FACILITY PLAN
for the
CITY OF TORRINGTON, CT
WATER POLLUTION
CONTROL FACILITY (WPCF)

October 2012

FINAL DRAFT

WRIGHT-PIERCE
Engineering a Better Environment™

TORRINGTON, CONNECTICUT

WPCF FACILITIES PLAN

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OCTOBER 2012

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CITY OF TORRINGTON, CONNECTICUT

WPCF FACILITIES PLAN

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EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

The City of Torrington owns and operates the Torrington Water Pollution Control Facility (WPCF). The WPCF was originally constructed as a primary treatment plant in 1935, and was upgraded to provide secondary treatment and sludge processing facilities in 1970. There have been additional modifications to the facility since the 1970 upgrade. The most recent process related improvements were completed in 1994, when nitrification and improved disinfection facilities were provided. A Regional FOG Receiving Facility was constructed and became operational at the Torrington WPCF in 2010.

Currently, the City of Torrington is facing a variety of challenges at the WPCF including:

- Increasingly stringent nitrogen removal requirements with increasing nitrogen credit costs, along with uncertainty over the long-term viability of the credit trading program.
- Stringent phosphorus limits as part of the WPCF's upcoming discharge permit renewal.
- Aging, energy inefficient unit processes, equipment, and building systems with increasing operating costs and increasing corrective maintenance requirements.
- The need to identify an improved biosolids disposal plan to address increasing disposal costs.

PURPOSE OF EVALUATION

The City decided to conduct a comprehensive wastewater facilities planning evaluation of the WPCF for several reasons including:

- The age and condition of the existing facility. The last plant upgrade was completed and came on line in 1994 and is approaching 20 years in service. In addition, much of the existing infrastructure such as buildings, concrete structures and electrical power distribution systems, dates back to the original secondary treatment upgrade constructed in the early 1970s.

- Completing a single, comprehensive upgrade at the WPCF would maximize the availability of funding assistance through the state's Clean Water Fund. The DEEP informed the City that the planned nitrogen removal project at the Torrington WPCF is highly ranked on the state's 2008/2009 Priority List "Future Fundable Projects" list. The DEEP indicated that a comprehensive upgrade of the WPCF could be done under the denitrification project to take advantage of the current ranking of the nitrogen removal project on the Priority List. The additional study effort qualifies for a 55% planning grant while the design and construction phases qualify for approximately a 20% grant, with Nitrogen and Phosphorus removal upgrade technology eligible for a 30% grant. The CT DEEP CWF will typically calculate an grant percentage to apply to all eligible costs based on the 20% and 30% values; this typically ranges between 23% to 24% grant applied to the entire project with the remainder available as a 2% interest loan over 20 years.
- The DEEP has indicated that if improvements were completed as separate projects, only the nitrogen and phosphorus removal project would receive sufficient priority points to have the potential to be reached by available funding. This means that DEEP grant/loan funds would likely not be available for future phases of the project.
- The DEEP has issued its phosphorus removal program for WPCF's throughout the state with freshwater discharges. Torrington will receive a seasonal phosphorus limit with an equivalent concentration requirement of 0.32 mg/l.

This Facilities Plan builds upon the previous facilities planning study and solids handling evaluations (*Draft Facilities Planning Study*, Wright-Pierce, 2007) to develop a comprehensive evaluation of the Torrington WPCF, with the goal of identifying the upgrade needs to meet current and future projected requirements of the City of Torrington and those areas of adjacent communities served by Torrington's water pollution control facilities. As part of the Facilities Plan, opportunities to increase the facility's efficiency in order to control operating costs were also investigated. The facility plan is also required to allow any modifications to the WPCF to qualify for financial assistance through the state's Clean Water Fund and the plan has been prepared to meet the requirements of the Connecticut DEEP to be eligible for the Clean Water Fund priority list.

BASIS OF DESIGN

The flows and loads design basis are presented in Table ES-1 below for both the initial year of operation (2014) and the design year (2035).

