg summer (days)	4
MLSS – annual avg summer (mg/l)	1,875
Recycle Rate (%)	700
Air Demand (scfm)	7,749
Secondary Treatment – Stage A Settlers	
Settlers, number	3
Settling area, each (sf)	15,012
Return Activated Sludge Rate	50%
Effluent TSS (mg/L)	20
Aeration Tanks – Stage B	**B Stage**
Number of Tanks	2
Capacity of each Tank (cf) (gal)	126,970 (950,000)
Average Water Depth (ft)	15

TABLE 6-9
BASIS OF DESIGN: TWO-STAGE ACTIVATED SLUDGE AB PROCESS

CONFIGURATION	DESIGN OPERATING CONDITIONS
Unit Volume	cf (gal)
Zone 1	62,214 (465,396) Aerobic
Zone 2	62,214 (465,396)Anoxic
Zone 3	33,812 (252,933) Aerobic
Zone 4	71,681 (536,217) Anoxic
Zone 5	24,070 (180,058) Aerobic
Zone 6	NA
Total Volume - B Stage Train	126,997 (950,000)
Total Volume – B Stage	253,993 (1,900,000)
Total Volume A & B Stage	711,582 (5,323,000)
SRT – annual avg summer (days)	10
MLSS– annual avg summer (mg/l)	1,885
Recycle Rate (%)	NA
Air Demand (scfm)	961
Tertiary Filtration	
Filters, number	4
Surface area, each (sf)	1,384
TSS Removal Efficiency (%)	60%

As summarized in Sections 6.3.1 and 6.3.2, the facility currently has six (6) 0.95 million gallon (mgal) aeration tanks, thereby providing a total volume of approximately 5.7 mgal. Under this nitrogen removal alternative it is proposed that a new 0.5 mgal aeration tank be constructed and the existing aeration tanks be reconfigured into two stages with multiple anoxic and aerobic zones. One of the significant advantages of the AB process is that the traveling bridge sand filters are reused.

6.4 COST ANALYSIS OF EXPLORED TOTAL NITROGEN ALTERNATIVES

In order to effectively evaluate and screen the economic impacts of each alternative, separate planning or study level probable cost estimates were developed for: Construction Costs;

Operation and Maintenance Costs; and Total Life-Cycle Costs (Present Worth – 20-year analysis) for each explored alternative treatment scheme.

The planning-level construction cost estimates presented herein were developed using standard cost estimating procedures consistent with industry standards utilizing concept level facility layouts, unit cost information, and planning-level cost curves, as necessary. Industry standard allowances were made for general contractor overhead and profit, undeveloped items and construction contingency. The conceptual cost estimates incorporate the following allowance items:

- 15% General Contractor overhead and profit
- 20% Contingency on un-identified work items and difficulties often associated with facility retrofit work
- 7% Project Inflation allowance (to inflate the presented cost to the mid-point of construction, assumed 3-4% per year)
- 18% Technical Services allowance (final design and construction administration services)

The opinion of probable construction cost information for the first three process presented herein were developed by Wright-Pierce is in 2011 dollars and is based on Engineering News Record (ENR) Construction Cost Index for January 2011. Escalations to future dates can be made with this date and associated ENR index as a basis. These planning level cost estimates have been developed primarily for evaluating alternative solutions and are generally reliable for determining the relative costs of various options. Many factors arise during final design (i.e., foundation conditions, owner-selected features and amenities, code issues, etc.) that cannot be definitively identified and estimated at this time. These factors are typically covered by the allowances described above. However, the allowance may not be adequate for all circumstances.

6.4.1 Opinion of Probable Capital Costs

Table 6-10 summarizes the estimated budgetary capital costs needed to implement each of the explored nitrogen removal process alternatives. It should be noted that the capital costs estimates in Table 6-10 were generated solely for the purpose of screening the explored nitrogen removal alternatives being evaluated in this section only. The costs presented in this section do not include the total costs associated with the required ballasted flocculation process as the cost of this unit process would be the same for all explored alternatives. In addition, these costs do not represent the total construction or program costs that will be associated with facility wide nitrogen, phosphorus or other facility improvements recommended in this Facility Plan (these program costs will be presented in Section 9 – Plan Selection).

TABLE 6-10
ESTIMATED COMPARISON BUDGETARY CAPITAL COSTS

Process Configuration	Cost
Four-Stage Bardenpho Process with IFAS Alternative	\$16.0 M[a]
Modified Ludzack-Ettinger Process + BAF Alternative	\$18.9 M[a]
Modified Ludzack-Ettinger Process + MBBR Alternative	\$15.4 M[a]
Two-Stage Activated Sludge AB Process and Traveling Bridge Filters	\$36.0M[b]

Note:

[a] Cost basis is a conceptual cost estimate for nutrient process upgrades only
[b] Cost basis is a design-build as –bid for total construction cost for nutrient process and other plant upgrades. Savings from using the two-stage AB process are being evaluated

6.4.2 Total Net Present Worth Analysis

The total net present worth for each alternative was calculated based on a 20-year Clean Water Fund (CWF) loan with an interest rate of 4%. Table 6-11 presents the annual differential costs and the total net present worth for each of the explored nitrogen removal alternatives. As stated above, the costs below represent the preliminary capital costs and an opinion of the associated operation and maintenance costs for each explored nitrogen treatment alternative to meet the new nitrogen limits and are not indicative of the total costs associated with constructing the all necessary system improvements at the Woonsocket WWTF as recommended in this plan. In addition, any operational and maintenance component that would be common (similar cost impacts) to each nitrogen process (i.e., methanol costs) have not been included in this initial analysis. The complete operational and maintenance cost analysis will be presented in Section 9 for the recommended process alternative.

TABLE 6-11
NET PRESENT WORTH ESTIMATE

Alternatives	**Four-Stage Bardenpho Process with IFAS**	**MLE Process + BAF**	**MLE Process + MBBR**	**AB Process + Traveling Bridge Filters**
Total Construction Cost	$16,000,000	$18,900,000	$15,400,000	$36,000,000
SRF Loan Rate	4.0%	4.0%	4.0%	- [4]
Loan Term, yr	20	20	20	- [4]
Capital Recovery (A/P, i%, n)	0.06 1	0.06 1	0.06 1	- [4]
Annual Debt Payment	**$1,177,000**	**$1,391,000**	**$1,133,000**	- [4]
Operational & Maintenance Cost				
Power Consumption				
Power Consumption (kW-hr/day)	4,078	823	1,463	- [4]
Power Cost ($/kW-hr)	$0.098	$0.098	$0.098	- [4]
Peak Load (kW)	185	426	91	- [4]
Demand Charge ($/kW-hr)	$3.86	$3.86	$3.86	- [4]
Power Cost ($/yr)	**$102,000**	**$30,000**	**$37,000**	- [4]
Maintenance Cost ($/yr)[1]	**$120,000**	**$69,000**	**$96,000**	- [4]
Incremental Power & Maintenance Cost ($/yr)[2]	**$222,000**	**$99,000**	**$133,000**	- [4]
Annual Differential Cost Analysis[3]	**$1,399,000**	**$1,490,000**	**$1,266,000**	- [4]

Notes:
1 Estimated based on 2% of the equipment cost.
2 Estimated power cost plus maintenance cost.
3 Incremental power and maintenance cost plus annual debt payment.
4 See Section 9.5 for contracted operation and maintenance cost.

6.5 CONCLUSIONS AND RECOMMENDATIONS

There are many variables to consider when evaluating different treatment system technologies and determining the most appropriate technology for further consideration. Of primary concern is permit compliance and the ability of the explored treatment technology to readily satisfy the current regulated permit limits. In addition, consideration should also be given to other secondary criteria, such as:

- Prior experience with low-level nitrogen and phosphorus removal applications
- Overall Life Cycle Cost optimization – both in terms of capital and long-term O&M costs (chemicals, energy, sludge, maintenance, etc.)

- Providing and optimum technical solution which allows enhanced process control and operational flexibility.
- Ease of operation and process control
- Use of proven technology and other full-scale successful applications (proven track record at other successful installations)
- Feedback comments from other similar installations (operations, maintenance, performance, etc.)
- Ease of construction and ability to fit within the plant's existing hydraulic profile and available land area.
- Schedule (manufacturing, construction and subsequent technical services)
- Manufacturer's process performance guarantee
- Potential impact of plant hydraulics caused by plant recycle flows
- System complexity – in terms of moving parts, magnitude of equipment requiring O&M, power consumption, chemical use, etc.

Based on the completed treatment technology investigations to-date, and understanding of the individual effectiveness and associated life-cycle costs for each evaluated alternative to reliably satisfy the more stringent permitted effluent nitrogen and phosphorus discharge limits, the two stage activated sludge AB process and existing traveling bridge filters is recommended for use at the Woonsocket WWTF. It is recommended that the City of Woonsocket upgrade their WWTF to incorporate use of the AB Process for the following reasons:

1. This alternative treatment scheme has the lowest capital and annual differential cost (annual debt retirement plus power and maintenance cost) of the four process alternatives evaluated.

2. The facility's nitrogen removal performance will be continually seeded and protected from washout during high flow periods with the two-stage activated sludge AB process configuration, thereby minimizing overall project costs for compliance.

3. The size and proposed location of this alternative will fit within the available land area of the existing facility's fenced property line.

4. Of the four short-listed process alternatives, the AB Process will be the simplest to operate and maintain and is endorsed by the plant's operating staff.

As previously identified, the cost for the selected alternative along with other recommended improvements is presented in Section 9, Plan Selection.

SECTION 7
DEVELOPMENT AND EVALUATION OF ANCILLARY WASTEWATER TREATMENT IMPROVEMENTS

7.1 INTRODUCTION

In the early part of 2011, Wright-Pierce conducted several site visits to the City of Woonsocket Regional Wastewater Treatment Facility (WWTF) to assess the condition of the existing facilities. The information gathered from visual observations during these site visits along with review of facility record drawings, reports and through discussions with WWTF staff was used to develop this facilities planning report including recommendations for equipment replacement and facility upgrades. As noted in Section 6 Detailed Evaluation of Nutrient Removal Alternatives, the recommended nutrient removal alternative requires that several other essential facility systems be upgraded in order to reliably achieve compliance with the facility's more stringent water quality discharge requirements. Upgrades evaluated in this section have been separated into two categories; those required as part of the nutrient removal system upgrades, and those upgrades to be addressed under the WWTF's long-term capital improvements plan.

The City of Woonsocket has had wastewater treatment facilities at this location dating back as early as 1897. Over the years, several significant upgrades have been completed at the WWTF to either upgrade the facility's unit process systems to ensure compliance with changing environmental regulations and standards, or to replace aging infrastructure systems which have exceeded their original design life. The current WWTF configuration is a compilation of upgrades that occurred in 1962, 1974, and several in the early 2000's. Many of the buildings, structures and equipment systems constructed and installed in the 1962 and 1974 upgrades are still in use today. By the time the new permit limits take effect on May 1, 2017, these buildings, structures and equipment systems, will have been in continuous service for approximately 40-50 years. Generally speaking, structures and buildings at wastewater treatment facilities are considered to have a useful design life of 50 years, while individual unit processes and equipment systems are designed for an approximate 20 to 25 year useful life expectancy. Typically, this is considered an appropriate design life and the process equipment systems would not be expected to last through the next 20 year planning period.

In addition to evaluating the existing facilities for their physical and operational condition, an evaluation of building support systems along with a review of compliance with current codes was also conducted and is incorporated within Section 8.

7.2 ALTERNATIVE EVALUATIONS

A summary of the condition of the evaluated facilities organized by unit treatment processes along with recommended improvements follows.

7.2.1 Screening Facilities

Wastewater from the City of Woonsocket and other contributing communities flows to the head of the WWTF where under normal operation the incoming flow is directed to the screening facilities located in the Comminutor Room (lower basement area) of the Operations Building, constructed in 1962. The existing screening facility consists of one catenary-type mechanical bar screen with 1-inch bar spacing, and is located in the main influent channel. The mechanical bar screen is designed to remove rags and coarse debris material from the influent wastewater in order to protect downstream process equipment and treatment systems. Once collected debris is removed from the bar screen it is mechanically conveyed by the screening system and deposited into polypropylene bags, which are lifted out of the Comminutor Room by an overhead electric hoist and ultimately transferred to a metal dumpster for disposal.

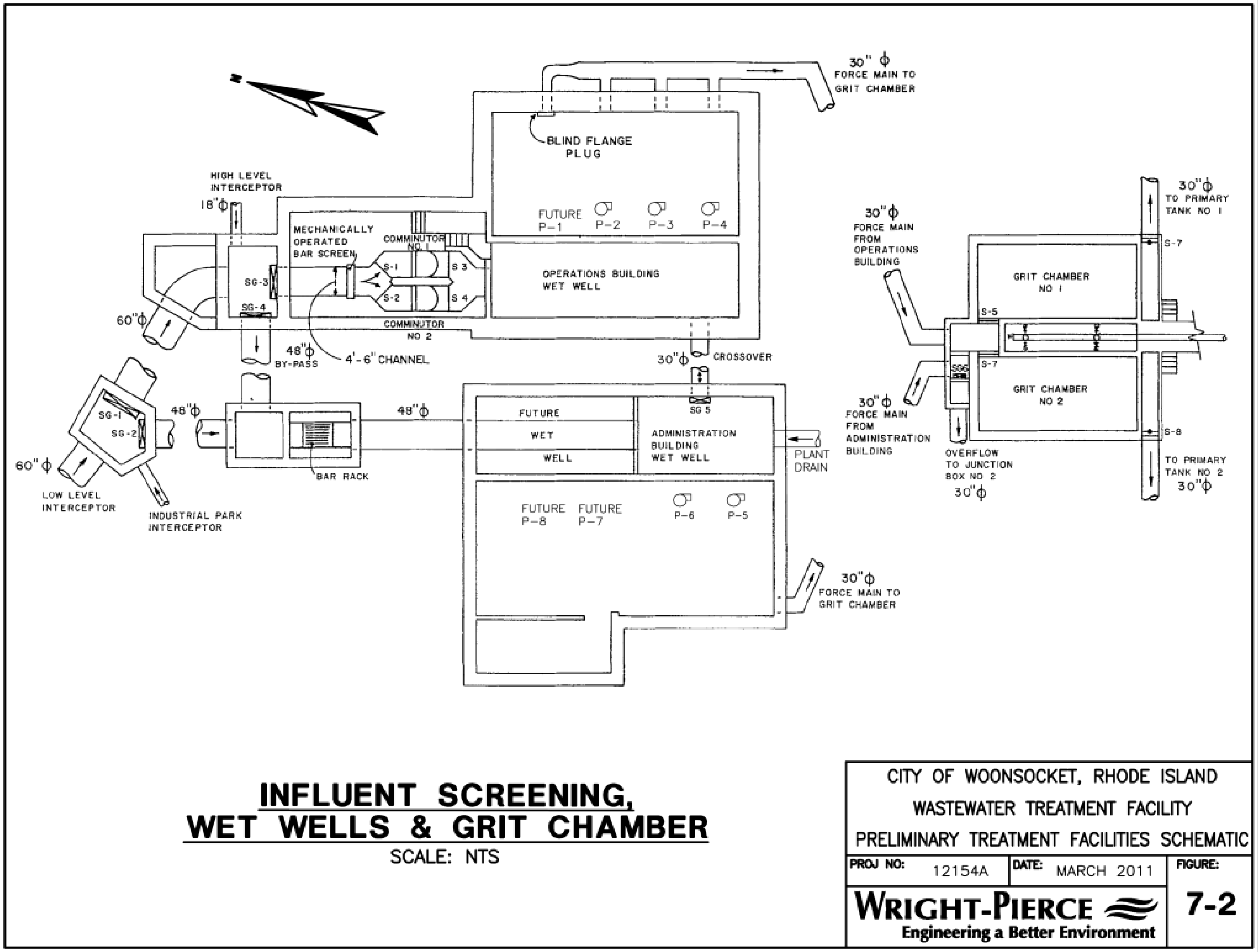
After the screening unit, the main influent channel splits into two channels followed by a manual bar rack with 1-1/2-inch bar spacing. The mechanically cleaned bar screen was installed as part of the 1974 upgrade and the two comminutors were installed as part of the 1962 upgrade and were not functional. The comminutors were recently removed. All influent flow passes through the mechanical bar screen before entering the hydraulically connected downstream wet wells in the Operations and Administration Buildings.

A bypass of the main screening facility was constructed as part of the 1974 upgrade and allows for all influent wastewater flows to be diverted to the Administration Building wet well. A manual bar rack with 1/2-inch bar spacing was built into this bypass. The manual bar rack can only be accessed through the junction box aluminum ladder located in the grassed area outside the Administration Building main entrance. Isolation sluice gates associated with the bypass were installed as part of the 1962 upgrade, but were originally designed to bypass flow to the river. A schematic of the existing main influent screening facilities and wet wells is depicted in Figure 7-2.

FIGURE 7-1
MECHANICAL BAR SCREEN AND COMMINUTORS

As described in previous sections of the Facility Plan, the Administration Building wet well also receives flow from the plant's process recycle line or drain line, which is primarily comprised of the waste streams from the on-site solids handling facility (Synagro). Currently, the recycle flows that discharge into the Administration Building wet well do not receive screening. The flows from the solids handling facility that discharge to the plant drain (recycle flows) also do not receive any screening. Per discussion with plant operations staff, this flow stream often contains significant quantities of rags and similar debris which can cause operational problems in subsequent downstream unit treatment processes.

FIGURE 7-2
PRELIMINARY TREATMENT FACILITIES SCHEMATIC



Conditions Assessment

- The mechanical bar screen within the Comminutor Room is approximately 40 years old and has reached the end of its useful service life. There are signs of significant deterioration and corrosion, resulting in operational problems. Failure of the mechanical bar screen results in back-up of flow into the collection system, and potentially creates a flooding hazard in the Comminutor Room.
- The current means of debris removal from the lower influent screening area is very labor intensive and potentially has concerns due to restricted access to the low basement area, poor ventilation systems, labor intensive removal methods, and general concerns for flooding during elevated wet weather flow events.
- The space constraints associated with the comminutor channels do not allow for the installation of two screenings units to be installed in place of the comminutors.
- The main screening facility (Comminutor Room) bypass isolation sluice gates (SG-3 & SG-4) are inoperable. Physical inspection of the gates was not possible due to their confined space location and hazardous environmental conditions.
- The overall reliability and capabilities of the existing WWTF headworks screening are not adequate for use with the technology selected to meet the new nitrogen and phosphorous limits.