TABLE ES-1
CITY OF TORRINGTON WPCF FACILITY PLAN
INFLUENT FLOW AND LOAD BASIS OF DESIGN

	Min Day[1]	Annual Average	Max Month	Max Day[2]	Hydraulic Peak
EXISTING - Raw Influent (Including Septage and Grease)					
Flow, mgd	2.84	5.51	11.62	21.98	22.50
Flow, mgd (98th percentile)				12.74	15.50
BOD5, lb/d		5,588	8,613	10,671	
TSS, lb/d		5,429	9,099	11,741	
TKN, lb/d		1,009	1,226	1,688	
FUTURE - Raw Influent (Including Septage and Grease)					
Flow, mgd	3.25	6.31	13.31	25.17	25.76
Flow, mgd (98th percentile)				15.93	18.76
BOD_5, lb/d		7,263	10,607	13,925	
TSS, lb/d		7,101	11,293	15,219	
TKN, lb/d		1,332	1,607	2,308	

Notes:
1. Minimum Day based on 3rd percentile value
2. Existing Maximum Day BOD_5 and TSS Loadings based on 98th percentile values

NITROGEN REMOVAL

The State of Connecticut's *General Permit of Nitrogen Discharges* was developed to address water quality issues in Long Island Sound. Based on discussions with DEEP officials, it appears that there is a strong possibility that the State will not be in compliance with the Long Island Sound Total Maximum Daily Load (TMDL) for nitrogen by 2014. Non-compliance may result in the State requiring each publicly-owned treatment works (POTW) to meet the 2014 nitrogen limit through treatment without the option to purchase nitrogen credits. Because of this, alternatives to reduce nitrogen to the 2014 limits at the WPCF without the need to purchase nitrogen credits was evaluated as part of the overall nitrogen removal evaluation. Concurrent with the evaluation of long-term nitrogen reduction alternatives, an assessment was made of short-term improvements that could be made to reduce effluent nitrogen until a full-scale upgrade could be implemented.

Short-Term Nitrogen Reduction

The plant staff has implemented operational strategies within the confines of the existing facilities to allow for increased nitrogen and phosphorus removal. The plant staff recently installed mechanical mixers in the anoxic zones, instead of using air to mix the anoxic zones. Additional intermediate improvements that can be done at the Torrington WPCF include increasing the internal recycle pump capacity to provide a better environment for denitrification and potentially decreasing the overall total nitrogen effluent concentration..

Long-Term Nitrogen Reduction

Potential process alternatives to enhance the nitrogen removal capabilities of the WPCF were developed and evaluated using the BioWin process model. Compliance with the *General Permit for Nitrogen Discharges* is achieved either by meeting the annual total nitrogen limit through treatment or through the purchase of equivalent nitrogen credit. The alternatives evaluated included those that achieve compliance with the *General Permit* either through the purchase of credits, the removal of the nitrogen, or a combination of both. The following process alternatives were developed to address the issues facing the Torrington WPCF:

- Do Nothing Alternative;
- Modified Ludzack-Ettinger (MLE) Process;
- Four-Stage Bardenpho Process;
- Four-Stage Bardenpho Process with IFAS;

Of these, the "Do-Nothing" alternative would rely on the continued purchase of nitrogen credits. The MLE process would provide for some level of additional nitrogen removal but would not meet the 2014 *General Permit* limit without the need to purchase credits. The four-stage Bardenpho process and the four-stage Bardenpho with IFAS process can each meet the *General Permit* limits.

The four-stage Bardenpho process alternative is recommended for implementation because it offers the following benefits:

- This process alternative can achieve the 2014 total nitrogen limit without the need to purchase equivalent nitrogen credits. There is a strong possibility that the State will not be in compliance with the Long Island Sound TMDL and therefore, may require each POTW to meet the 2014 limit through treatment (i.e. elimination of the nitrogen credit trading program).

- Of the two process alternatives that achieve the 2014 nitrogen limits, the four-stage Bardenpho process alternative has the lowest capital cost.

- The layout of the four-stage Bardenpho process allows for greater process flexibility than the other alternatives. Specifically, the ability to operate in alternative configurations, alternative aerated/unaerated zones and the number of aeration tanks online.