Improvements Alternatives

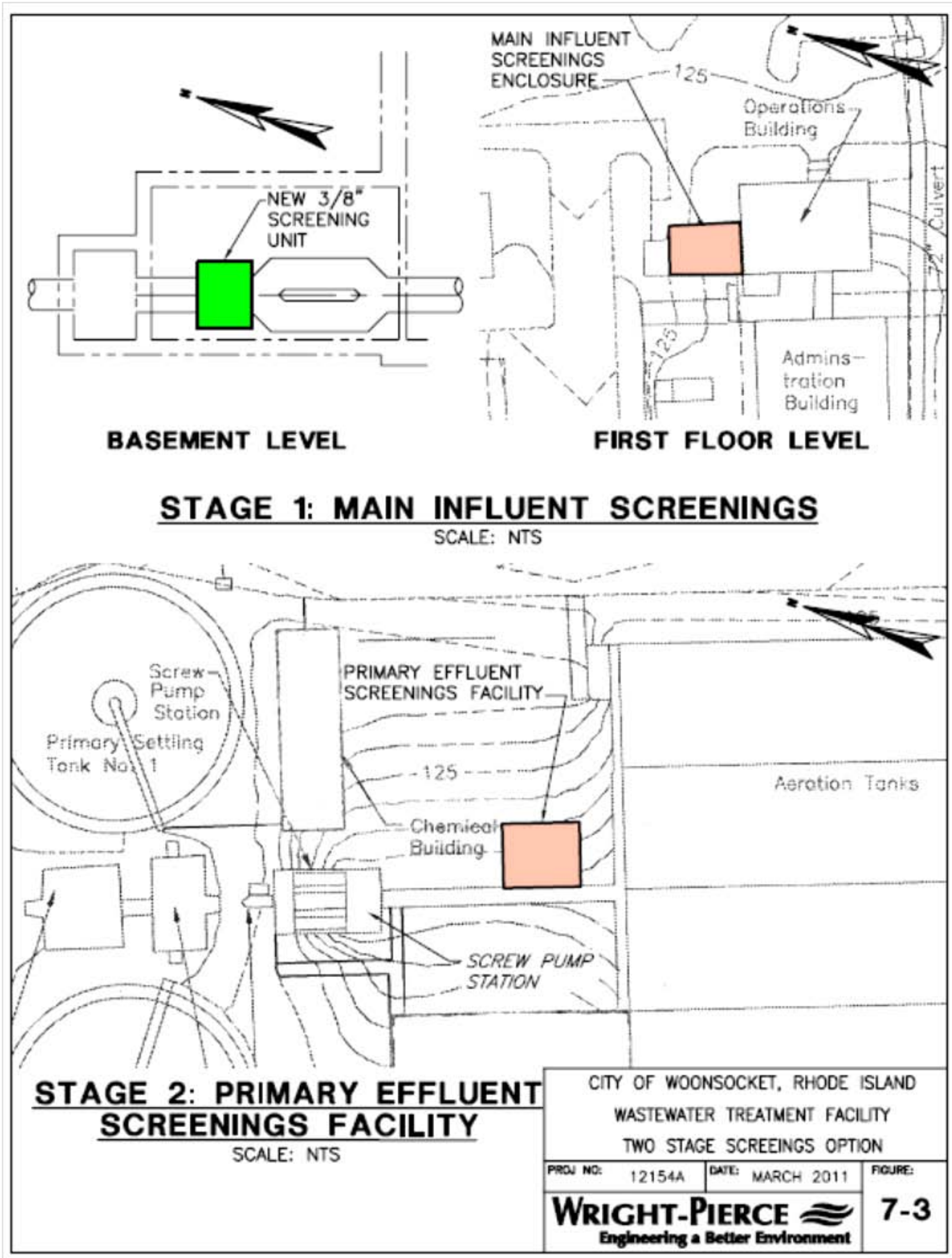
- The existing influent screen at the WWTF needs to be replaced with a new screen with smaller openings to remove more screenings and to protect downstream equipment. Two options for achieving the required level of screening were evaluated, for the March 2011 Draft Facility Plan Amendment, and descriptions of the options are provided below:
  - **Option 1:** This option includes installation of two stages of screening at the WWTF (refer to Figure 7-3). The first stage of screening would be accomplished by replacing the existing mechanical bar rack with a new screening unit with 3/4-inch bar spacing, and the second stage of screening would be implemented through construction of a new screenings facility with two 1/4-inch screening units located downstream of the primary effluent pump station and upstream of the aeration tanks. The replacement main influent screening unit would discharge screenings above-grade to a washer-

compactor unit. This would require that a weather-proof enclosure be constructed around the new screening unit and washer/compactor to reduce odors, provide weather protection and prevent freezing. The second stage of screening would require a new screening weather-proof enclosure housing two 1/4-inch screening units. The washer compactor units and associated channel modifications could be constructed adjacent to the existing channel with limited excavation (± 8-ft).

- **Option 2:** This option would have all of the screening located in the headworks area of the plant (refer to Figure 7-3). This would require the construction of a new main influent screening building adjacent to the existing screening facility which would then become the bypass channel. The new screenings building would house two screening units with a 3/8-inch bar spacing with discharge of screenings above-grade to a washer-compactor unit. This would require that a building be constructed around the new screening units and washer/compactors to reduce odors, provide weather protection and prevent freezing. Construction of this option would require extensive excavations (± 25- ft) to match existing channel and wet well inverts elevations.

As part of the recommended plan described in Section 9, it will be necessary for the Solids Handling Facilities (Synagro) to implement adequate screening of all waste lines that discharge to the WWTF. The RIDEM has placed requirements on Synagro that requires the installation of a screenings unit on their centrate discharge flows to prevent rags and other large debris from entering the plant's recycle line/drain.

FIGURE 7-3
TWO STAGE SCREENING OPTION



- The Comminutor Room (lower basement area) is subject to flooding during extreme wet weather flows and related storm events. Any selected equipment system that is to remain in this area must be suited for a periodic submersible environment (or moist, wet environment subject to flooding). It is recommended that all electrical and instrumentation devices within the lower screening room area be upgraded and/or replaced to prevent possible damage from flooding and be protected from the hazardous moist corrosive environment.

- A new screenings unit will allow for the elimination of the 1-1/2- inch bar screen.
- Upgrades associated with the screening facilities are required and need to be included as part of the nutrient removal upgrade project.

7.2.1.1 Planned Influent Screening Capital Improvements

The planned capital improvements include removing the coarse bar screen and installing a new fine screen with 3/8-inch openings. The screen will be located in the channel in the lower level of the Operations Building and will lift the screenings from the channel, above the existing screenings channel top slab. The screen will be provided with a washer/ compactor. The screen and washer/compactor will be housed in a new screenings building, to be located above the influent channel area. A new odor control system will be provided to treat air ventilated from the screening facility and Operations Building and Administration Building wet wells.

7.2.2 Raw Influent and Recycle Flow Pumping Facilities

The raw influent wet well and associated pumps are located in the basement of the Operations Building which was originally constructed in 1962. As shown in Figure 7-2, there are three raw influent pumps which were installed as part of the 1962 upgrade and there is provisions for a fourth pump. The recycle wet well and associated pumps are located in the basement of the Administration Building constructed in 1974. There are two recycle pumps; one of which was relocated from the raw influent wet well and original to the 1962 upgrade, and one that was installed as part of the 1974 upgrade. The Administration Building wet well is divided by a concrete knockout wall and is currently utilizing approximately half the available capacity. The two wet wells are hydraulically connected by a 30-inch pipe with an isolation sluice gate (SG-5) located in the Administration Building wet well. During wet weather events the two wet wells act as one.

The overall peak combined pumping capacity of the influent pumping station is approximately 42 mgd, or 32 mgd with the largest pump out of service. A summary of raw influent pump information is presented in Table 7-1. The variable frequency drives for the raw influent and recycle pumps were replaced in 2005 and should be adequate for the 20-year planning period.

TABLE 7-1
RAW INFLUENT AND RECYCLE FLOW PUMPS INFORMATION

Pump	Location	Capacity	HP	TDH	Description	Installed
P-1	Operations Building	Provisions for future pump are in place.				
P-2	Operations Building	8 mgd	50	28.5 ft	Vert. pedestal mounted non-clog centrifugal	1962
P-3	Operations Building	8 mgd	50	28.5 ft	Vert. pedestal mounted non-clog centrifugal	1962
P-4	Operations Building	8 mgd	50	28.5 ft	Vert. pedestal mounted non-clog centrifugal	1962
P-5	Administration Building	10 mgd	60	31 ft	Vert. pedestal mounted non-clog centrifugal	1974
P-6	Administration Building	8 mgd	50	28.5 ft	Vert. pedestal mounted non-clog centrifugal	1962
P-7	Administration Building	Provisions for future pumps are in place.				
P-8	Administration Building					

The raw influent pumps have a common 30-inch discharge pipe to the grit removal facility installed as part of the 1962 upgrade. The recycle pumps also share a common 30-inch discharge pipe to the grit facility installed as part of the 1974 upgrade.

The recycle pump room in the Administration Building has a duplex sump pump which was installed as part of the 1974 upgrade. The sump pumps discharge to the recycle pump discharge line. One of the sump pumps was rebuilt in 2007 and new floats were installed.

Conditions Assessment

- The existing combined peak pumping capacity is not adequate to meet the future peak design capacity (raw influent and recycle flows) of the WWTF with the largest pump out of service.

- Each of the influent/recycle pumps are approximately 40-50 years old and have reached the end of their useful lives. Generally speaking, the pumping units have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can reliably operate. Parts are either no longer available or extremely difficult to find. It is uncertain whether the pumping units can reliably and efficiently serve their intended function for the next 20-years.

FIGURE 7-4
RAW INFLUENT AND RECYCLE FLOW PUMPS

- The protective equipment (i.e. bar screen) installed ahead of the pumps are just as old and in poor condition. In recent years they have provided minimal protection to the pumps. Also, the influent/recycle pumps and associated piping have been subject to wear due to their placement upstream of grit removal.

- The raw influent discharge piping within the Operations Building dry well shows significant signs of deterioration, specifically around the wall thimbles. In February of 2011, the Veolia staff had the pipe thickness of the raw influent discharge piping checked. The results showed that in some locations approximately half of the pipe thickness had been worn away on the straight sections of pipe. The majority of the discharge piping, including the elbows and tees, is below-grade outside the building and therefore was not inspected; it can be expected that the pipe thickness has worn considerably more at the discharge piping fittings (i.e. elbows and tees).

- Significant surface corrosion or deterioration was not observed on the recycle flow discharge piping within the Administration Building dry well area. This piping typically does not experience the same wear from pumping raw influent grit as is associated with the Operations Building wet well area.
- The isolation sluice gate (SG-5) associated with the wet well interconnection pipe is reportedly inoperable. This gate was not able to be observed due to safety concerns related to its location (confined space) within the wet well(s).
- The Administration Building recycle pump room sump pumps are approximately 40 years old and have reached the end of their useful lives. Generally speaking, the pumping units have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can reliably operate. Parts are either no longer available or extremely difficult to find. It is uncertain whether the pumping units can reliably serve their intended function for the next 20-year planning period.
- Seal water for the influent/recycle pumps is currently provided only by domestic water.

Improvements Alternatives

- It is recommended that the influent and recycle pumping facilities be improved by adding new reliable pumping capacity in order to adequately handle future design flow conditions with the appropriate pumping unit redundancy. There are several options for achieving this and each option has associated benefits and drawbacks. The most viable options include:
  - **Option 1:** Replacement of three (3) existing 8 mgd pumps in the Operations Building with new 10 mgd pumps and installation of one (1) new 10 mgd pump in the available unused pump location. This would equate to 40 mgd of new reliable pumping capacity. The existing 10 mgd pump in the Administration Building would provide the necessary redundancy plus an additional 8 mgd of emergency pumping capacity by the second pump in the Administration Building. Benefits with this

approach include the ability to reuse existing VFDs, limits risk of having to address code issues in other areas of the plant (i.e. MCC room, generator room, etc.) and reduces cost by salvaging one of the existing pumps. A disadvantage to this approach includes relying on equipment that is well beyond its design life. Based on the overall project needs and recognizing the need to control capital costs, this option is the recommended approach.

- **Option 2:** Replacement of all five (5) existing pumps with new 10 mgd pumps for a total of 50 mgd pumping capacity or 40 mgd capacity with one pump out of service. Benefits with this approach are similar to Option 1 which include the ability to reuse the existing VFDs, limits risk of having to address code issues in other areas of the plant (i.e. MCC room, generator room, etc), but does not include continuing use of the existing pumps.

- **Option 3:** Replace one (1) existing 8 mgd pump with a new 10 mgd pump and install three (3) new 10 mgd pumps in the available unused pump locations. This would equate to 40 mgd of new reliable pumping capacity. The existing 10 mgd pumps in the Operations Building would provide redundancy requirements plus an additional 24 mgd of emergency pumping capacity by the two existing pumps in the Administration Building. Benefits with this approach include the maximization of the influent/recycle pumping capacity. Disadvantages to this approach include added costs associated with required modifications to the Administration Building wet well and required addition of 3 new pump VFD's which in turn could introduce risk of having to address code issues in other areas (Operations Building) unless new code compliant locations could be found, and introduces potential for hydraulically over loading the downstream pump station or other unit processes if safety precautions were over ridden.

- Replace the raw influent discharge piping within the Operations Building dry well, the buried yard piping between the Operations Building dry well and the aerated grit chamber due to the age and condition of the piping.

- Replace the 30" square isolation sluice gate (SG-5) associated with the wet well interconnection pipe to allow for the reliable isolation of the wet wells. Equip the sluice gate with an electric actuator suited for placement and use within the wet well.
- Replace the sump pumps and appurtenances in the dry wells of the Operations Building and the Administration Building.
- Extend the plant water system to provide supply to the Operations Building and the Administration Building to allow for use as seal water for the raw influent and recycle flow pumps.

Planned Capital Improvements

The planned capital improvements include adding a new 8 mgd capacity vertical non-clog influent pump. Magnetic flow meters will be installed in the influent pumps' discharges to more accurately measure influent flow. A magnetic flow meter will also be installed in a meter vault in the yard downstream of the location where the pumped influent and recycle flows combine. Additional improvements to influent and recycle pumping and facility improvements will be provided over a period of time as part of annual maintenance repair and replacement.

7.2.3 Grit Removal Facilities

The grit removal facilities, originally constructed in 1962, consist of two aerated grit chambers that serve to remove grit from the combined raw influent and recycle flows prior to the primary clarifiers. Each chamber is 26-ft long, 12-ft wide and 13-ft deep. Odor control covers were added to the grit chambers as part of the 1991 odor control improvements. Two blowers supply air to the grit chambers, original to the 1962 upgrade, and are located in the Primary Sludge Pump Building. Each of these 15 HP centrifugal blowers can supply 200 cfm at 6 psig. The blowers are constant speed and are not able to be throttled to control the air flow rate to the aerated grit chambers. Originally, settled grit was manually removed by an overhead clamshell bucket hoist system. However, in recent years, due to operational issues the with the clamshell bucket system, settled grit solids have been removed from the grit chamber by means of vactor truck

Adjacent to the Grit Removal Facility is an above-grade 30-ton lime silo, variable feed auger, and vibration delivery system which were installed in 2003. Hydrated lime is added to the grit chamber for various process reasons, including alkalinity and pH control for the activated sludge process, and enhanced chemical precipitation in the downstream primary clarifiers.

Condition Assessment

- The grit removal system as a whole, including concrete tanks, piping, gates, valves, and clam shell and bucket, are approximately 50 years old and have reached the end of their useful life. The clamshell was removed recently due to inoperability.
- The clamshell monorail and associated exposed structural steel exhibits signs of surface corrosion and finish deterioration.
- The aerated grit blowers are approximately 50 years old and have reached the end of their useful life. Generally speaking, the blower units have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can reliably operate. It is uncertain whether the blower units can reliably serve their intended needs for the next 20-year planning period.

FIGURE 7-5
GRIT REMOVAL FACILITIES

- The aerated grit chamber odor control covers have been in place for approximately 23 years and show signs of wear which appears mainly due to exposure to the elements. The lime storage and feed system is approximately 10 years old and was observed to be in good condition with no operational issues being reported. The capacity of the lime silo will be extended from a few days to a few weeks. Due to the changes in the process, lower lime dosages will be required.

Improvements Alternatives

- Install new aerated grit blowers with variable frequency drives. The VFDs will allow for a more efficient and better control of the grit removal process. As flows to the aerated grit chamber increase, the ability to control the air rate to the grit chamber will result in improved capture rates.
- Increase the capacity of the lime silo to allow for additional storage.

Maintenance Improvements

- Grit system modifications are not included in the planned capital improvements work. Grit system maintenance will occur as part of the maintenance repair and replacement work, as required.

7.2.4 Primary Treatment Facilities

The primary treatment facilities, constructed in 1962, consist of two primary clarifiers and a Primary Sludge Pump Station. Each clarifier has a diameter of 90-ft with a sidewall depth of 11 ft. The primary clarifier sludge collection mechanisms were installed as part of the 1962 upgrade. The drive mechanisms were replaced in 2003. At the time of W-P site visits, both primary clarifier units were on-line so it was not possible to conduct visual inspections of the concrete tanks, launder channels, clarifier mechanisms and other associated appurtenances as they were submerged.

As part of an odor control improvements project in 1991, the Primary Clarifier effluent launders were covered to reduce odors; the odor control carbon scrubber unit is housed within the Primary Sludge Pump Station.

The Primary Sludge Pump Station houses two duplex plunger type primary sludge pumps, one double diaphragm type scum pump and one centrifugal dewatering pump. These process pumping systems are housed in the lower basement of the Primary Sludge Pumping Station. The primary sludge pumps are 10 HP duplex plunger pumps; each with a rated capacity of 255 gpm. Primary sludge pump PS-1 was installed, in 2007, to replace the original pump. Primary sludge pump PS-2 was originally installed in 1962 and was recently replaced. The primary clarifier dewatering pump is a 10 HP centrifugal pump installed as part of the 1962 upgrade. The primary scum pump is a 5 hp double diaphragm pump with rated capacity of 100 gpm at 75 TDH and was installed circa 2003 to replace the original scum pump.

The aerated grit chambers blowers and are also housed on the First Floor of the Primary Sludge Pump Station. The blower units and the station's primary electrical gear are all housed in the upper floor area of the building. Electrical systems are described in Section 8.

FIGURE 7-6
PRIMARY CLARIFIERS

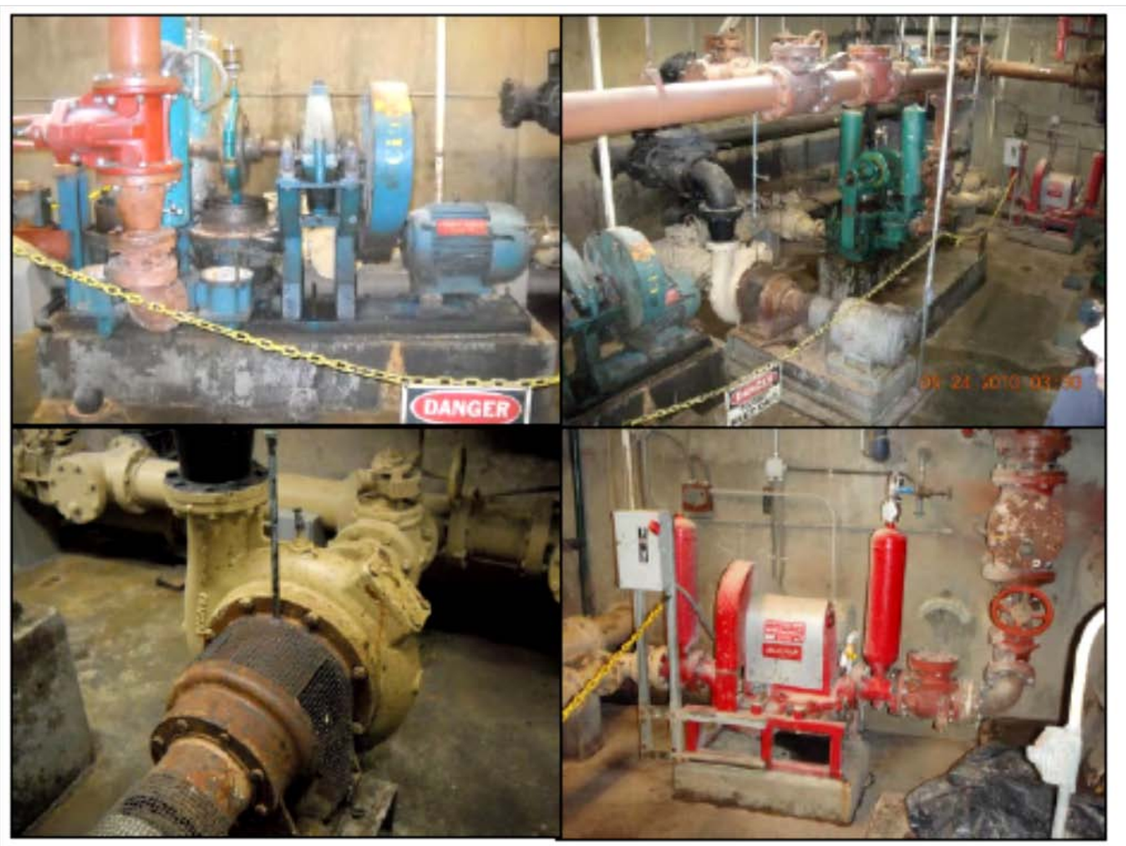
Condition Assessment

- A conditions assessment of the interior portions of the concrete tanks could not be performed during the site visits because the tanks were online. The concrete above the water line is in fair condition with the need for some localized spot repairs.

- The primary clarifier sludge collection mechanisms are approximately 50 years old and have reached the end of their useful lives. Generally speaking, the clarifier mechanisms have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can provide reliably operation. The portions of the mechanisms that could be visually inspected show signs of corrosion and deterioration.

- Primary sludge pump PS-1 is approximately 4 years old, appears to be in good condition, and is reportedly in good working condition. With regular maintenance this pump is expected to continue to operate adequately for much, if not all, of the next design period.

FIGURE 7-7
PRIMARY SLUDGE PUMP STATION



- Primary sludge pump PS-2 was recently replaced.
- The primary scum pump is approximately 8 years old, appears to be in good condition and reportedly is in good working condition. With regular maintenance this pump is expected to continue to operate adequately for much, if not all, of the next design period.
- The dewatering pump for the primary clarifiers is approximately 50 years old and has reached the end of its useful life.

Improvement Alternatives

- Install new sludge collection mechanisms for both primary clarifiers.
- Conduct further evaluations of the Primary Clarifier submerged mechanisms (rake arms, scraper blades, and center drive cage) and exposed turntable drive units during a future temporary shut-down of the Primary Clarifier units in order to assess the condition and remaining useful life of the clarifier mechanisms and system appurtenances. At this time, also evaluate the condition of the units various safety devices (i.e., mercury cut-out switch, etc.) to ensure they are in sound conditions and will operate reliably.
- Conduct further evaluation of submerged wetted concrete tanks surfaces during a scheduled shut-down in order to fully assess the structural condition of the existing clarifier tanks and launders.
- Perform spot repairs of miscellaneous damaged concrete surfaces around the primary clarifier and primary sludge pumping buildings.
- Replace primary sludge pump PS-2 to maintain adequate redundancy.
- Replace the dewatering pump for the primary clarifiers.
- Raise the perimeter concrete side wall of the primary scum well to prevent both flooding and surcharging of the primary scum wells during periods of elevated wet weather flow events.

Maintenance Improvements

- Primary clarifier system modifications are not included in the planned capital improvements work. Primary clarifier system maintenance will occur as part of the maintenance repair and replacement work, as required.

7.2.5 Primary Effluent Pumping Facilities

The Primary Effluent Pumping Station was constructed in 1974. The pump station currently uses two Archimedes type screw pumps each with a capacity of 16 mgd and two submersible pumps each with a capacity of 8 mgd to lift the primary effluent wastewater from Junction Box No. 2 to the primary effluent Parshall flume channel. One of the Archimedes screw pumps is original and one was replaced in-kind, circa 2002; both have had their drives replaced in 2009. The two submersible pumps were also installed, circa 2002, to replace the center screw pump which failed.

The overall rated peak combined pumping capacity of the Primary Effluent Pumping Station is approximately 48 mgd or 32 mgd with one of the larger pump out of service. Individual pump information is presented in Table 7-2.

TABLE 7-2
PRIMARY EFFLUENT PUMPS INFORMATION

Pump	Location	Capacity	HP	TDH/Lif	Description	Installed
SP-1	Screw Pump Station	16 mgd	30	~ 5 ft	Screw Pump	1974
SP-2	Operations Building	8 mgd	30	~ 5 ft	Submersible Pump	2000
SP-3	Operations Building	8 mgd	30	~ 5 ft	Submersible Pump	2000
SP-4	Screw Pump Station	16 mgd	30	~ 5 ft	Screw Pump	1974

FIGURE 7-8
PRIMARY EFFLUENT PUMPING STATION

Conditions Assessment

- **The existing combined peak pumping capacity is not adequate to meet the future peak** WWTF design capacity of 37 mgd with one of the larger pumps (i.e. one screw) out of service.
- At the time of inspections, the Archimedes screw pumps were in operation and it was not possible to inspect the pumping units inclined flights or rotating mechanisms. Therefore the condition of the existing helical screw units is uncertain.
- Reportedly the screw pumps are believed to be in relatively good condition and have no significant operational problems at this time. With regular maintenance these pumps are expected to continue to operate adequately in the near future. However, they have exceeded their original design life and have been in continuous operation over the years and it remains uncertain what their remaining useful life might be. As such, it is recommended that budgetary allowances be made to anticipate their replacement (or partial replacement) over the 20-year planning period.
- The submersible pumping units are approximately 9 years old and are reportedly in fair condition. With regular maintenance these pumps are expected to continue to operate adequately for much if not all next design period.

Planned Capital Improvements

Due to the higher pumping head required for the revised bioreactor configuration, four new 12.6 mgd submersible axial flow pumps will be provided for a total capacity of 38 mgd, with one pump out of service. The pumps will be provided with variable frequency drives. The pumps will discharge flow to each of the first stage basins, with flow meters and flow control valves provided for splitting flow to each basin.

7.2.6 Secondary Treatment Faculties

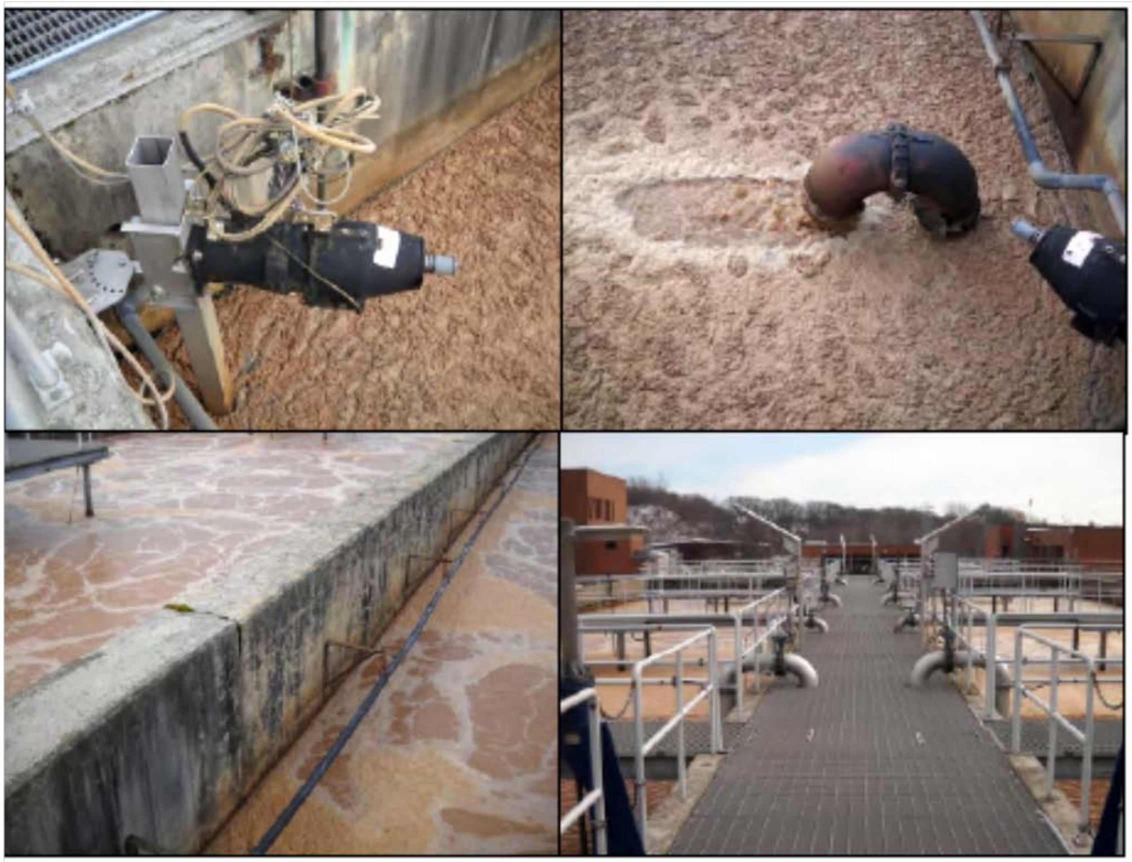
The secondary treatment facilities consist of six aeration tanks, three secondary clarifiers and a Return Sludge Pump Station. The secondary treatment facilities were constructed as part of the 1974 plant upgrade with the exception of aeration tanks 5 and 6 which were added as part of the 2001 upgrade.

Aeration Tanks

There are a total of six (6) aeration tanks, each having a total storage volume of approximately 0.95 million gallons. In the 2000 plant upgrade, the aeration tanks were upgraded to operate in the MLE process configuration. This upgrade include the installation of submersible mixers, nitrate recycle pumps, sluice gate replacements and membrane disc fine bubble diffusers. The stainless steel submersible mixers are each 4.4 HP and rated for 6,180 gpm with a propeller diameter of 23.5-inches. The nitrate recycle pumps no.1 through no. 6 are each 15.4 HP and rated for 5500 gpm at 3.4 TDH.

The four aeration blowers are also housed in the lower basement area of the Return Sludge Pump Station. Two of the centrifugal blowers are original to the 1974 plant upgrade and have a rated capacity of 6500 cfm at 7.5 psig with 300 HP motors. The other two high speed turbo blowers and corresponding VFDs were installed in 2010 using grant funding from National Grid. These new blowers also have a rated capacity of 6500 cfm at 7.5 psig with 300 HP motors.

FIGURE 7-9
AERATION TANKS



Secondary Clarifiers

Each secondary clarifier has a diameter of 110-ft with a sidewall depth of 11-ft. The secondary clarifiers and associated sludge collection mechanisms were installed as part of the 1974 upgrade. The drive mechanisms for SC-1 were replaced in 2001, SC-2 drive mechanisms were replaced in 2001, and the drive mechanisms for SC-3 were replaced in 1999. At the time of site visits, the secondary clarifier units were on-line so it was not possible to conduct visual inspections of the submerged portions of the concrete tanks, launder channels, clarifier mechanisms or their associated appurtenances.

FIGURE 7-10
SECONDARY CLARIFIERS

Return Sludge Pumping Station

The Return Sludge Pumping Station houses four centrifugal return activated sludge (RAS), pumps two centrifugal secondary scum pumps, two centrifugal waste activated sludge (WAS) pumps and one progressive cavity waste activated sludge pump. Individual pump information is presented in below Table 7-3.

FIGURE 7-11
RETURN SLUDGE PUMPING STATION

TABLE 7-3
RAS, WAS, AND SECONDARY SCUM PUMP INFORMATION

Pump	Location	Capacity	HP	TDH	Description	Installed
RAS PUMP-1	Return Sludge Pumping Sta.	3500 gpm	75	50	Vert. pedestal mounted non-clog centrifugal	1974
RAS PUMP-2	Return Sludge Pumping Sta.	3500 gpm	75	50	Vert. pedestal mounted non-clog centrifugal	1974
RAS PUMP-3	Return Sludge Pumping Sta.	3500 gpm	75	50	Vert. pedestal mounted non-clog centrifugal	1974
RAS PUMP-4	Return Sludge Pumping Sta.	3500 gpm	75	50	Vert. pedestal mounted non-clog centrifugal	1974
WAS PUMP-1	Return Sludge Pumping Sta.	350 gpm	25	25	Vert. pedestal mounted non-clog centrifugal	Post 1974
WAS PUMP-2	Return Sludge Pumping Sta.	350 gpm	25	25	Vert. pedestal mounted non-clog centrifugal	Post 1974
WAS PUMP-3	Return Sludge Pumping Sta.	350 gpm	25	35	Progressive cavity	1974
SC SCUM PUMP-1	Return Sludge Pumping Sta.	N/A	N/A	N/A	Vert. pedestal mounted non-clog centrifugal	1974
SC SCUM PUMP-2	Return Sludge Pumping Sta.	N/A	N/A	N/A	Vert. pedestal mounted non-clog centrifugal	1974

Condition Assessment

Aeration Tanks

- Refer to Section 4 and 6 for operational assessments.
- The submersible mixer retrieval mechanisms reportedly function poorly and retrieval of the submersible mixers is difficult and in some cases not possible.
- Air flow control valves are not adequate to control the air distribution into the aeration tanks.

- The exposed concrete surfaces of the aeration tanks were observed to be in sound condition with no known structural concerns or damage observed or reported by plant personnel.

- Two of the aeration blowers are approximately 40 years old, inefficient and have reached the end of their useful design lives. These blowers have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can provide reliable operation.

- The return activated sludge discharge piping at each of the aeration tanks discharges the RAS above the water surface allowing for air entrainment in the anoxic zone.

Secondary Clarifiers

- A CH2M HILL corrosion control engineer visited the treatment plant site. Secondary Clarifier 1 was out of service during the site visit.

- The concrete above the water line appears to be in fair condition with the need for some localized spot repairs.

- The secondary clarifier sludge collection mechanisms are approximately 40 years old and have reached the end of its useful lives. The clarifier mechanisms have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can provide reliably operation. The secondary clarifier mechanism has significant corrosion, with no coating remaining and some of the steel eroded away due to corrosion. The corrosion engineer recommended replacement of the mechanisms within the next 3 years.

Return Activated Sludge Pumping Station

- The four RAS pumps are approximately 40 years old and have reached the end of their useful lives and show signs of deterioration. The RAS pumps have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can provide reliable operation.

- The RAS piping is reportedly in poor condition and several holes have developed. In addition there are limited valves on and inadequate ability to isolate the RAS line. This is a major concern for performing maintenance on the piping. The valves and sludge lines are currently being replaced and should be completed by the end of April 2011.

- The centrifugal WAS pumps are approximately 30 years old and have reached the end of their useful lives and show signs of significant deterioration. The WAS pumps have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can provide reliable operation.

- The progressive cavity WAS pump is approximately 40 years old and has reached the end of its useful life and show signs of significant deterioration. The WAS pumps have been in continuous operation well beyond their original design life and it remains uncertain how much longer they can provide reliable operation.

- The two secondary scum pumps are approximately 40 years old and have reached the end of their useful lives. The secondary scum pumps have become obsolete; the secondary scum well is no longer in service or necessary, as the full scum trough provides adequate scum removal.

Recommended Improvement Alternatives

Aeration Tanks

- The planned capital improvements to the aeration tanks were developed and presented in Section 6, Detailed Evaluation of Nutrient Removal Alternatives.

- The improvements include reconfiguring the basins to provide 3 stage “A” activated sludge basins, followed by lamella plate settling and two stage “B” activated sludge basins, with stage “B” settling provided in the existing secondary clarifiers.

- A new first stage RAS pumping system remove sludge from the lamella plate settlers downstream of the stage “A’ activated sludge basins.

- The basin anoxic zones will be provided with new mixers.

- The basin aerobic zones will be provided with new aeration diffusers, new air piping, flow measurement, and flow control valve modifications, as required, for distribution to the aerated zones.

- Blower modifications are being evaluated and will include additional turbo blower capacity to meet the requirements of the two-stage AB process. The blower system will be able to meet peak week requirements with one blower out of service.

Secondary Clarifiers

The planned capital improvements work does not include work at the secondary clarifiers. Work on these facilities will occur as part of the ongoing maintenance repair and replacement program, as needed.

- A CH2M HILL corrosion engineer made a site visit in November 2012 and provided an assessment of the WWTF condition relative to corrosion. As part of the site visit and evaluation, one of the secondary clarifiers was taken out of service to evaluate the condition of the mechanism and significant amounts of corrosion were identified. As a result of this assessment, mechanism replacement will be prioritized as part of the maintenance program work.

- Conduct further evaluation of submerged wetted concrete tanks surfaces during a scheduled shut-down in order to fully assess the structural condition of the existing clarifier tanks and launders.

- Perform routine spot repairs of miscellaneous damage concrete surfaces around the secondary clarifier and Return Activated Sludge Pumping Station, as required.

Return Activated Sludge Pumping Station

- The RAS pump valves and piping located in the Return Sludge Pumping Station have been recently replaced.

- Replace the RAS piping between the Return Activated Sludge Building and the head of the aeration tanks and install new isolation valves.

- Replace the WAS pumps located in the Return Sludge Pumping Station.

Planned Capital Improvements

- The existing RAS pumps discharge will be modified to discharge RAS to the stage “B” activated sludge basins, as required.

- Other modifications will be addressed as part of the annual maintenance program, as required.

7.2.7 Solids Handling Processes

The solids handling processes were not evaluated as part of this facilities plan amendment. Currently, no changes to the solids handling processes are being proposed. The City of Woonsocket considers the solids handling process to be the responsibility of Synagro per their contract with the City of Woonsocket and is engaged in cooperative working sessions whereby CH2M HILL is the City's designated firm in charge of coordination of the overall WWTF site. As such, CH2M HILL works cooperatively with Synagro and reports to the City whenever corrective actions, per the City – Synagro contract, may be required. The City then directs Synagro to achieve compliance.

7.2.8 Flow Metering

Flow measurement at the WWTF is described in Section 2 of this facility plan. Flow measurement of the wastewater entering at the facility is accomplished by measuring flow at the primary effluent Parshall flume and the final effluent flow meters (i.e. chlorine contact effluent weirs). The WWTF currently has no direct measurement of raw influent flow or internal recycle flows. For reporting purposes, the raw influent flows are assumed to be equal to the final effluent flows (flow in equals flow out). Recycle flows are determined by difference of the flow measurements made at the primary effluent Parshall flume and the final effluent flow meters.

Condition Assessment

- A conditions assessment of the interior portions of the primary effluent Parshall flume could not be performed during the site visits because the flume is in service. Areas of the Parshall flume not submerged were observed to be in fair condition. Cracks were observed on the exterior concrete wall near the level transducer.
- The level transducers associated with the primary effluent Parshall flume are routinely calibrated. The level transducers are reportedly in good working order.
- The level transducers associated with the final effluent chlorine contact weirs are routinely calibrated. The level transducers are reportedly in good working order.

Planned Capital Improvements

The planned capital improvements include installation of magnetic flow meters in the discharge piping from each influent pump. A magnetic flow meter will also be installed in a meter vault, located at the recycle pumps discharge piping.

7.2.9 Odor Control Facilities

The odor control facilities at the WWTF generally consist of treatment of the Administration Building wet well (Recycle Flows), the covered aerated grit chambers, the covered primary clarifier launders and the covered gravity thickener tank. The Administration Building wet well odor control system is a 3,000 cfm chemical wet air scrubber which was installed in 1991. Odor control covers have also been installed inside the wet well to reduce the area the odor control unit has to draw from. An activated carbon odor control system with a capacity of 1,120 cfm is used to control odors from the covered aerated grit cambers and from the covered primary clarifier launders. The aerated grit chamber and primary clarifier odor control system is housed in the Primary Sludge Pump Station and was installed in 1991. The gravity thickener tank odor control system is a chemical wet air scrubber which was installed as part of the 2000 upgrade.