PHOSPHORUS REMOVAL

The DEEP has recently issued the phosphorus limits that will be incorporated into Torrington's permit when it is renewed. The WPCF will be required to comply with the new limits within four years after issuance of the new permit. The WPCF will need to meet a seasonal average limit between April 1 and October 31 of 17.29 lb/d.

At the current annual flow of 5.5 mgd, the seasonal average limit is equivalent to an effluent phosphorus concentration of 0.37 mg/l. At the design plant flow of 6.3 mgd, an average effluent concentration of 0.32 mg/l will need to be achieved. The total phosphorus limit determines the treatment technology requirements to achieve that limit. Given the long-term capital and operational cost implications associated with the selection of a phosphorus removal technology, the potential for future reductions of total phosphorus effluent limits were considered. Based on statements from the DEEP that these limits could be reduced in the future, the facilities planning evaluation considered that future effluent total phosphorus limits could conceivably be as low as 0.1 mg/l.

To achieve the effluent limits identified by the DEEP, a tertiary treatment process is recommended. Several alternatives were evaluated including chemical precipitation followed by:

- Effluent filtration using cloth-disk filters
- Effluent filtration using granular media filters.
- Ballasted flocculation.

Of these only the ballasted flocculation and granular media filter processes would meet the seasonal mass limit at the future design flow rate of the facility and offer the potential to achieve limits as low as 0.1 mg/l should more stringent limits be incorporated into future permits. Should limits below 0.1 mg/l be required, a future, second-stage filtration process could be incorporated downstream of one of these tertiary processes.

LONG-TERM BIOSOLIDS HANDLING

Wright-Pierce completed a sludge disposal study for the Torrington Water Pollution Control Authority (WPCA) in 2002. The objectives of the study were to evaluate the anticipated future cost of liquid versus dewatered sludge disposal, options for dewatering of the thickened sludge, and other potential improvements to the existing solids handling facilities. The evaluation of the solids handling process was updated in 2007 and again in this Facilities Plan Report for the City of Torrington WPCF.

As part of this Facilities Plan, the following objectives were identified:

- Calculate future design year 2035 sludge production rates, including tertiary sludge;
- Investigate sludge hauling and disposal costs for liquid and dewatered sludge to determine how the market has changed since the last evaluation update;
- Evaluate cost alternatives for waste sludge thickening;
- Evaluate dewatering options and determine capital improvement costs; and
- Develop a layout/site plan for solids handling improvements.

As part of this solids handling evaluation, multiple alternatives were evaluated, including the alternative to dewater liquid sludge prior to off-site disposal. This evaluation considered three alternatives for future solids handling as described below.

Option 1- Continue to Dispose of Thickened Liquid Sludge

- In the future, after the addition of a Tertiary System, co-settle tertiary sludge (seasonally) and primary sludge in primary clarifier. Thicken co-settled sludge in gravity thickener;
- Do not co-settle Waste Activated Sludge (WAS); store WAS in unthickened sludge holding tanks. Use mechanical sludge thickening equipment to thicken WAS; Store thickened WAS separately from thickened primary/tertiary.
- Haul thickened liquid sludge and dispose of off-site at an incinerator facility.

Option 2- Dewater Thickened Liquid Sludge

- Use same thickening process as summarized in Option 1;
- Blend thickened primary/tertiary and WAS in a blend tank. Blend sludge at 6% solids;
- Dewater blended/thickened sludge using dewatering technology; and
- Haul dewatered sludge and dispose of off-site at an incinerator facility.