CH2M HILL had an odor control expert visit the site and areas surrounding the plant in November 2012 to make an assessment. As a result of this assessment, an odor sampling program will be completed in the near term and a phased approach to properly prioritizing odors and acting on upgrades will be prepared for the City.

FIGURE 7-12
ODOR CONTROL FACILITIES

Condition Assessment

- The main screening facilities and Operations Building wet well currently do not have odor control facilities.
- The Administration Building wet well odor control system is approximately 23 years old and near the end of its useful life. The effectiveness of this odor control unit is questionable but the facility has not been subject to nuisance odor complaints. The odor control piping inside the wet well was not observed due to confined space entry concerns.
- The aerated grit and primary clarifier odor control system is approximately 23 years old, appears to be in good condition and is reportedly in good working condition. The carbon

has been replaced a few times over its life. The odor control piping appears to be in good condition and no deficiencies were observed.

- The gravity thickener odor control system is approximately 10 years old, appears to be in good condition and is reportedly in good working condition. With regular maintenance this system is expected to continue to operate adequately for much, if not all, of the next design period.

Planned Capital Improvements

The odor control system for the Administration Building wetwell will be replaced with a new high capacity carbon type odor control system to handle the main influent screening facilities, Administration Building wetwell, and the Operations Building wetwell.

SECTION 8
EVALUATION OF BUILDING AND SUPPORT SYSTEMS

8.1 INTRODUCTION

The assessment of the existing building and support systems at the Woonsocket WWTF is summarized in this Section including architectural/structural, heating, ventilation and air conditioning (HVAC), instrumentation and controls, and electrical. The assessment of needs is based on information gathered from site visits, review of facility record drawings, previous studies/reports, and discussions with WWTF operations staff. The assessment includes both improvements that are needed as part of the immediate project to achieve enhanced nitrogen and phosphorus removal, and additional long-term capital improvements for the 20-year planning period.

Table 8-1 summarizes the typical useful life of building and support systems from a recent publication (Daigger, WS&T, 2011). The Woonsocket WWTF has numerous systems that have served the City well, exceeding these useful life guidelines. The overall condition of the facility is acceptable, but significant capital improvement needs were identified as discussed further below. The facility's electrical systems and equipment are the source of the greatest upgrade needs that must be addressed as part of the immediate improvements project. The need to address standby/emergency power is also an immediate priority and the facility's instrumentation and controls will also require a major upgrade.

TABLE 8-1
USEFUL LIFE OF WASTEWATER FACILITY COMPONENTS[A]

Item	Useful Life, years	Comment
Structures	50-100	Lifetime of Concrete Structures (including rehab)
Mechanical Equipment Technology	15-40 10-20	Rotating Equipment Determined by effluent standards and evolution of technology
Electrical Equipment	10-20	Determined by obsolescence
Instrumentation and Controls	5-15	Determined by obsolescence

Note:
[a] Daigger, G.T., A Practitioner's Perspective on the Uses and Future Developments for Wastewater Treatment Modeling, Water Science and Technology, 63.3, 2011.

8.2 ARCHITECTURAL/STRUCTURAL

The existing buildings and tanks were visually inspected in terms of the integrity of the structural and architectural systems. The existing buildings that were evaluated as part of this Facility Plan Amendment included the following:

- Operations Building and Influent Structure
- Administration Building
- Primary Sludge Pumping Station
- Primary Effluent Pump Station
- Chlorination Building
- Return Sludge Pump Station
- Effluent Sand Filter Building

The existing tanks that were evaluated included the following:

- Aerated Grit Facility
- Primary Clarifiers
- Aeration Basins
- Secondary Clarifiers
- Chlorine Contact Tank

In general, the condition of the buildings and tanks was very good in terms of structural integrity, especially given the age. The recommended capital improvements are essentially routine provisions for maximizing the useful life of these facilities. Generally speaking, many of the noted capital improvements will be addressed in a prioritized manner as part of the facility's ongoing preventative and predictive maintenance program for the WWTF over the 20-year planning period.

8.2.1 Operations Building and Influent Structure

The Operations Building and Influent Structure were constructed as part of the 1962 facility upgrade with significant modifications during the mid-1970s upgrade. The Operations Building has an approximate footprint of 50-feet by 40-feet and was constructed with a cast-in-place concrete frame and reinforced CMU face brick infill as shown in Figure 8-1. The roof is comprised of pre-cast concrete "tee" slabs along with insulation and built-up roofing.

The first floor of the building contains the original electrical and control room, an office, and laboratory. The building was modified in the mid-1970's upgrade to provide an access corridor to the Administration Building and a new power distribution and control room. The laboratory was relocated to the Administration Building as part of these changes. The original electrical and control room houses MCC-1, the raw influent pump VFDs, and the main SCADA panel. The basement level of the Operations Building contains the boiler room, a locker room with showers and toilets, and storage space. The sub-basement level is one of two pump rooms for the influent pump station plus the primary wetwell for raw influent. The influent structure and wetwell are accessed from a separate stairway into the influent structure and then to the wetwell. The influent structure is enclosed below-grade with only a concrete slab for the top level.

FIGURE 8-1
OPERATIONS BUILDING AND INFLUENT STRUCTURE

Condition Assessment

- The building is in fair condition overall considering it is approaching 50 years old.
- The built-up roofing, roof drains, and roof flashing appear to be the original and should be replaced.
- Most of the windows and doors are in fair condition. Some of the windows appear to have been replaced in the 1970s improvements.
- The brick and mortar are in fair condition.
- The concrete deck level of the influent structure is in fair condition.

Building Code Assessment

- Egress for the pump room is marginal and may need to be addressed if there are significant modifications.

- There are significant NFPA 820 issues as discussed in Section 8.3.

Recommended Improvements

- Remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system along with rigid insulation.

- The aluminum door that provides access to the headworks/wetwell area does not close properly. Replace this door with a FRP door and frame.

- Seal the exterior brick and mortar to prevent moisture penetration which will cause structural problems.

- Seal the concrete deck of the influent structure to prevent moisture penetration.

8.2.2 Administration Building

The Administration Building was constructed as part of the facility upgrade in the mid-1970's as shown in Figure 8-2. The building has an approximate footprint of 65-feet by 45-feet and has a connecting entrance/corridor to the Operations Building. The superstructure was constructed with a cast-in-place concrete frame and reinforced CMU face brick infill. The roof is a cast-in-place concrete slab with insulation and built-up roofing. The sub-basement floor houses the second influent pump room and the second wetwell. The basement floor contains the boiler room, electrical/generator room, and a locker room/shower area. MCC-1A is located adjacent to the generators in the electrical room. The first floor of the building contains the reception lobby, offices, bathrooms, closets, and storage space. The second floor includes offices, the laboratory, bathrooms, and a kitchen/lunch room.

The Administration Building wetwell does not have an adjacent influent structure, and access is provided through outside vertical ladders to a walkway level above the typical water surface in the wetwell. Prior to the wetwell, there is a small bypass structure with a manual coarse bar rack. This wetwell was only intended to serve as a back-up to the Operations Building wetwell for raw influent. However, the WWTF's main plant drain system discharges directly to the Administration Building wetwell, and there are currently no back-up provisions for discharging the plant drain to the Operations Building wetwell. The Administration Building pump room houses two raw influent pumps with provisions for two more in the future. The wetwell was constructed with a temporary wall so that half of the available wetwell storage volume or space is not used to prevent unnecessary solids accumulation until the two future pumps are installed.

FIGURE 8-2
ADMINISTRATION BUILDING

Condition Assessment

- The building is in fair condition overall.
- The built-up roofing, roof drains, and roof flashing appear to be adequate at the present time, but are anticipated to need replacement over the 20-year planning period.
- Most of the windows and doors are in adequate condition.
- The brick and mortar are in adequate condition.

Building Code Assessment

- Egress for the pump room is marginal and may need to be addressed if there are significant modifications.
- There are significant NFPA 820 issues as discussed in Section 8.3.

Recommended Improvements

- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system.
- Seal the exterior brick and mortar as soon as possible to prevent moisture penetration which will cause structural problems.

8.2.3 Primary Sludge Pump Station

The Primary Sludge Pump Station was constructed during the 1962 WWTF upgrade as shown in Figure 8-3. The building has an approximate footprint of 40-feet by 20-feet and was constructed with a cast-in-place concrete frame and CMU brick infill. The roof is a comprised of pre-cast concrete "tee" slabs along with insulation and built-up roofing. Primary sludge pumps, a dewatering pump, and a scum pump are all located on the basement floor of this building. There are two scum wells located on the east and west sides of the building where scum from both the aerated grit chambers and primary clarifiers is collected and pumped to the sludge holding tank. The main level of this building contains MCC-2, two aerated grit chamber blowers, and an activated carbon odor control system that treats exhaust from the grit chambers and primary clarifier launders.

FIGURE 8-3
PRIMARY SLUDGE PUMP STATION

Condition Assessment

- The building is in fair condition overall considering it is approaching 50 years old.
- The built-up roofing, roof drains, and roof flashing appear to be the original and should be replaced.
- Most of the windows and doors are in fair condition.
- The brick and mortar are in fair condition.

Building Code Assessment

- No building code issues were observed, but there are notable NFPA 820 and NEC issues as discussed in Section 8.3 and 8.5.

Recommended Improvements

- Remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system along with rigid insulation.
- Seal the exterior brick and mortar to prevent moisture penetration which will cause structural problems.

8.2.4 Primary Effluent Pump Station

The Primary Effluent Pump Station as shown in Figure 8-4 was constructed as part of the mid 1970's facility upgrade. The building portion of the structure has an approximate footprint of 20-feet by 10-feet (not including screw pump troughs) and was constructed with a cast-in-place concrete foundation with reinforced CMU walls and face brick infill exterior. The roof is a combination of pre-cast and cast-in-place concrete slabs with insulation and built-up roofing. This building contains MCC-8, electrical control equipment for the screw and submersible pumps, and the primary effluent sampler.

Condition Assessment

- The building is in fair condition overall.
- The built-up roofing, roof drains, and roof flashing appear to be adequate at the present time, but are anticipated to need replacement over the 20-year planning period.
- Most of the windows and doors are in adequate condition.

- The brick and mortar are in adequate condition.

Building Code Assessment

- No building code issues were observed, but there are notable NFPA 820 and NEC concerns as discussed in Section 8.3 and 8.5.

FIGURE 8-4
PRIMARY EFFLUENT PUMP STATION

Recommended Improvements

- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system.

- Seal the exterior brick and mortar as soon as possible to prevent moisture penetration which will cause structural problems.

8.2.5 Chlorination Building

The Chlorination Building was constructed during the mid-1970's upgrade as shown in Figure 8-5. The building has an approximate footprint of 110-feet by 70-feet and was constructed with a cast-in-place concrete frame with CMU walls and brick infill exteriors. The roof is of similar construction to the Primary Effluent Pump Station that includes built-up roofing containing concrete planks, insulation, asphalt, and pre-cast concrete coping. The first floor includes an electrical area, abandoned sludge conditioning system, abandoned chlorine gas system and a sodium bisulfite storage tank as well as a shop and garage for maintenance. The basement is a pump room which contains froth spray pumps, plant effluent water pumps, incinerator scrubber water pumps, and a plant water pump.

FIGURE 8-5
CHLORINATION BUILDING

Condition Assessment

- The building is in fair condition overall.
- The built-up roofing, roof drains, and roof flashing appear to be adequate at the present time, but are anticipated to need replacement over the 20-year planning period.
- Most of the windows and doors are in adequate condition.
- The brick and mortar are in adequate condition.

Building Code Assessment

- No building code issues were observed, but there are some NEC issues with the electrical room as discussed in Section 8.5.

Recommended Improvements

- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system.

- Seal the exterior brick and mortar as soon as possible to prevent moisture penetration which will cause structural problems.

8.2.6 Return Sludge Pump Station

The Return Sludge Pump Station was also constructed during the mid-1970's upgrade as shown in Figure 8-6. The building has an approximate footprint of 110-feet by 70-feet and was constructed with a cast-in-place concrete frame with CMU brick infill. The roof is of similar construction to the Primary Effluent Pump Station and Chlorination Building with built-up roofing containing concrete planks, insulation, asphalt, and pre-cast concrete coping. The first floor contains electrical and control rooms, locker room, first aid room, storage room and a large training room. The basement floor contains process equipment including the aeration tank blowers, secondary scum pumps, return sludge pumps, and waste activated sludge pumps.

Condition Assessment

- The building is in fair condition overall.
- The built-up roofing, roof drains, and roof flashing appear to be adequate at the present time, but are anticipated to need replacement over the 20-year planning period.
- Most of the windows and doors are in adequate condition.
- The brick and mortar are in adequate condition.

Building Code Assessment

- No building code issues were observed, but there are NFPA 820 issues in the basement as discussed in Section 8.3 and significant NEC issues with the electrical room as discussed in Section 8.5.

Recommended Improvements

- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system.
- Seal the exterior brick and mortar as soon as possible to prevent moisture penetration which will cause structural problems.

FIGURE 8-6
RETURN SLUDGE PUMP STATION

8.2.7 Effluent Filter Building

The Effluent Filter Building as shown in Figure 8-7 was constructed in 2001 to provide an enclosure for the traveling bridge filters. The building has an approximate footprint of 125-feet by 105-feet and was constructed with a cast-in-place concrete foundation. The building is pre-engineered metal frame with batt insulation, metal siding exterior walls, and standing seam metal roof. An electrical room approximately 15-feet x 10-feet enclosed by CMU walls is located within the northeast corner of the building. All electrical/control equipment for the building including VFDs and the main distribution panel are located in the electrical room. The pump station for the filter influent is located in the northwest corner of the building.

The traveling bridge filters will continue to be used for final effluent filtration downstream of the two-stage AB activated sludge and settling processes. The filter media will be replaced as part of the planned capital improvement work.

FIGURE 8-7
EFFLUENT FILTER BUILDING

Condition Assessment

- Because of the high moisture/humidity associated with the building serving as an enclosure for the traveling bridge filters, there are significant signs of corrosion in the metal frame. The metal building system is only in fair condition for its age.
- The concrete is in excellent condition.

Building Code Assessment

- No building code issues were observed.

Recommended Improvements

Some form of rehabilitation of the existing coating and insulation system will be needed to ensure a 20-year service life. One possibility is to utilize the Stayflex insulation/coating system from Preferred Solutions, Inc. or other equivalent improvement/ rehabilitation products.

8.2.8 Existing Process Structures

The existing process structures include the aerated grit chambers, primary clarifiers, screw pumps, aeration basins, secondary clarifiers, chlorine contact tanks, distribution structures, and interconnecting channels were visually inspected during the preparation of this Facility Plan Amendment. Preliminary inspections of the process tankage indicates that they are in fair to good conditions given their age and environmental exposure. The structures exhibit normal patterns cracking and efflorescence of the concrete that occur over time. It is recommended to take reasonable measures to rehabilitate the concrete at each structure at the time of process equipment improvements/upgrades. The recommended maintenance measures to maximize the useful life of these structures are straightforward as follows:

- **Aerated Grit Chamber:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection. Because the tanks are enclosed for odor control, coat the area above the water line with a 100% solids epoxy coating system.

- **Primary Clarifiers:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection. Coat all enclosed areas including the launders with a 100% solids epoxy coating system.

- **Screw Pumps:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection.

- **Aeration Basins:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection.

- **Secondary Clarifiers:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection. Consider coating the launders with a 100% solids epoxy coating system to maximize useful life.

- **Chlorine Contact Tanks:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection.

- **Distribution Structures and Channels:** Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection.

Generally speaking, many of the above noted maintenance measures will be addressed in a prioritized manner as part of the facility's ongoing preventative and predictive maintenance program for the WWTF over the 20-year planning period.

8.3 HEATING, VENTILATION, AND AIR CONDITIONING

HVAC evaluations were performed to assess the condition of the various HVAC units and their ability to provide continued service to the WWTF for the next 20 years. Most of the facility's HVAC equipment is functional at this time, but much has reached the end of its typical service life. In general, more efficient systems are available. While the majority of the HVAC equipment is operating satisfactorily, it is unlikely that this equipment will operate reliably for the next 20 years. In addition, the National Fire Protection Association (NFPA) has established NFPA 820 "Standard for Fire Protection in Wastewater Treatment and Collection Facilities" since these facilities were constructed. NFPA 820 includes significant minimum ventilation standards that affect the electrical classification of spaces. The building code references the NFPA 820 standards, and thus these ventilation and electrical classification standards need to be met for any areas that are being modified, even for equipment replacement. The ramifications of the new standards are discussed for each building system below. Based on site visit evaluations and the latest NFPA code requirements, it was determined that upgrades to the Operations and Administration Buildings are a priority.

8.3.1 Operations Building and Influent Structure

The Operations Building and Influent Structure were constructed as part of the 1962 facility upgrade, with significant renovations in the mid-1970s upgrade. All of the existing heating and ventilation equipment is a minimum of 35 years old, and some of it is approaching 50 years old. Under NFPA 820, the influent structure and wetwell are electrically classified as Division 1 spaces unless ventilated at 12 AC/hr continuously. The pump room must be electrically classified as a Division 2 space unless ventilated at 6 AC/hr, and interconnecting spaces must also be classified. In this regard, it is important to note that this building houses the existing main electrical distribution equipment and a second electrical and control room. The recommended plan is to overhaul all of the existing ventilation and heating systems as follows:

- For the Influent Structure and wetwell, provide new ventilation system with exhaust to a new activated carbon odor control system as discussed in Section 7. In accordance with NFPA 820, the ventilation system must include mechanically supplied make up air and exhaust. Provide continuous ventilation at 3 AC/hr to the odor control system, and provide supplemental ventilation for a total of 12 AC/hr for when entering Influent Structure.

- For Pump Room, provide new ventilation system encompassing entire Operations Building that provides 6 AC/hr capacity within the Pump Room with turndown to 3 AC/hr in winter. Incorporate air-to-air heat exchanger if possible.
- Replace existing heating system with new more efficient hot water boiler. Consider conversion to natural gas.
- For electrical rooms, provide positive pressure ventilation.

8.3.2 Administration Building

The Administration Building was constructed as part of the mid-1970s facility upgrade. The existing heating and ventilation equipment are approaching 35 years old, and are in need of a major overhaul. There are problems with ductwork that has failed, and the heating system is inefficient. Similar to the issues with the Operations Building, the pump room must be electrically classified as Division 2 unless 6 AC/hr continuous ventilation is provided. The WWTF staff note that it is imperative that the wetwell be ventilated to prevent odor migration into the building. When this system has not been operating, there have been problems with odor issues within the building. The recommended plan is to overhaul all of the existing ventilation and heating systems as follows:

- For the wetwell, provide new ventilation system with exhaust to a new activated carbon odor control system as discussed in Section 7. In accordance with NFPA 820, the ventilation system must include mechanically supplied make up air and exhaust. Provide continuous ventilation at 3 AC/hr to odor control system, and provide supplemental ventilation for a total of 12 AC/hr for when entering wetwell.
- For Pump Room, provide new ventilation system encompassing entire Administration Building that provides 6 AC/hr capacity within Pump Room with turndown to 3 AC/hr in winter. Incorporate air-to-air heat exchanger if possible.
- Replace existing heating system with new more efficient hot water boiler. Consider conversion to natural gas.

- For electrical room, provide positive pressure ventilation.

8.3.3 Primary Sludge Pump Station

The Primary Sludge Pump Station has an odor control system in the first floor and primary sludge pumps in the basement that require these spaces to be classified as Division 2 unless ventilated at 6 AC/hr. The odor control equipment also requires a 5-foot envelop around all leakage points to be classified as Division 2. MCC-2 is located in the first floor, and is from the 1962 upgrade. The recommended plan is to provide a new heated make-up air system that incorporates an air-to-air heat exchanger to provide up to 6 AC/hr in the warm weather, and 3 AC/hr in the winter to allow this space to be declassified, except for a 5-foot envelop around the odor control system that must be classified as Division 2.

8.3.4 Primary Effluent Pump Station

The Primary Effluent Pump Station must be considered electrically classified as a Division 2 space unless ventilated at up to 6 AC/hr. The recommended plan is to provide a new heated make-up air system that incorporates an air-to-air heat exchanger to provide up to 6 AC/hr in the warm weather, and 3 AC/hr in the winter to allow this space to be declassified.

8.3.5 Chlorination Building

The Chlorination Building was found to be in compliance with existing NFPA requirements for electrical classification. The existing heating and ventilation system is adequate for present purposes. It is anticipated that replacement will be necessary at some point during the 20-year planning period. The one recommended improvement is to completely isolate the sodium bisulfite area to prevent migration of bisulfite odors to other portions of the building. This room should be provided with a two speed exhaust system with up to 6 AC/hr capacity for use when staff must enter.

8.3.6 Return Sludge Pump Station

The basement of the Return Sludge Pump Station must be considered electrically classified as a Division 2 space unless ventilated at up to 6 AC/hr. The recommended plan is to provide a new heated make-up air system that incorporates an air-to-air heat exchanger to provide up to 6 AC/hr in the warm weather, and 3 AC/hr in the winter to allow this space to be declassified.

8.3.7 Effluent Filter Building

The Effluent Filter Building was found to be in compliance with existing NFPA requirements for electrical classification. The existing heating and ventilation system is adequate for present purposes, and during the preliminary design phase will be evaluated for reuse as part of the MBBR facility.

8.4 INSTRUMENTATION AND CONTROLS

The instrumentation and control system includes local instruments and control panels that are wired into PLCs, and a PLC network that is connected to a supervisory control and data acquisition (SCADA) system. The SCADA system has had significant upgrades, but portions of the PLC network are antiquated. Consequently, there are various improvement needs especially given the additional input/output requirements associated with the proposed nitrogen and phosphorus removal facilities. Other system upgrade needs will likely be addressed through the WWTF's ongoing maintenance budget on a priority need basis over the 20-year planning period.-

The existing PLCs installed throughout the facility and collection system were manufactured by GE Fanuc and date back to the early 1990s. The PLC network at the WWTF includes the main PLC control panel and additional PLCs and panels at the Primary Sludge Pump Station, Primary Effluent Pump Station, Chlorination Building, and Return Sludge Pump Station. The PLC control panel for the Effluent Filter Building connects through the Return Sludge Pump Station. The PLC network at the WWTF is connected via Cat 5 copper wiring, and may include fiber optic between some of the buildings.

The collection system includes 10 PLC control panels as well that also have GE Fanuc PLCs. Nine of the ten panels communicate back to the WWTF via a VHF telemetry system that was recently upgraded, and the other panel is connected via a teledialer.

The GE Fanuc PLCs are obsolete, and it is increasingly difficult and expensive to find replacement hardware. The existing PLC network is not suitable to accommodate expansion for monitoring and control of the new nitrogen and phosphorus systems. In addition, the facility has been vulnerable to equipment losses due to lightning strikes during severe weather. The existing surge protection for the PLC panels only provides protection from the 1 10V line side of incoming utility power. The PLC panels need to be upgraded to include surge protection on the 4-20mA signal output lines. The recommended improvements include the following:

1. Replace all of the existing PLCs and PLC panels with new panels and PLCs utilizing Allen-Bradley PLCs with Ethernet communication protocol at both the WWTF and in the collection system. The panels and PLCs would incorporate enhanced surge protection to reduce problems with lightning strikes. Replacement of the main PLC panels is a particular priority.

2. Incorporate Operator Interface Terminals at all larger PLCs to provide direct access to operating information.

3. Selectively upgrade the main communication wiring at the WWTF to fiber optic as necessary.

The SCADA system monitors and controls the alarm functions of all equipment and systems within the facility and collection system. The existing SCADA system has been recently updated to iFix software running the latest version of Microsoft XP Professional. The system includes two server licenses and five remote nodes. This system is suitable for expansion to encompass the additional I/O from the proposed new nitrogen and phosphorus removal facilities. In addition, the City would like to expand this system to include data from the Synagro SCADA system at the Solids Handling Facilities. The system utilizes Ops32 for the operations database.

8.5 ELECTRICAL AND EMERGENCY POWER

The electrical and emergency power systems of the WWTF require a significant upgrade due to the age of the equipment and changes in the requirements of the National Electrical Code (NEC) and other standards, such as NFPA 820. One of the biggest drivers is the need to upgrade the emergency power system to provide standby power for the entire WWTF. The most critical of the aging equipment is the 1960's era motor control centers (MCCs) which are very difficult to find replacement parts for. In addition, several of the main electrical rooms are not in compliance with NEC guidelines for arc flash and egress safety. The NEC requires that any existing system be brought into compliance if it must be modified in any way. In addition, there are changes in the electrical distribution to the Solids Handling Facilities that should be completed as part of the improvements, and this is addressed further below.

8.5.1 Site Electrical Service

The plant receives electrical service from National Grid via an overhead 13.8 kV utility line east of the Operations Building as shown in Figure 8-8. Originally, the majority of the WWTF is fed from e a City-owned 2000 kVA, 13.8kV-480V pad-mounted transformer. The transformer supplies the Main Distribution Switchboard in the Operations Building.

There is a second 1000 kVA 480V pad-mounted transformer at the Return Sludge Pump Station, as shown in Figure 8-9. MCC-6A and the blowers for the diffused aeration system are the only loads fed from this second transformer.

In 2007, Synagro constructed a new 13.8kV electrical service for the Sludge Handling Facilities when the new fluidized bed incinerator was constructed. This new services includes a 4,160V transformer to feed the 700hp fluidized bed air blower of the incinerator and a 480V transformer for the remainder of the facility. However, only the new equipment was wired into the new distribution switchgear and MCCs that were installed as part of the upgrade, and not existing equipment. Thus, portions of the existing thickening, dewatering, and incineration facilities are still wired through the WWTF Main Distribution Switchboard.

Because of the changes by Synagro, there are two utility meters on the incoming line from National Grid. One meter is for the WWTF; the other is for the “new” portions of the Synagro facility. There is also submetering at the Main Distribution Switchboard, MCC-1 and MCC-5 for the sludge thickening, dewatering and incineration facilities, and service water pumping. Synagro is directly metered and charged for all of its power use.

The contract between the City and Synagro includes a requirement for the City to provide a power connection from the WWTF to Synagro.

It is proposed that, as a part of the overall plant upgrade, that the plant’s medium-voltage distribution system be controlled through a single connection to National Grid. New 13.8kV switchgear would control:

- incoming power from National Grid
- the interconnection between the WWTF and Synagro
- power from the standby generator

- distribution feeders to WWTF site loads

Synagro has begun work on a 2MW cogeneration system. This new generator is scheduled to be on line in late 2013. The new 13.8kV electrical power distribution system for the WWTF is planned to be online by mid-2014 to allow the WWTF to start using the power generated by Synagro.

FIGURE 8-8
AERIAL VIEW OF ELECTRICAL POWER SERVICE – NORTH END

FIGURE 8-9
AERIAL VIEW OF ELECTRICAL POWER SERVICE – SOUTH END

8.5.2 On-Site Low-Voltage Distribution System

From the 2000kVA padmount transformer, the incoming service to the Main Distribution Switchboard at the Operations Building is 3000A, 480V, 3-phase, 3-wire.

As part of the improvements following the 2000 Facility Plan Amendment, a number of MCCs were added. However, these were all fed from the existing Main Distribution Switchgear via subfeeds from existing MCCs.

Table 8-2 summarizes the existing MCCs on the 480-volt power distribution system, except for any new MCCs installed by Synagro as part of the fluidized bed incinerator upgrade. The MCCs are single ended receiving a single power supply from the Main Switchgear. Most of the MCCs date back to the 1977 upgrade, and as noted above several date back to the 1960s upgrade. None of this equipment is in production any longer. In many cases, replacement parts are difficult or impossible to find. Fortunately, the same equipment manufacturers are still in business and they offer direct replacements of motor starter and feeder circuit breaker cubicles. It is recommended that as new loads are added to the MCCs, any available "spare" starters or circuit breakers not be used. Instead, the old equipment should be removed and replaced with completely new cubicles.

Additionally, as components fail in the remaining cubicles repair should be made through the replacement of entire cubicles.

TABLE 8-2
SUMMARY OF MOTOR CONTROL CENTERS

Name	Location	Date of Installation
MCC-1	Operations Building – First Floor	1962
MCC-1A	Administration Building – Basement Floor	1977
MCC-2	Primary Sludge Pump Station	1962
MCC-3	Gravity Thickener Building (Synagro)	1977
MCC-4 & MCC-4A	Dewatering Building – First Floor (Synagro)	1977
MCC-5	Chlorination Building – Control Room	1977
MCC-5A	Chlorination Building – Control Room	2001
MCC-6 & MCC-6A	Return Sludge Pump Station – First Floor	1977
MCC-7	Dewatering Building – First Floor (Synagro Incineration Facilities)	1977
MCC-8	Primary Effluent Pump Station	1977
MCC-9	Chemical Building	2000
MDP-EF	Effluent Filter Building	2001

8.5.3 Standby Emergency Power

The emergency power system includes two 500kW generators in the Administration Building, and was set up to provide standby power for the Main Distribution Switchboard, but not the second transformer that feeds MCC-6A. In the 1990s, the diffused aeration system was upgraded to fine-bubble and the mechanical surface aerators were abandoned. From this time, the WWTF has not had emergency power back-up for the aeration system. The inability to maintain aeration to the basins does not meet current WWTF design standards, such as TR-16, and must be addressed, as discussed further below.

The generators are approximately 35 years old, and were upgraded with new power centers at some point. The useful life of both generators has passed, and it is increasingly difficult to find spare parts for maintenance. Also, there is a need for additional generator capacity for the new nitrogen and phosphorus removal processes. The recommended improvements include providing a new 2500 kVA generator in an outdoor walk-in sound-attenuated enclosure to provide backup power for the 13.8kV Main Distribution Switchgear.

The new standby power system would be designed to provide the appropriate level of standby electrical power to maintain all necessary equipment to provide the level of treatment required to meet the discharge limits of the RIPDES permit.

8.5.4 Main Distribution Switchboard – Operations Building

The Main Distribution Switchboard is located on the first floor of the Operations Building and is approximately 35 years old. The room has several NEC code violations including inadequate working space at the front of the switchboard and inadequate egress pathways from the room. Any modifications to the systems in these areas will require bringing the area up to current code requirements. There is also a mechanical duct that runs approximately 1 foot above the top of the Main Distribution Switchboard. The NEC requires a clear working space of 3'-6" in front of the electrical gear and 6 feet above the gear. For equipment over 1200 amps, there needs to be two separate means of egress unless there is double the working clearance and a clear egress path. The egress door currently opens into the room, but the NEC requires that all electrical room egress doors open outward and are equipped with panic hardware. The NEC requires that these code violations be addressed if the Switchboard is to be modified in any way. There is insufficient space within the Operations Building to address the existing clearance issues.

Another issue is that the existing switchgear and MCCs are single ended. Current WWTF design guidelines favor the use of switchgear and MCCs that are fed from multiple sources, because they provide improved maintainability and greater redundancy and thus enhance the reliability of the electrical feed system. Consequently, all new switchgear and MCCs are recommended to be configured as main-tie-main. Where it is practicable, existing MCCs will be modified to have two incoming feeders.

Based on the observed deficiencies, and the need for modifications, the recommended plan is to replace the existing 480V Main Distribution Switchboard with new main-tie-main switchgear in a new electrical building. Possible locations include the garage for the collection system as shown in Figure 8-10 or in an outdoor enclosure south of the Operations Building.

The approach of replacing the existing Switchboard will simplify construction sequencing issues as well.

Portions of the existing thickening, dewatering and incineration facilities are still fed through the Main Distribution Switchgear even though Synagro has installed new transformers and switchgear as part of the fluidized bed incinerator upgrade. As the plant upgrades are constructed, the interconnections between the WWTF and Synagro will be removed. Synagro has been actively working toward moving their loads off of the WWTF feeders.

8.5.5 Operations Building

In addition to the Main Distribution Switchgear, the Operations Building has a separate electrical and control room that houses MCC-1 and the five variable frequency drives (VFDs) for the raw influent pumps. The MCC was installed as part of the 1960's upgrade, and served as the main distribution switchgear initially. For this reason, MCC-2 and MCC-3 are fed from MCC- 1. The MCC is a single-ended type, manufactured by Square D, and is no longer supported due to age. The Raw Sewage Pump VFDs were originally installed in the 1990s, and then replaced and updated in 2005 with units rated for 75-hp motors. There is inadequate clearance between MCC-1 and the VFDs to meet current NEC requirements. The following recommendations were developed:

1. Move the drives further away from MCC-1 to restore the NEC-required front working space.
2. Provide a new second feeder to MCC- 1.
3. Feed MCC-2 which serves the Primary Sludge Pump Station from the proposed new 480V Main Distribution Switchgear.
4. Eliminate the feed to MCC-3 at Synagro
5. Reuse the existing VFDs to feed the new Raw Sewage Pumps.

8.5.6 Administration Building

The equipment in the Administration Building is powered through MCC-1A, which is located adjacent to the generators in the basement. The MCC is a Cutler Hammer Unitrol and is approximately 35 years old. The MCC is powered through the Main Distribution Switchboard. The VFDs for the Administration Building Pump Room are located in the electrical room of the Operations Building. As noted above, the recommended plan is to demolish the two existing generators, and replace them with a new unit located adjacent to the proposed new 13.8kV main distribution switchgear. In addition, the following recommendations were developed:

1. Provide a second feeder to MCC-1A.
2. With the removal of the 500kW generators, it should be possible to install VFDs for the Administration Building Pump Room within this Electrical Room if desired in the future.

8.5.7 Primary Sludge Pump Station

The equipment in the Primary Sludge Pump Station is powered through MCC-2, which is located in the first floor. The motor control center was manufactured by Square D and is approximately 45 years old. As noted in Section 8.3, this MCC is located a process area that is currently classified as Class 1, Division 1 or 2. The recommended plan for ventilation would alleviate the electrical classification issues. However, the space would remain a process area. The recommended plan is to retain the existing MCC and provide it with a second feeder.

8.5.8 Primary Effluent Pump Station

The existing screw and submersible pumps at the Primary Effluent Pump Station are powered through MCC-8 located in the first floor room. The MCC is a Cutler Hammer Unitrol and is approximately 35 years old. There is a hoist mounted to the floor in front of this motor control center, which does meet the 3'-6" working clearance required by the NEC. The recommended plan for primary effluent pumping will require that the pump station building be demolished. The MCC can remain in service until the time the building is removed.

8.5.9 Chemical Building

The Chemical Building includes several existing electrical panels and transformers. Most of the panels are rusted and have some corrosion due to the environment. The recommended plan is to remove the existing sodium hydroxide storage tank and pumps (other tanks will also be evaluated for relocation during preliminary design), and create a new room for electrical and I&C equipment in that space. The existing electrical panels in the Chemical Building will be replaced.

8.5.10 Chlorination Building

The equipment within the Chlorination Building is powered through MCC-5 is located in the first floor. It is a Cutler Hammer Unitrol and is approximately 35 years old. Much of the equipment fed by MCC-5 has been disconnected and is no longer in use. In addition, MCC-5A was installed in this area as part of the Aeration Tank upgrades in 2001 to supply power for the submersible mixers and internal recycle pumps, and is fed through MCC-5. The six AFDs for the internal recycle pumps were installed in separate enclosures. MCC-5A is an Allen Bradley unit located approximately 3 feet in front of the older Cutler Hammer unit, which does not meet current NEC clearance requirements. The following recommendations were developed:

1. Provide a new double-ended motor control center to replace existing MCC-5A in a new electrical room. It appears possible to create the necessary electrical room in the former chlorine cylinder area. The MCC-5 will remain and should be re-fed from the new electrical service. Process changes in the aeration basin will lead to the removal of the recycle pump AFDs.

2. With the creation of the new electrical room, the main service for the building enters through a new main-tie-main distribution switchboard in the room. Two new 13.8kV-480Y/277V padmount transformers will be installed outdoors adjacent to the new electrical room.

8.5.11 Return Sludge Pump Station

The equipment in the Return Sludge Pump Station is powered through two existing Cutler Hammer Unitrol motor control centers (MCC-6 & MCC-6A) located in the first floor electrical room. As previously noted, MCC-6A is fed from a separate 1000kVA 480V transformer to supply power to the aeration blowers. When the diffused aeration system was converted to fine-bubble, the mechanical surface aerators powered through MCC-6 were demolished. Because of this excess capacity, the Main Distribution Panel for the Effluent Filter Building is powered through MCC-6. The existing electrical room does not comply with current NEC requirements for clearance and egress. Equipment over 1200A requires two separate means of egress unless there is double the working clearance and a clear egress path. There is only one means of egress from the room, and the door opens into the room. The recommended improvements include the following:

1. Expand the existing electrical room into the existing locker room and create a second egress to the outside.

2. In the new electrical room, provide a main-tie-main switchboard to distribute power to

the four blowers and MCC-6. Serve the new switchboard from two 13.8kV-480Y/277V transformers to be installed outside the southeast corner of the building.