Option 3 - Dewater Thickened Primary and Tertiary Sludge and Unthickened WAS

- Co-settle tertiary sludge (seasonally) and primary sludge in primary clarifier. Thicken co-settled sludge in gravity thickener;
- Do not thicken WAS. Store WAS in unthickened sludge holding tanks;
- Blend thickened primary/tertiary sludge and unthickened WAS in a blend tank.
- Dewater the blended sludge using dewatering technology
- Haul dewatered sludge and dispose of off-site at an incinerator facility

A life-cycle cost analysis revealed that each of the three alternatives has similar net present values and no one alternative was identified as offering a significant cost advantage. To determine the recommended alternative, a comparative analysis considered other impacts, including ease of operation and maintenance, flexibility and expandability, constructability and construction sequencing, recycle flow impacts, site availability, odor potential and safety. Based on this assessment, Option 3, Dewater Thickened Primary and Tertiary Sludge and Unthickened WAS, offered the most advantages, including:

- Cost differential of disposing and hauling of dewatered sludge versus liquid sludge.
- Sludge storage capacity available in existing tanks for thickened and unthickened tertiary/primary, waste, and blended sludge.
- Impact on the overall treatment process, including Operation and Maintenance requirements for each option, chemical storage/usage for thickening/dewatering process and the footprint/building configuration and space available for required selected equipment.
- Overall cost for each option. A Net Present Worth cost evaluation of each option was completed to determine the most economically feasible alternative.
- Hours and reliability of operation for each option, including redundancy of process and complexity of the selected equipment.

RECOMMENDED CAPITAL IMPROVEMENT PROJECT

In addition to the evaluation of nitrogen and phosphorus removal and solids handling and disposal, the Facilities Plan included an evaluation of all other unit processes, building systems, instrumentation and control and electrical service and distribution. Recommendations for these unit processes primarily included replacement of aging equipment with more modern, energy efficient equipment and systems, rehabilitation of aging structures, providing updated instrumentation and control systems, and implementing building system improvements to improve the energy efficiency of existing buildings. The overall recommended improvements to the WPCF include the following:

Screenings Building

- Provide a concrete spill containment wall for the Vactor truck dumping area upstream of the mechanical screens.

- Replace the existing mechanical bar screens with finer screening equipment that is more efficient and less susceptible to maintenance concerns.

- Install screenings handling (grinding, washing, dewatering, compacting, and disposal) equipment.

- Remove concrete flooring above the screen channels and install aluminum diamond plating with integral access panels.

- Expand the building to provide sufficient access to all equipment for operation and maintenance activities.

- Provide odor control for the headworks area and incorporate the Screenings Building and Septage Receiving Facility. Include covering and exhausting air from the Screenings Building, Septage Receiving Facility, Siphon Chamber, and Primary Sludge Degritting Facility and treating odorous air. At this time, a containerized biofilter system is recommended. However, during preliminary design, available alternatives should be considered and re-evaluated.

Septage Receiving Facility

- Install a stand-alone septage receiving pretreatment unit that would screen incoming septage, and automatically meter and record the volume of septage from each hauler.

- Replace or modify the septage tank mixer to provide more efficient mixing.

- Relocate the septage pump piping discharge to a location upstream of the mechanical screens.

- Provide a new submersible chopper-type septage pump in the septage holding tank. Either leave the existing septage pump in place as a standby unit or purchase a second submersible chopper pump as a shelf spare.

Primary Settling Tank

- Construct a fourth primary settling tank.
    - The configuration and dimensions of the new tank would replicate the existing tanks.
    - Effluent from the fourth primary settling tank would be directed to Distribution Box No. 2.

- Modify the existing or construct a new primary influent distribution box. Modify influent distribution within each tank
- Modify primary effluent distribution. Relocate the RAS discharge to the primary effluent distribution box.
- Modify the influent baffles within each existing tank to dissipate inlet velocity and reduce hydraulic short-circuiting.
- Reconfigure the drive location for the primary sludge cross collectors. The cross collector drives should be placed on the wall opposite the primary sludge pump suction so that sludge and grit are pushed away from the drive end of the screw.
- Provide automated scum removal.
- Provide five new primary sludge pumps with VFDs.
- Provide dehumidification in the Primary Pump Gallery.
- Provide odor control for the primary clarifiers. Including covering and exhausting air from the influent and effluent distribution boxes as well as the primary effluent launders and treating odorous air. At this time, a containerized biofilter system is recommended. However, during preliminary design, available alternatives should be considered and re-evaluated.

Biological Wastewater Treatment

- Modify each of the four aeration tanks to operate in the Four-Stage Bardenpho Process configuration for nitrogen reduction.