3. With the replacement of the two "old" Hoffman blowers, it will be possible to demolish MCC-6A.

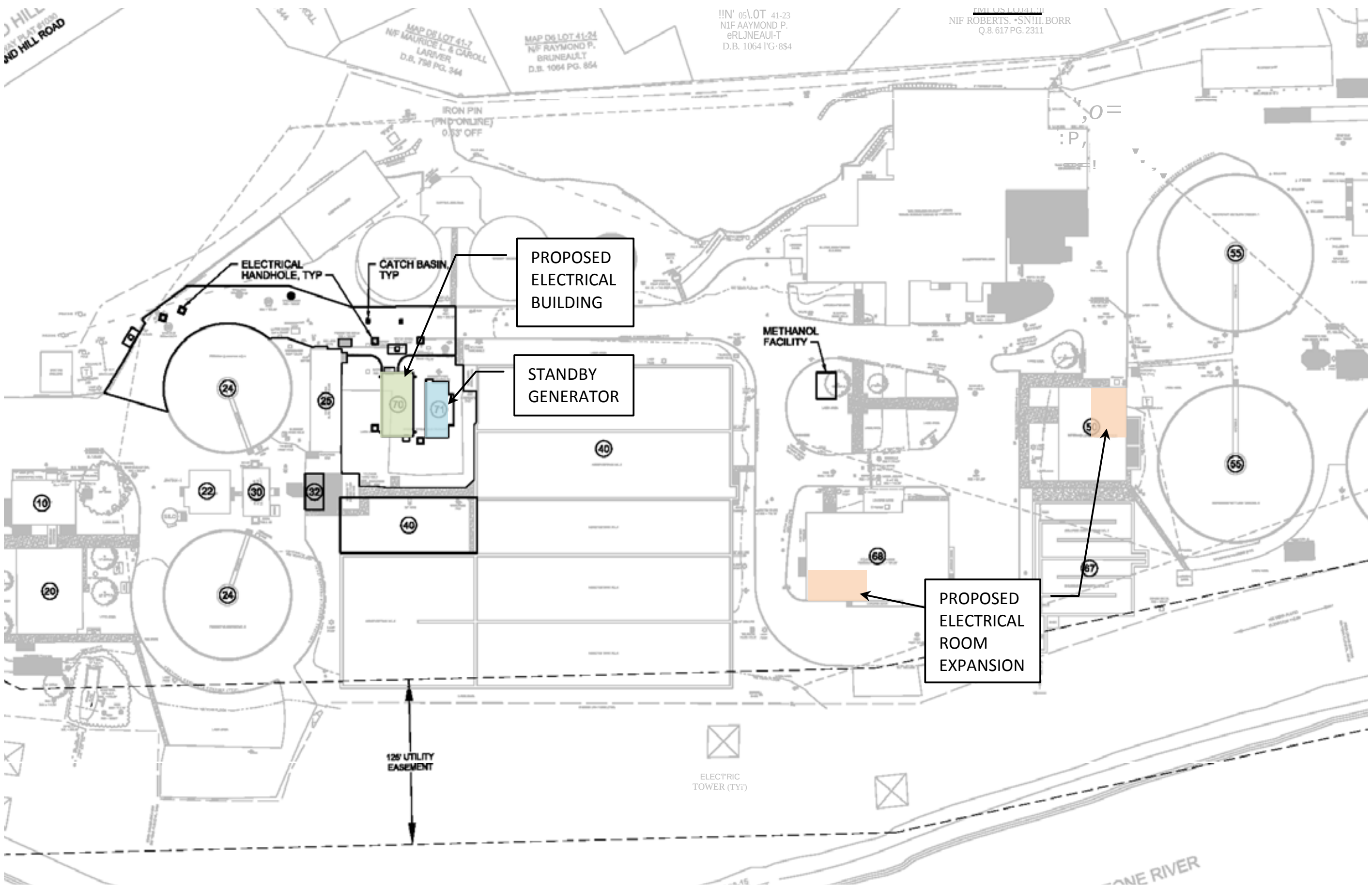
8.5.12 Summary

The most significant elements of the electrical systems upgrade needs include:

- The plan for a new electrical feed and medium-voltage switchgear
- Interconnection to the Synagro cogeneration system
- Removal or abandonment of existing 480V connections between WWTF and Synagro facilities
- New standby generator, sized to support the entire RIDEM-mandated WWTF process.
- New 13.8kV distribution loop, with greater redundancy than the existing radial feeders. The distribution loop will supply sets of two transformers each at:
    - Operations Building
    - Chlorination Building
    - RAS Pumping Station
- New Main Distribution Switchboard,
- New I&C/electrical room in the Chlorination Building
- The expanded electrical room in the Return Sludge Pump Station.

The planned capital improvements to the electrical service equipment are shown in Figure 8-10.

FIGURE 8-10
SITE PLAN – PROPOSED ELECTRICAL IMPROVEMENTS



SECTION 9
PLAN SELECTION

9.1 INTRODUCTION

The previous sections in this Facility Plan Amendment described the existing facilities and unit treatment processes at the Woonsocket WWTF, and the corresponding nitrogen and phosphorus removal upgrades and improvements required to meet the new RIPDES discharge permit limits. In addition to the upgrades required to meet the new permit, additional upgrades to various other unit process were identified. These additional upgrades will need to be addressed as part of the City's on-going long-term capital improvements over the 20-year planning period. The facilities and unit treatment processes planned capital improvements that are needed to address the permit-driven nutrient removal, plant operation, and hydraulic capacity requirements include:

- Replace existing influent bar screen with new fine screen
- Add new carbon odor control system for screening facility and Operations and Administration Buildings wetwells
- Add influent pumping capacity
- Add influent flow metering
- Replace primary effluent pumping system
- Modifications to the activated sludge system required for nutrient removal, including basin modifications to operate as a two stage activated sludge AB system
- SCADA system upgrades, including SCADA system replacement
- Select electrical and standby emergency power upgrades
- Select ventilations system improvements

The facilities and processes which can be part of an on-going capital improvements program but which will need to be addressed over the 20-year planning period include:

- Existing Raw Influent and Recycle Flow Pumping Facilities
- Aerated Grit Chambers

- Primary Clarifiers
- Secondary Clarifiers and RAS/WAS pumping
- Odor Control Facilities for the balance of the plant.
- Plant-wide Support Systems (Electrical, HVAC, Plant Water), except for those included in the planned capital improvements.
- Miscellaneous Building Systems Improvements

This section summarizes the planned capital improvements at the WWTF both to meet the new RIPDES discharge permit limits and to ensure continued reliable operation over the next 20-year planning period and includes the WWTF design build project planned improvements capital costs.

9.2 DESCRIPTION OF RECOMMENDED PLAN

9.2.1 Screening Facilities

Improving the WWTF's screenings facilities is required to improve the system reliability and to reduce maintenance on downstream facilities. Screening Modifications will involve replacing the existing coarse mechanical screen with a new 3/8-inch mechanical screening system, complete with a washer-compactor unit. The new screening unit would be located in the main influent channel of the lower basement area of the Operations Building. The screening unit would be designed to convey collected screenings above-grade to a washer-compacter system located on top of the existing raw influent screenings room (which is the equivalent of the first floor level of the Operations Building). The washer compactor unit will not function properly in an outdoor environment due to potential freezing concerns during colder months; therefore it will be necessary to construct a weather-proof enclosure around the washer-compactor unit. This enclosure will also provide an odor control containment area around the screening systems and will help mitigate any fugitive odors which could be generated from the lower influent channel and screening room areas.

9.2.2 Raw Influent and Recycle Flow Pumping Facilities

Some improvements to the raw influent and recycle flow pumping facilities are required to provide additional pumping capacity for the WWTF's future design flows previously presented in

Sections 2 – Existing Flows and Loads and Section 3 – Future Flows and Loads of this Facility Plan Amendment. One new 8 mgd influent pump will be installed in the current vacant pumping unit location to provide appropriate pumping system redundancy to satisfy recommended process design guidelines and standards. The remaining recommended influent pumping system improvements should be implemented as part of the WWTF's on-going long-term capital improvements plan to ensure the facility will continue to be able to reliably maintain influent pumping operations over the next 20-year planning period. The full recommended plan for improving the raw influent and recycle flow pumping facilities, in the future, as part of the ongoing maintenance repair and replacement, includes installing three (3) new 10 mgd pumping units, while retaining one (1) of the existing 8 mgd pumps and one (1) existing 10 mgd pumping unit for system redundancy. This plan will ensure that the WWTF has adequate and reliable pumping capacity to meet the design flows described in Section 3 – Future Flows and Loads for the next twenty year planning period.

Additional recommended influent pumping system improvements include:

- Installation of a new AFD (required as part of permit-driven upgrades);
- Replacement of existing raw influent piping and valves will be considered as part of the maintenance program
- Associated required electrical system improvements, which are discussed in subsequent sections.

9.2.3 Grit Removal Facilities

Improvements to the aerated grit removal facilities are not a requirement of the permit-driven nitrogen and phosphorous upgrade needs but are recommended as part of the WWTF's on-going long-term capital improvements plan to ensure the facility will continue to maintain reliable operations over the next twenty year planning period. The recommended plan for improving the aerated grit removal facilities, in the future, generally includes:

- Replacement of the two aeration blowers;
- Addition of AFDs for the blowers allowing operational flexibility.

9.2.4 Primary Treatment Facilities

Improvements to the primary treatment facilities are not a requirement of the permit-driven nitrogen and phosphorous upgrade needs but are recommended as part of the WWTF's on-going long-term capital improvements plan to ensure reliable facility operations over the next twenty year planning period. The recommended plan to upgrade the primary treatment facilities in the future includes:

- Replacement of the two primary clarifier sludge collection mechanisms and drives;
- Perform miscellaneous spot repairs of the primary clarifier concrete tanks;
- Replace the primary clarifier dewatering pump;
- Raise the perimeter concrete side walls of the primary scum wells;

9.2.5 Primary Effluent Pumping Facilities

Improvements to the primary effluent pumping facilities are required as part of the permit-driven nitrogen and phosphorous upgrade based on an analysis of the WWTF's future design flows previously presented in Section 3 – Future Flows and Loads. Because the hydraulic grade line will be higher for the modified activated sludge basins, the primary effluent pump station will be replaced with a new submersible pump station. Providing a new pump station will facilitate the construction sequence for the basin modifications. The pump station will include four (4) submersible pumps equipped with adjustable speed drives. Each pump will have 12.6 mgd capacity, to provide 38 mgd, with one pump out of service. The discharge piping to the basins will have flow meters and flow control valves to split flow to the three first stage (i.e., Stage “A”) activated sludge basins. The primary effluent pump station modifications will include:

- Integrating new pumping system improvements into the WWTF SCADA system;
- Associated required electrical improvements which are discussed in subsequent sections.

9.2.6 Secondary Treatment Facilities

Improvements to the secondary treatment facilities can be separated into two categories, (1) those that are a requirement of the permit-driven nitrogen and phosphorous upgrade and (2) those that should be implemented as part of the WWTF's on-going long-term capital improvements plan to ensure reliable facility operations over the next twenty year planning period.

The operation of the secondary treatment facilities was discussed in Section 4 – Performance of the Secondary Treatment Facilities and Process Model Development. Modifications to the basins, as described in Section 4, includes providing first and second stage activated sludge systems and first stage settling using lamella plate settlers in the existing basins. These modifications will require modifications to hydraulic channels, air piping, diffusers replacement, new mixers, retrofit for plate settlers installation, and installation of a new first stage return activated sludge system.

Additional blower capacity will be required and the blowers providing air to the A stage will need to operate at a higher pressure than the B stage blowers. Alternatives are being evaluated for providing a combination of new turbo blower capacity and existing blowers to provide process air to both stages with a redundant blower at peak week conditions.

The recommended plan for secondary clarifier improvements that should be part of the long-term capital improvements plan includes:

- Replacement of the three secondary clarifier sludge collection mechanisms and drives;
- Perform miscellaneous spot repairs of the secondary clarifier concrete tanks as discovered upon inspection;
- Replace the four return activated sludge pumps;
- Replace the three waste activated sludge pumps;

9.2.7 Solids Handling Facilities

Under this Facility Plan Amendment there are no recommended improvements to the Solids Handling Facilities since Synagro is responsible for these facilities as part of their merchant sludge incineration facilities.

9.2.8 Flow Metering

Improvements to the flow metering methods at the WWTF, while not directly required as part of the permit-driven nitrogen and phosphorous upgrade needs, are recommended to provide influent and recycle flow measurement. New magnetic flow meters will be installed in the influent pump discharge piping. A magnetic flow meter will be installed on the recycle pump discharge.

Additional recommended improvements associated with this plan include:

- Flow meter integration into the WWTF SCADA system.

9.2.9 Odor Control Facilities

Improvements to the odor control facilities are not a requirement of the facility's permit-driven nitrogen and phosphorous upgrade needs but the recommended improvements should be implemented as part of the WWTF's on-going long-term capital improvements plan to help the facility avoid nuisance odor problems. Planned improvements for the facilities odor control systems includes installation of a new high capacity carbon type odor control system to handle the raw influent screenings facilities, Administration Building wetwell and the Operations Building wetwell. Additional planned improvements associated with this plan include new odor control piping to the raw influent screening facilities and the Operations Building wetwell.

9.2.10 Building Systems Recommended Improvements

Section 8 provided detailed evaluations of WWTF buildings, tanks, and other plant-wide support systems. This section will summarize the recommended improvements based on the evaluations and are organized by location.

9.2.10.1 Operations Building and Influent Structure

Architectural/Structural

The following list of improvements will be considered as part of the treatment plant ongoing repair and replacement maintenance program:

- Remove existing roofing, including roof insulation, roof drains and roof flashing, and replace with a new EPDM membrane system along with rigid insulation.
- Replace the aluminum door that provides access to the headworks/wetwell area.
- Seal the exterior brick and mortar.
- Seal the concrete deck of the influent structure.
- Provide separation between the pump room and remaining areas of the building to limit the extent of spaces requiring ventilation, to reduce classification.

Heating, Ventilation, and Air Conditioning

The following heating, ventilation, and air conditioning improvements are planned as a part of the treatment plant planned capital improvements:

- For the Influent Structure and wetwell, provide a new ventilation system with exhaust to new activated carbon odor control system as discussed in Section 7. The Influent Structure and wetwell will have an NEC classification of Class 1 Division 1. The ventilation system must include mechanically supplied make-up air and exhaust and provide continuous ventilation at 2 AC/hr to the odor control system. Combustible gas detection will be provided.

- For pump room, provide new ventilation system encompassing the pump room that provides 6 AC/hr capacity within the Pump Room with turndown to 3 AC/hr in winter. Incorporate air-to-air heat exchanger, if possible.

- For electrical rooms, provide positive pressure ventilation and cooling using an air-conditioning system.

- For control room, provide positive pressure ventilation and cooling using an air-conditioning system.

- For new SCADA room, provide positive pressure ventilation and cooling using an air-conditioning system.

- In the future, replacement of the existing hot water heating system and boiler will be part of the ongoing maintenance repair and replacement.

Electrical

- Provide a new second feeder to Motor Control Center. As part of modifications to MCC-1, remove the feeder to MCC-2 and serve that MCC from the new 480V Main Distribution Switchgear. Eliminate the feed to MCC-3 which serves the Synagro Gravity Thickener Facility.

- Continue to use the existing AFDs to feed the new Raw Sewage Pumps. Provide one new AFD with the new pump.

9.2.10.2 Administration Building

Architectural/Structural

- Remove the circular stair case connecting the pump room and the generator room and seal the opening.

The following list of improvements are considered part of the treatment plant ongoing repair and replacement maintenance program:

- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains and roof flashing, and replace with a new EPDM membrane system.
- Seal the exterior brick and mortar as soon as possible.

Heating, Ventilation, and Air Conditioning

- For the wetwell, provide new ventilation system with exhaust to new activated carbon odor control system as discussed in Section 7. The wetwell will have an NEC classification of Class 1 Division 1. The ventilation system must include mechanically supplied make up air and exhaust and provide continuous ventilation at 2 AC/hr to the odor control system. Combustible gas detection will be provided.
- For pump room, provide new ventilation system encompassing for the pump room that provides 6 AC/hr capacity within pump room with turndown to 3 AC/hr in winter. Incorporate air-to-air heat exchanger if possible.
- For electrical room, provide positive pressure ventilation and cooling using an air-conditioning system
- In the future, replacement of the existing hot water heating system and boiler will be part of the ongoing maintenance repair and replacement.

Electrical

- Provide a second feeder to MCC-1A.
- With the removal of the generator, install AFDs for the Administration Building Pump Room within this Electrical Room.

9.2.10.3 Primary Sludge Pump Station

Architectural/Structural

The following list of improvements are considered part of the treatment plant ongoing repair and replacement maintenance program:

- Remove existing roofing, including roof insulation, roof drains and roof flashing, and replace with a new EPDM membrane system along with rigid insulation.
- Seal the exterior brick and mortar.

Heating, Ventilation, and Air Conditioning

Planned capital improvements include the following:

- Provide a new heated make-up air system that incorporates an air-to-air heat exchanger, where possible, to provide up to 6 AC/hr in the warm weather, and 3 AC/hr in the winter to allow this space to be declassified, except for a 5-foot envelope around the odor control system that must be classified as Class 1 Division 2.

Electrical

Planned capital improvements include the following:

- Provide a second feeder to MCC-2.

9.2.10.4 Primary Effluent Pump Station

The primary effluent pump station will be replaced as part of the capital improvements work.

Heating, Ventilation, and Air Conditioning

The primary effluent pump station will be replaced as part of the capital improvements work.

Electrical

- A location will need to be identified for the MCC serving the new primary effluent pump station

9.2.10.5 Chemical Building

Work in the chemical building will include adding new metering pumps for ferric chloride. A new phosphoric acid system will be added to balance the phosphorus required in the activated sludge process. A new methanol storage and feed system will be added south of the chemical building to provide supplemental carbon needed for the two-stage AB activated sludge nutrient removal process.

Architectural/Structural

Re-configure a portion of the building to create a new space for I&C and electrical equipment.

HVAC

- Provide positive pressure ventilation and heating and cooling to the new I&C/electrical room using an air-conditioning system.
- Provide a new make-up air unit and verify capacity of existing exhaust fan to provide continuous ventilation to the space, at 1 cfm per square foot.

9.2.10.6 Chlorination Building

Architectural/Structural

- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains, and roof flashing, and replace with a new EPDM membrane system.
- Seal the exterior brick and mortar as soon as possible.
- Modify the unused chlorine storage space to serve as a new room for electrical and I&C equipment.

Heating, Ventilation, and Air Conditioning

- Isolate the sodium bisulfite area, from return air to the remaining areas of the building, to prevent migration of bisulfite odors to other portions of the building. Provide this room with an exhaust system with 1 cfm per square foot capacity.
- Provide positive pressure ventilation and heating and cooling to the new electrical room using an air-conditioning system.

Electrical

- Provide new main-tie-main switchboard to distribute power to MCCs at the facility.
- Provide two feeders from the new switchboard to existing MCC-5.
- Provide a new main-tie-main motor control center to serve the loads (mixers and pumps) at the aeration basins.
- Demolish the Allen Bradley AFDs and MCC-.

9.2.10.7 Return Sludge Pump Station

Architectural/Structural

- Expand the existing electrical room into the existing locker room and create a second egress to the outside. Renovate the remaining space into a new restroom/locker room facility, possibly incorporating existing storage space.
- At some point over the 20-year planning period, remove existing roofing, including roof insulation, roof drains and roof flashing, and replace with a new EPDM membrane system.
- Seal the exterior brick and mortar as soon as possible.

Heating, Ventilation, and Air Conditioning

- Provide a new heated make-up air system that incorporates an air-to-air heat exchanger, where possible, to provide up to 6 AC/hr in the warm weather, and 3 AC/hr in the winter to allow this space to be declassified.
- For electrical room, provide positive pressure ventilation and heating and cooling using an air-conditioning system.

Electrical

- Provide new main-tie-main switchboard, served by two new padmount transformers. Distribute power from the switchboard to the four process air blowers and to MCC-6
- Provide a second feeder to MCC-6.

9.2.10.8 Effluent Filter Building

Filters

- Provide new media in the filters. This work is planned as part of the capital improvements work.

Architectural/Structural

- Some form of rehabilitation of the existing coating and insulation system will be needed to ensure a 20-year service life. One possibility is to utilize the Stayflex insulation/coating system from Preferred Solutions, Inc. or other equivalent improvement/ rehabilitation products.

Heating, Ventilation, and Air Conditioning

No changes are anticipated.

9.2.11 Plant-wide Support Systems Recommended Improvements

9.2.11.1 Process Structures

The existing process structures include the aerated grit chambers, primary clarifiers, screw pumps, aeration basins, secondary clarifiers, chlorine contact tanks, distribution structures and interconnecting channels. The process tankage was evaluated and described in detail in Section 8. The recommended measures to maximize the useful life of the structures are common to all concrete structures/tanks and include:

- Clean and grout any areas with spalling. At all expansion joints, remove existing sealant, and re-seal. Repair any significant cracks using epoxy injection.

9.2.11.2 Instrumentation and Controls

The following improvements are planned as part of the WWTF Instrumentation and Controls capital improvements work:

- Replace all of the existing PLCs and PLC panels with new panels and PLCs utilizing Allen-Bradley PLCs with Ethernet communication protocol at the WWTF. The panels and PLCs will incorporate enhanced surge protection to reduce problems with lightning strikes. Replacement of the main PLC panels is a particular priority. The collection system PLCs do not require replacement at this time.

- Incorporate Operator Interface Terminals at all larger PLCs to provide immediate access to operating information.

- Selectively upgrade the main communication wiring at the WWTF to fiber optic as necessary.

- Expand the SCADA system to include data from Synagro's SCADA system at the Solids Handling Facilities. The system utilizes Ops32 for the operations database, and it is recommended this system be updated to OpsSQL.

Allen Bradley Control Logix controllers will replace the existing aging GE Fanuc PLCs. The Allen Bradley PLCs will be new generation L72 ControlLogix, 100 Mbit Ethernet enabled, with redundant processor power supplies.

The Genius serial communications platform will be replaced with a 100 Mbit Ethernet remote I/O system, with enhanced security features, including secure firewalls between business networks and SCADA networks, and between SCADA and T1 line (Internet) access. Encrypted, secure Wi-Fi access will be installed for operation and maintenance of the WWTF process controls.

To provide security for SCADA communications, a self-healing ring fiber optic topology will serve the local area SCADA network to be isolated from business networks and outside wide area networks by routers and firewalls to prevent unauthorized access. SCADA access will be controlled through the use of encryption on visible networks, and password protection on hardware ports. Operator access to SCADA will be determined based on job function and need to know. Vulnerable hardware, such as switches and PLCs, will be located in secure areas where space permits, and lockable.

Reliability and Redundancy Features

Analog I/O will be upgraded to provide surge protection to each point, and all plant control panels and SCADA hardware will be provided with uninterruptable power sources, external to the enclosure or planned into the enclosure space. Workstation displays will be duplicated to allow plant operators continued access should a single or multiple workstations need maintenance or have a drive failure. In addition, the existing iFIX SCADA will be migrated to newly installed redundant servers which will allow workstation PCs and encrypted Wi-Fi tablets to run as SCADA clients. A separate Historian server will decrease the processing burden on the SCADA servers and allow for the backup of critical plant data, alarming, trending, and reporting functions.

Self-Diagnostic, Data Analysis, and Alarm Capabilities

The Allen Bradley PLCs to be installed are equipped and programmed with self-diagnostic features that trigger alarms if a system component fails. The SCADA system monitors the “processor health” of each PLC, and sets an alarm if a fault condition is triggered. PLC communications are configured in a self-healing ring topology, so that if one communications path is in fault, another will allow the processors to continue to communicate with SCADA.

The iFIX HMI will be retained with modifications to the standard iFIX software program to allow operation as a client-server architecture, and to the application-specific software and control WWTF functions, to include process upgrades.

All activity in the SCADA system is tracked and monitored, time stamped, and recorded on a secure server, including login and logout of operators, set point changes, and alarms. Operators are provided with individual login passwords that only they know, and the access they are allowed within the SCADA system is relative to their job description and the requirements of their work.

Alarms are generated for any process variable that is out of bounds or in a faulted state. All alarms are dated and time stamped and recorded on the same secure server, which can only be accessed by authorized personnel. Alarm summaries can be printed from SCADA. Alarms are displayed on the SCADA screen and can be “acknowledged,” but will not go off the screen until the alarm condition is corrected.

All alarms can be disabled by authorized operators with the proper SCADA clearance level. This usually includes operators who are responsible for maintenance, and supervisory personnel. Alarms on variable process measurements can be set point adjusted by authorized operators.

Analog and discrete inputs from field instruments will be monitored by the PLC controller for loss of signal or instrument failure. Motors are monitored for run status and faults. Valves are monitored to confirm they reach their full opened/full closed positions.

The existing OPS WWTF water reporting system will be replaced with an updated water reporting tool and maintenance management system. Complete, up-to-date wiring diagrams and operation and maintenance materials for SCADA components will be generated and incorporated into the plant maintenance management system. A tag numbering system will be deployed, assigning a unique component and facility number to each instrument and piece equipment at the plant. A SCADA network printer is included for alarm, report, and work order printing.

Expansion Capacity

The work of upgrading the WWTF SCADA system would be of limited value, if in 4 or 5 years, the system was again at its operational capacity. For this reason, the SCADA upgrades are being planned with an eye toward long-term plant growth and in anticipation of the need for future facility expansions.

- Control panels will be designed with at least 25 percent spare capacity, with processors that can be upgraded and enlarged if necessary.
- Use of the server/client topology for the SCADA HMI will allow for additional control screens and workstations, as needed.
- Network infrastructure, fiber optic connectivity, and hardware will provide adequate bandwidth well beyond the expected lifespan of much of the other plant equipment, and will allow for the possibility of future technical advancements.
- Replacement of the small PLCs at the collection pump stations and siphons will be executed over time as the need for updating the stations arises.

9.2.11.3 Standby Power

A new standby power system will be added as part of the capital improvements work to provide an appropriate level of standby electrical power to maintain all necessary equipment to provide the level of treatment required to meet the discharge limits of the RIPDES permit. The recommended improvements include the following:

- Provide a new 2500 kW generator in an outdoor walk-in sound-attenuated enclosure to provide back-up power for the 13.8kV Main Distribution Switchgear.

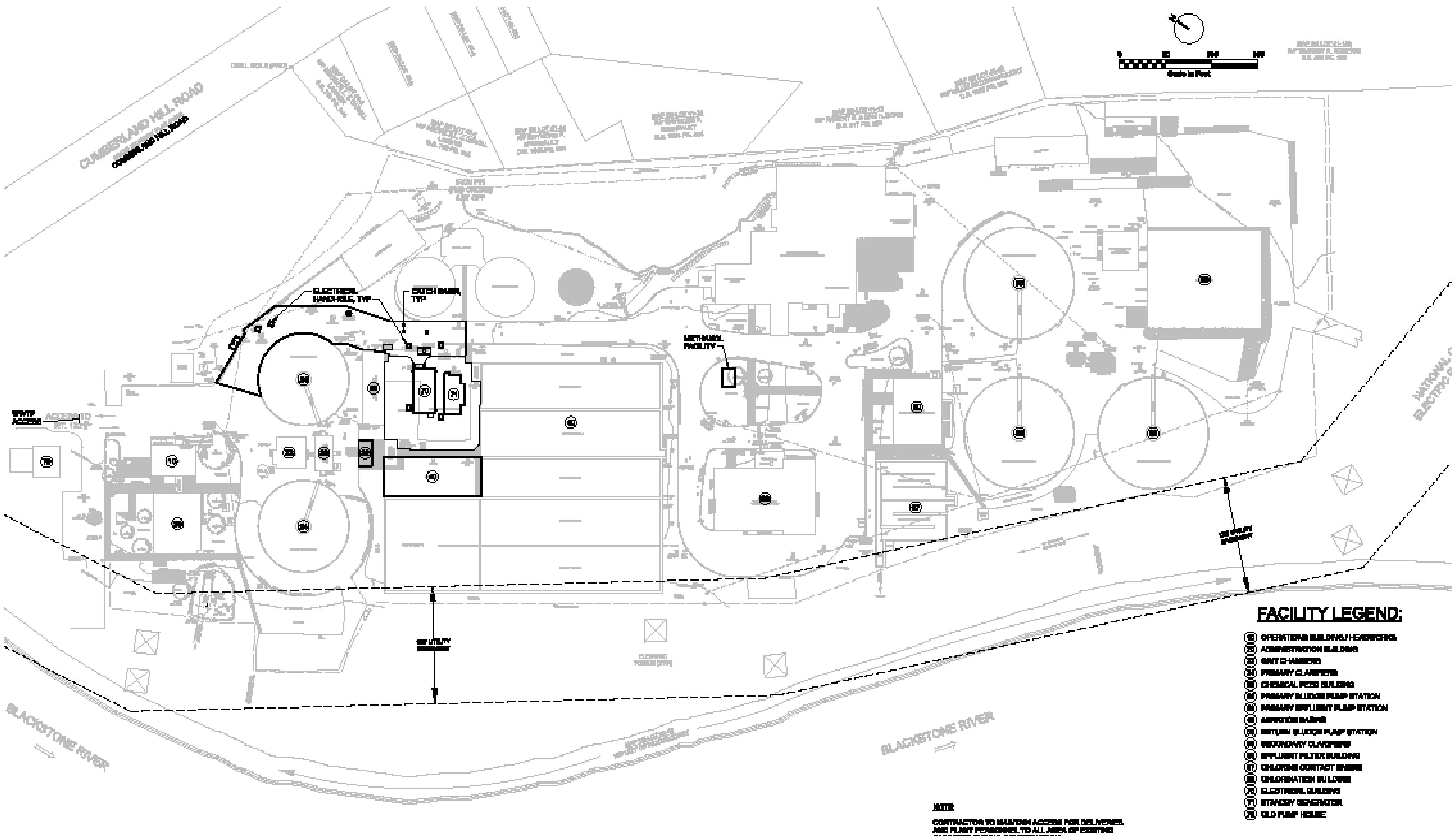
9.2.11.4 480V Main Distribution Switchboard – Operations Building

Based on the observed deficiencies as described in Section 8.5, and the need for modifications, the recommended plan is as follows:

- Replace the existing 480V Main Distribution Switchboard with new main-tie-main main switchgear. The garage will be considered for locating the switchgear as shown in Figure 8-10. As an alternative, the switchgear may be installed in an outdoor enclosure to be located south of the Operations Building The new switchgear will be fed from two new 480V pad-mounted transformers.

9.3 CONCEPTUAL LAYOUT OF PROPOSED IMPROVEMENTS

FIGURE 9-1
PROPOSED IMPROVEMENTS



9.4 PLANNED CAPITAL IMPROVEMENTS PROJECT COSTS

The WWTF design build capital improvements cost is $36,899,314 based on the contract agreement between the City and CH2M HILL for construction and engineering services for the treatment facility modifications to meet the nitrogen and phosphorus removal requirements and to provide the facility modifications including the following:

- New influent pump
- New influent screening facility with odor control system
- New influent flow measurement
- New primary effluent pump station
- Activated sludge modifications to include first and second stage (i.e., Stage A and Stage B) activated sludge basins, lamella plate first stage settling, and new first stage return and waste sludge pumping
- Electrical power system modifications and new standby generator
- HVAC modifications to selected facilities
- I&C modifications

9.5 OPERATIONS AND MAINTENANCE

CH2M HILL is contracted with the City to provide operation and maintenance for the WWTF. The plant operations commenced on October 1, 2012. The operation and maintenance contracted annual service fee subsequent to the capital improvements is $2,028,774.

9.6 ENVIRONMENTAL ASSESSMENT

9.6.1 Introduction

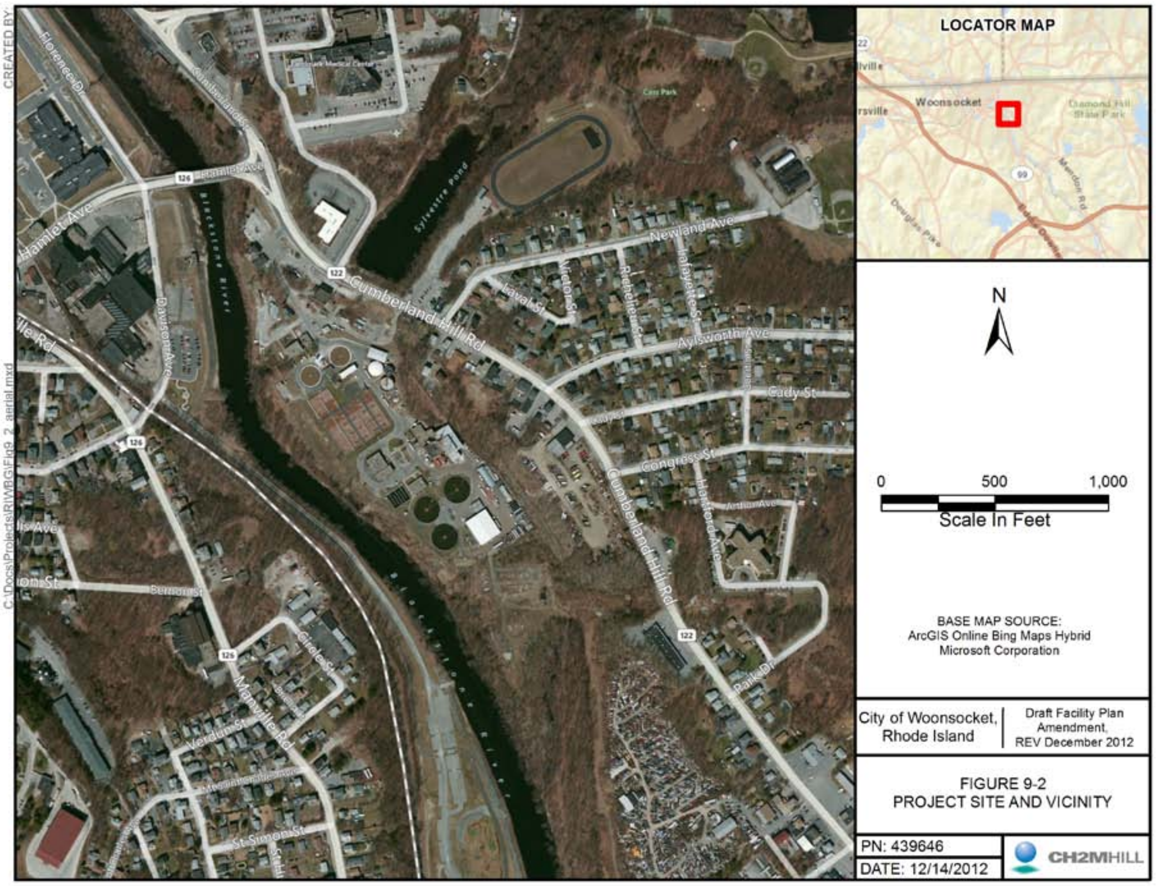
The Environmental Assessment (EA) serves as a basis from which RIDEM can determine whether to issue a Finding of No Significant Impact (FONSI), or require that further environmental review be undertaken in an Environmental Impact Statement (EIS). This section of the Facility Plan Amendment will briefly document compliance with the state review requirements if no EIS is required.

This EA was prepared to comply with the RIDEM *Rules and Regulations for State Revolving Fund (SRF) Program, September 2001* (Regulation #12-190-020). It is meant to meet the requirements of the State Revolving Loan Fund (SRF) by analyzing and evaluating the environmental impacts associated with the project. The EA will provide sufficient evidence and analysis of the effects and impacts on the environment in the vicinity of the proposed project in order that RIDEM may determine the appropriate ruling on the need for further environmental review.

9.6.2 Project Description and Location

The EA presented in this section is for facilities planning for wastewater treatment improvements to the Woonsocket Regional Wastewater Treatment Facility (WWTF) owned by the City of Woonsocket, and operated and maintained by CH2M HILL, Inc. The scope of this EA also includes planned upgrades to the wastewater collection system owned by the City of Woonsocket, and managed in partnership with Veolia Water through a comprehensive underground asset management (UGAM) program.The facility is located at 11 Cumberland Hill Road in Woonsocket, Rhode Island on the eastern bank of the Blackstone River, north of the Cumberland/Woonsocket City Line and just south of the intersection of Cumberland Hill Road with Hamlet Avenue, refer to Figure 9-2, Locus Map. The Blackstone River and its watershed comprise 640 square miles in southeastern Massachusetts and northern Rhode Island. It is a major drainage of the Narragansett Bay Estuary.

FIGURE 9-2
LOCUS MAP



For the purpose of defining a "planning area" for the scope of this EA, the 120-mile wastewater collection system and the WWTF property will be considered the planning area. Upgrades to the existing WWTF are being implemented in order to improve quality of the effluent from the facility and to promote the enhancement of water quality in the Blackstone River and Narragansett Bay overall. Revised stringent RIPDES permit effluent limits for nitrogen and phosphorus cannot be achieved under the facility's current configuration and treatment processes. Reconfiguration of the activated sludge basins to provide additional treatment along with other ancillary improvements will be necessary to allow the WWTF to perform in compliance with its permit discharge requirements.

As a result of the City and VWNA implementation of UGAM throughout the community, several improvements to the wastewater collection system have been accomplished and planned.

Funding for these improvements will be sought through the State Revolving Fund and therefore are discussed in the content of the EA yet should be considered ancillary to the WWTF, which is the focus of the Facility Plan Amendment. The collection system upgrades include, but are not limited to, the following items:

- Sewer expansion along Beausoeil Street, Mendon Road, Comstock Court, Dudley Street, Jillson Avenue, Roy Avenue, Hillview Street, and Cranston Street
- Proposed Capital Repairs FY 2012-13, dig and replace:
  - Sewer main on Wagon Wheel Lane
  - Sewer main on Rachel Street
  - 168-feet of sewer main on Avenue C
  - 50-feet of sewer main on Huntington Street
  - 20-feet of sewer main on River Street
  - 20-feet of sewer main on Lambert Street
  - 10-feet of sewer main on Mt. St. Charles Avenue
  - 75-feet of sewer main on St. Francis Street
  - Entire section of main on Knight Street
  - Entire segment of main on Rose Avenue
  - 6-feet of sewer main on Cumberland Street
  - 6-feet of sewer main on Williams Street
  - Entire segment of main on Walnut Hill Road
  - Entire segment of main on Knollwood Drive
  - Entire segment of main on Grandview Avenue
  - 15-feet of sewer main on Grandview Avenue
  - 10-feet of sewer main on Grandview Avenue

These upgrades will be generally covered within the EA to follow.

9.6.3 Summary of Alternatives Considered

Section 5 and 6 of the Facilities Plan Amendment provide a thorough explanation of the treatment and removal technological alternatives that were considered as WWTF upgrades aimed at complying with the more stringent RIPDES permit effluent limitations for nutrients. A "no build" alternative is not a viable option as the current WWTF is unable able to operate and meet new effluent limits so no additional discussion about this alternative will occur. The alternative technologies that were removed were screened for their potential to lower nutrient effluent levels of the WWTF. Those technologies that could theoretically be applied (installed) at the WWTF and provide the nutrient removal required were further evaluated against practical considerations such as, spatial requirements, cost, compatibility with existing equipment, etc. Ultimately, only one technology was selected for future engineering and design – the preferred alternative – comprising the WWTF upgrade that is the subject of this Plan Amendment.

A description of the preferred alternative – the two-stage activated sludge AB Process – has been provided in Section 6.0. The existing conditions at the WWTF are outlined in Section 2 – Existing Flows and Loads of the Facility Plan Amendment. The physical upgrades to the WWTF are summarized below:

- Existing aeration tanks expanded for additional volume
- New influent pump stations
- New influent screening facility with odor control system
- New influent flow measurement
- New primary effluent pump station
- Activated sludge basin modifications to include retrofit of first and second stage activated sludge basins, lamella plate first stage settling, and new first stage return and waste sludge pumping
- Electrical power system modifications and new standby generator
- HVAC modifications to selected facilities
- I&C modifications

No appreciable increase in the existing footprint of the WWTF will occur as a result of the proposed upgrades described in this Facility Plan Amendment. The greater proportion of upgrades involves modifications to existing equipment and structures to enhance treatment through nutrient removal. Proposed changes related directly to the treatment process will positively impact the effluent discharging from the WWTF thus achieving the desired, mandated water quality improvements. Upgrades to ancillary equipment and associated structures, such as pump stations, electrical system, heating and ventilation, emergency power, may involve new or renovated structures within the WWTF fence line and in previously built-upon or disturbed areas. An EA was included in the 2000 Facility Plan Amendment, and after RIDEM review, a FONSI was issued.

9.6.4 Project Purpose and Need

The purpose and need for this project are a result of the facility's revised RIPDES permit limits. Wastewater collection system upgrades are the result of the implementation of a comprehensive UGAM and needed to improve the function of the wastewater system overall.

Under the new permit, the facility is now required to meet a seasonal total effluent nitrogen concentration of 3.0 mg/L between May 1 and October 31, and a total effluent phosphorus concentration of 0.1 mg/ L between April 1 and October 31 along with a 1.0 mg/L effluent limit from November 1 to March 31. To address these new permit modifications, the City entered into a Consent Agreement (RIA-368) with the RIDEM (finalized and signed February 2011) agreeing to submit a Facilities Plan Amendment to develop a proposed solution and implementation schedule for necessary improvements to achieve compliance with the new effluent total nitrogen and total phosphorus limits. A more detailed description of the permit modifications and the purpose and need for this Facilities Plan Amendment is provided in Section 1.

9.6.5 Direct Environmental Impacts

This section reviews the potential *direct* environmental impacts associated with the planned upgrades to the WWTF and wastewater collection system. Direct environmental impacts are commonly considered to be those impacts that may occur during the construction, operation or maintenance of the proposed activity (i.e. upgrades) at the location of these activities (i.e. WWTF, various points along the collection system). The order of the environmental issues addressed is based on RIDEM’s *Facilities Plan (FP) Review Checklist.*

Traffic & Business

The Woonsocket WWTF is located in an urbanized area immediately adjacent to the Blackstone River. The site sits below surrounding parcels and is accessed by a driveway next to the Woonsocket Fire Department accessed directly from either travel lane of Cumberland Hill Road.

The area is predominantly industrial and commercial although there is residential neighborhood just to the north of the site.

The construction phase of the WWTF Upgrades is estimated to be approximately 2.5 years beginning in March of 2014. The construction will need to be sequenced to maintain plant operations and permit compliance. Construction is required to be completed by January 1, 2017. During the construction phase of the Project, additional vehicle traffic will result from materials, equipment, and workers transporting to the WWTF. Vehicle traffic from workers commuting to the site, and from deliveries of construction material, would moderately increase the traffic flow during normal working hours over the course of the construction period. The additional traffic is expected to be temporary and localized to Cumberland Hill Road and the driveway to the WWTF. Collection system upgrades will take place over the next several years. Related construction activities will occur at specific locations throughout the community on a short-term basis and will, at times, involve work along roads and sidewalks. When work occurs within traffic zones, minor disturbances to traffic flow may be experienced from the diversion of traffic to protect the work zone. Within the context of the urbanized setting and corresponding levels of existing traffic, only negligible to minor direct impacts to traffic in the area can be anticipated due to periodic, short-term increases in congestion. The construction, operation and maintenance of the proposed upgrades will not have any long-term adverse impact on traffic or business.

Historical, Archaeological, Cultural or Recreational

No direct, temporary or permanent, adverse impacts to historical, archaeological, cultural or recreational resources will result from the construction, operation, and maintenance of the proposed upgrades to the WWTF or collection system. The proposed upgrades involve minimal, localized earth disturbance within the WWTF's fence line from equipment traffic and staging. The area has been previously disturbed from previous construction and facility operation so the risk of encountering any previously undiscovered archaeological resources is negligible. Any new structures within the facility or collection system will occur within existing structure footprints and no use or visual impacts to cultural resources are expected.

The most notable recreational resource located within the Project's vicinity that will experience direct impacts from the Project is the Blackstone River. Water quality of the river is expected to improve over time because of the anticipated improvements to the effluent discharging from the

WWTF resulting from the proposed upgrades. The direct impact overall to the river will be beneficial.

Sensitive Ecosystems

No adverse, direct impacts to sensitive ecosystems are expected as a result of the proposed facility or collection system upgrades. There are no known rare, threatened or endangered ecological resources documented in the planning area[2]. The Project is located within the 200-foot riverbank wetland and floodplain of the Blackstone River. Proposed upgrades to the WWTF will involve modifications to existing structures with no anticipated increase in footprint or encroachment on the Blackstone River, 200-foot riverbank wetland or floodplain. No loss of flood storage will result from the proposed structural modifications to the aeration tanks because these modifications will occur within the walls of the WWTF's existing tanks. As previously mentioned, water quality of the river is expected to improve over time because of the anticipated improvements to the effluent discharging from the WWTF resulting from the proposed upgrades. The direct impact to the river and the aquatic ecosystem it supports will be beneficial overall.

Coastal Zone Management

The City of Woonsocket and project site are not located in a coastal zone; therefore, there are no adverse impacts to coastal areas expected. The project will not have any impact on environmentally sensitive coastal areas.

Surface Water Impacts /Erosion & Sedimentation Controls

Negligible impacts to surface water quality from construction-related erosion are expected during the construction of the proposed upgrades. At this time, less than one acre of earth disturbance is estimated during the construction phase of the WWTF Project and collection system upgrades. Therefore, the Project will not require coverage under the *Rhode Island Pollution Discharge Elimination System General Permit for Stormwater related to Construction Activity* (General Permit). However, in accordance with the City's erosion and sediment control ordinance, construction best management practices and appropriate erosion and sedimentation controls will be incorporated into the Project's site plans and implemented, where appropriate. In the event that the final estimate of earth disturbance exceeds one acre, the City will file a Notice of Intent for coverage under the General Permit and complete a Stormwater Pollution Prevention Plan in conformance with RIDEM regulations and technical guidance.

[2]RIDEM, 2012. RI Natural Heritage GIS Datalayer, Environmental Resource Map. Accessed on December 4, 2012 at http://www.dem.ri.gov/amps/index.htm.

Water Quality (effluent – construction & operation)

No adverse direct impacts to the Blackstone River (receiving waters) from the effluent produced from the WWTF are anticipated. The water quality of the Blackstone River and ultimately Narragansett Bay is affected by the quality of the effluent from the WWTF. As discussed in previous sections, the intent of implementing the recommended improvements in this Facility Plan Amendment is to enhance wastewater treatment and reduce final effluent nitrogen and phosphorus concentrations. The upgrades to the WWTF and collection system will ultimately result in improving water quality in the Blackstone River, and therefore no significant adverse impact on surface waters is anticipated.

Displacement of Existing Land Uses

No direct impacts to current land uses, business or households in and around the planning area are anticipated. The WWTF is located entirely on city-owned land zoned for industrial uses. The WWTF was established there in 1897 and has been in operation, with several expansions and modifications over the years, since that time. An old landfill lies upriver on a parcel adjacent to the WWTF site and it is crossed by overhead, high-voltage electrical transmission lines. With the exception of the residential neighborhood lying northeast of the WWTF, the dominant land uses in the area are industrial, commercial, and institutional. The proposed upgrades to the WWTF and collection system will not require a zoning change or alter the existing land uses, on-site or in the immediate vicinity.

Noise

The WWTF and collection system are located within an urbanized, industrialized area of Woonsocket with noise levels consistent with such environments. The residential neighborhoods in the vicinity of the WWTF and collection system would be potential receptors of any additional noise generated from proposed construction activities. The nature and level of existing noise in the area are typical of a busy commercial and industrial area with moderate to heavy periods of traffic. In addition, the proximity of the WWTF to the Woonsocket Fire Department and the Landmark Medical Center result in frequently heard sounds from emergency vehicles sirens. The City of Woonsocket’s Animal Shelter is also located nearby and frequent barking of impounded dogs can be heard in the vicinity of the WWTF. Any receptors of noise from activity at the WWTF would be exposed to these other ambient sounds on a regular basis.

During construction of the proposed upgrades, temporary noise impacts would result from construction traffic and activities. This noise will predominantly occur during normal working hours and will be mitigated as all equipment shall have sound-control devices in working order which are no less effective than that provided when the equipment was purchased. During operation and maintenance of the upgraded WWTF, no appreciable increase in sound-producing activity is expected. Given the urban and industrial character of the WWTF location, noise resulting from the construction, operation and maintenance of the project will be negligible and not be expected to cause adverse, direct impacts to the area.

Visual

No visual impacts will result from the proposed WWTF and collection system upgrades. Physical modifications to the facility will not result in any significant changes to the profile or footprint of the WWTF.

Air Quality

Negligible impacts to air quality are expected from the proposed facility and collection system upgrades. Overall, WWTF upgrades are expected to improve air quality through existing or upgraded odor control; therefore, any direct impacts will be positive. Under implementation of the recommended plan, installation of a new odor control system will help to alleviate any odor issues adjacent to the Operations and Administration Building.

A new standby generator will be installed at the WWTF and it is anticipated that it will be covered under RIDEM’s Diversion of Air Resources General Permit for Smaller-scale Electrical Generation. The generator to be installed will be compliant with RIDEM Air Pollution Control Regulation No. 43 and EPA regulations for nonroad emission sources (43 CFR § 89.112).

In the vicinity of the project site, air quality would be representative of an urban industrialized area. Synagro operates a sludge incinerator immediately adjacent to the WWTF and is categorized as a major source of air pollutants. The incinerator operates under a RIDEM air permit. This plan does not apply to the Synagro operation.

Solid Waste

Small volumes of construction waste will generated during the construction phase of the project. Construction and demolition waste will be hauled away and disposed of properly at an appropriate facility.

Water Supply

No impacts to existing public water supplies will occur as a result of the proposed upgrades. The project site is not located within a public drinking water supply watershed, aquifer protection area, or wellhead protection district. Potable water is supplied to the existing site area by the City of Woonsocket. Construction, operation and maintenance of the proposed upgrades will not result in increased usage of city water.

9.6.6 Summary of Direct Environmental Impacts

Negligible to minor, localized and short-term direct impacts to traffic, noise, air quality, and water quality are possible during the construction of the proposed upgrades to the WWTF and the collection system. Direct impacts can be avoided and minimized in most cases through proper planning and implementation of best management practices throughout the construction phase with respect to vehicle and equipment operation and maintenance; noise control; erosion & sedimentation controls; and waste management. Where applicable, strict adherence to any project environmental permits will also minimize potential environmental impacts. No impacts to sensitive ecosystems, land use, cultural and aesthetic (visual) resources are expected during the construction of the proposed upgrades.

Any issues related to environmental concerns will be brought to the attention of the engineer and/or facility staff immediately and the general contractor will be directed to properly address such issues. Table 9-1 summarizes specific environmental impacts associated with the proposed WWTF and collection system improvements as they compare to existing conditions at the site.

TABLE 9-1
SUMMARY OF DIRECT ENVIRONMENTAL IMPACTS

Environmental Concern	Direct Impact
Traffic	Additional traffic is expected to be moderate during construction. Therefore, no adverse long-term direct impact is expected.
Historical, archaeological, cultural or recreational	No direct adverse impacts
Sensitive ecosystems	No direct adverse impacts. Water quality improvements will benefit the aquatic ecosystem.
Erosion and Sedimentation Controls (of surface waters)	Negligible to minor direct impacts during construction, mainly from vehicle and equipment movement and staging
Land Use	WWTF property is all owned by the City and there is no anticipated impact on existing land use in and around the planning area. No. businesses or household will be displaced as a result of the Project.
Coastal Zone Management	There are no adverse impacts to coastal waters.
Air Quality	Proposed improvements will help to improve air quality; therefore the direct impact is positive.
Visual	No adverse direct aesthetic impacts are anticipated.
Water Quality	Proposed improvements will result in a long-term positive impact on the water quality of the Blackstone River.
Water Supply	Proposed improvements will not have any direct impact on the City of Woonsocket water supply.

9.6.7 Indirect Impacts

Indirect impacts are commonly thought of impacts from an action that are experienced later on or in a spatial context different than the project location. Construction, operation and maintenance of the proposed WWTF and collection system upgrades will not result in any significant long-term indirect impacts. The anticipated indirect impacts will be temporary in nature and may include urbanization, impacts to local businesses and recreation, economics, public safety, visual aesthetics, etc. These possible indirect impacts are discussed in more detail below.

Urbanization

The proposed improvements at the WWTF and collection system upgrades will not result in any additional urbanization of the community. The majority of the City of Woonsocket is densely settled and currently has sewer service; therefore implementation of the proposed project will not result in any unplanned urbanization. Upgrades are planned for existing components of the established system.

Impact on Local Economy

Improvements at the WWTF and collection system have the potential to provide short-term benefits to local skilled and unskilled workers with jobs in various construction trades. Construction also creates a short-term demand for materials such as pipes, soils, pumps and asphalt, plus the work will promote local economic benefits due to worker needs for food, beverage, and equipment. Local vendors will also receive some increase in business during construction.

Recreation

The primary benefit for implementing the proposed improvements at the WWTF is to help improve the water quality of the Blackstone River. Overall improved water quality of Rhode Island's largest river and bay is the main reason for the construction project. Water quality does have an indirect impact on recreational activities in both the Blackstone River and Narragansett Bay, especially during the summer months.

Economics

It is anticipated that there will be minor, short-term indirect impact to the local economy during construction from incidental purchasing activity of goods and services by project workers. Increased nutrient removal from the WWTF effluent is anticipated to improve the water quality of the Blackstone River and may improve public perceptions about the river and promote additional recreational usage.

9.6.8 Future Environment without the Proposed Project

If the proposed recommended improvements are not implemented at the WWTF and collection system, it is expected that the water quality of the Blackstone River would remain the same or worsen with secondary impacts to the riparian ecosystem as a whole. Again, this project is driven by the facility's modified RIPDES permit limitations. Therefore, maintaining the current level of treatment at the WWTF would be in violation of the RIDEM Consent Agreement and RIPDES permit.

9.6.9 Future Environment with the Proposed Project

Overall, the proposed recommended improvements will have a positive impact on the environment. The improvements will include the construction of necessary treatment processes that will allow the WWTF to comply with the current RIPDES permit. Specifically, the facility's ability to remove solids, BOD, nitrogen, and phosphorus will be greatly enhanced. This will certainly result in increased water quality in the Blackstone River. Collection system upgrades will help improve the overall operational efficiency and function of the wastewater system in general.

9.6.10 Agency Review

The RIDEM along with several other state and federal agencies have reviewed this environmental assessment via an Intergovernmental Review. An executive summary along with the EA were sent to review agencies on March 8, 2011, and again the week of December 17, 2012. Per RIDEM guidelines a review period of 21 days was allotted for agency comments. Response letters received by the City are included in Appendix E. RIDEM will either issue a FONSI or will require that an EIS be prepared. Table 9-2 provides a list of the contacts at the respective governmental agencies required to review this document:

TABLE 9-2
LIST OF REVIEW AGENCIES

Agency	Contact
RIDEM, Division of Fish and Wildlife	Mr. Christopher J. Raithel
RIDEM, Office of Technical and Customer Assistance	Mr. Joseph Antonio
U.S. Department of the Interior, Fish and Wildlife Service, New England Field Office	Mr. Anthony Tur
RI Statewide Planning Program	Ms. Nancy Hess
RI Department of Transportation	Mr. J. Michael Bennett, P.E.
Historical Preservation & Heritage Commission	Mr. Edward F. Sanderson, Executive Director
Narragansett Indian Tribal Historic Preservation	Mr. John Brown
Coastal Resources Management Council	Mr. Grover J. Fugate, Executive Director
NOAA/NMFS Habitat Conservation Division, New England Office	Ms. Susan Tuxbury

SECTION 10
PLAN IMPLEMENTATION

10.1 INSTITUTIONAL RESPONSIBILITIES

The City of Woonsocket owns all wastewater treatment facilities including the collection system located within city limits, the treatment plant, and solids handling facilities except for the sludge incineration facilities (Synagro owns incinerator facilities per long-term contract with the City). In June 2012, following a selection process for an Operate-Design Build-Operate (O-DBO) proposal, the City contracted with CH2M HILL to operate and maintain the treatment facility and also to ensure compliance with plant's RIPDES permit. Additionally the City has contracted with CH2M HILL to design-build the nutrient removal upgrades to the treatment facilities, as further defined in the schedule. This includes subcontractor management, procurement of the process equipment, construction, startup activities, performance acceptance testing, scheduling, as required to complete the recommended improvements plan.

Veolia's UGAM Group is performing collection system monitoring and maintenance, and Synagro is under contract with the City to operate and maintain the solids handling/incinerator facilities at the treatment facilities.

10.2 PLAN IMPLEMENTATION

CH2M HILL is currently under contract to complete the following items as scheduled below:

- Submit Facility Plan Amendment by December 17, 2012
- Approval of the Final Facility Plan by May 17, 2013
- Order of Approval by November 1, 2013
- Initiate Construction of the Approved Modification by March 1, 2014
- Complete Construction of the Approved Modifications by January 1, 2017
- Meet Final Nutrient Limits by May 1, 2017

This plan will be implemented over approximately a 5 year Final Design and Construction period. The connection to the Synagro Co-generation Facility will be implemented in the future by the City. The Nutrient Removal Upgrades will be constructed starting in March 2014 and will require a construction sequence that will allow the treatment plant to remain in service and meeting permit requirements during construction.

A preliminary schedule of Implementation is included in Figure 10-1.

FIGURE 10-1
WOONSOCKET, RI WWTF
FACILITY PLAN AMENDMENT
PROJECT IMPELMENTATION SCHEDULE

Phase and Tasks	Dates
Respond to RIDEM Comments and Submit Upda	December 17, 2012
RIDEM Review	Dec 17 - Mar 28, 2013
Respond to RIDEM Comments and Submit Final	Mar 29 - May 3, 2013
RIDEM Approval of Final Facilities Plan	May 17, 2013
Early Tasks	
Site visit - electrical and I&C investigation	Oct 15 - 19, 2012
Survey and develop mapping	Oct 15 - 26, 2012
Design	
30% - Schematic Design	Jun 14 - Jul 30, 2013
30% - Schematic Design QC	Jul 31 - Aug 13, 2013
60% - Design Development	Aug 14 - Oct 16, 2013
60% - Design Development QC	Oct 17 - 30, 2013
90% - Construction Documents	Oct 30 - Dec 27, 2013
90% - Construction Documents QC	Jan 2 - 15, 2014
100% - Design	Jan 16 - Mar 1, 2014
Construction	
Construction	Mar 1, 2014 - Jan 1, 2017
Substantial Completion	October 1, 2016
1st Performance Testing	Dec 1 - 30, 2016
2nd Performance Testing	May 1 - 30, 2017
Revised Permit	May 1, 2017

10.3 PUBLIC WORKSHOP AND MEETING

An important part of this project is to maintain communication with the public. A Public Workshop was held on February 21, 2011 to introduce the public to the project, review status, and to obtain public comment. The workshop was advertised by the City Department of Public Works in advance of the meeting. A copy of the article is included in Appendix D.

The Public Workshop/Hearing was structured to review project goals and objectives, review completed Facility Plan Amendment work tasks, present the nutrient removal treatment technologies evaluated, and present the preliminary recommended improvement plan. The presentation included descriptions, figures, and technical summaries for each of the evaluated alternatives and the recommended tertiary treatment solutions. The material presented at the public workshop is included in Appendix D.

Appendix E also contains a listing of the Intergovernmental Review Contacts who were contacted and informed of the upgrade being proposed at the Woonsocket WWTF under this Facility Plan Amendment. In addition, copies of letters sent out to the Agency Review contacts, certified mail receipts, and any responses from the various agencies are also included in this appendix.

A final Public Hearing is scheduled for May 29, 2013. The intent of the hearing will be to inform the public of the treatment options that were evaluated along with recommended facility improvements. In addition, the public hearing will outline the various other facility upgrades that are required for the WWTF to address concerns with aging infrastructure systems.