- Subdivide the anoxic zone so that, if necessary, a portion of the anoxic zone could be operated anaerobically for biological phosphorus removal. Maintain diffusers in anoxic zones so that they can be operated aerobically, if necessary, during cold weather periods to maintain nitrification.

- Replace the existing nitrate recycle pumps in each aeration tank with higher capacity units. Provide multiple discharge locations to allow for operating an anaerobic zone ahead of the anoxic zone, if necessary. Provide VFDs for the nitrate recycle pumps.

- Evaluate the existing hoists for nitrate recycle pumps in Aeration Tanks No. 3 and No. 4 to determine if they can be reused with the larger recommended recycle pumps. If possible, relocated and reuse hoists with new system.

- Complete cost evaluation to determine if it is beneficial to replace the existing blowers with new more efficient blowers at future air demands for the recommended Four-Stage Bardenpho Process. Also evaluate reliability of remaining useful life of existing blowers.

- Pipe effluent from Aeration Tanks No. 3 and No. 4 to Aeration Tanks No. 1 and No. 2 influent channel. Pipe effluent from Aeration Tanks No. 1 and No. 2 to Distribution Box No. 5. Raise the effluent weir in Aeration Tanks No. 1 and No. 2 by approximately 12 to 18 inches to provide additional hydraulic head.

- Replace existing isolation valves on aeration system drop pipes.

- Install additional Nitrate, ORP and pH meters in the aeration tanks (as necessary).

- Install a Supplemental Carbon and Alkalinity adjustment chemical feed system.

- Construct one new 80-foot diameter circular final settling tank with a 16-foot side water depth (SWD) (Final Settling Tank No. 6). Consider installation to the south of Final Settling Tank No. 5.

- Modify the existing Distribution Box No. 5 or construct a new distribution box for Final Settling Tanks No. 4, No. 5, and No. 6.

- Remove concrete overflow structure to storm drain in Final Settling Tanks No. 4 and No. 5.

- Provide algae sweeps and full-radius scum removal on all circular final settling tanks.

- Provide turbidity meters at effluent end of all final settling tanks.

- Provide new RAS/WAS pumps in the existing pump room for new Final Settling Tank No. 6.

- Replace VFDs on existing RAS pumps.

- Clean and repair concrete cracks and wall penetrations that contribute to water leakage in the pump room.

- Seal below grade electrical conduit to prevent groundwater leakage, or provide above-grade junction boxes for the pump room.

- Provide pipe and valve modifications in Secondary Pump Gallery No. 2 to facilitate tank draining and isolation.

- Provide dehumidification in Secondary Pump Gallery No. 2.

Disinfection and Effluent Discharge

- Provide new outfall pumps to replace aging system.

- Provide a catwalk around the outfall pumps to facilitate access.

- Provide hoist for outfall pump removal.

- Provide in-line chlorine analyzers.
- Provide effluent flow monitoring.

Tertiary Treatment

- Install new Ballasted Flocculation Tertiary Treatment Process (Actiflo)
- Install Tertiary Treatment equipment room and Tertiary Treatment influent pump station.
- Install Chemical storage and feed equipment room. This room shall also include chemical storage and feed equipment for alkalinity adjustment for the secondary treatment process.

Sludge Disposal

- Provide coating on interior of thickened sludge storage tanks to reduce potential for corrosion.
- Abandon the existing gravity belt thickener and install three screw press type dewatering equipment.
- Replace the existing gravity belt thickener feed pumps with two progressing cavity or rotary lobe pumps, which will be used to feed the screw press type dewatering equipment.
- Provide magnetic flow meters downstream of the dewatering feed pumps and the thickened primary sludge pump.
- Improve mixing capabilities in sludge holding tanks.
- Provide overflow drains on the sludge storage tanks
- Cover primary sludge thickener and sludge holding tanks.
- Provide an odor control system for the covered tanks, the truck loading area, and the secondary sludge thickening area. At this time, a containerized biofilter system is

recommended. However, during preliminary design, available alternatives should be considered and re-evaluated.

Plant Support Systems and Facilities

- Construct new Maintenance Building/Garage.
- Provide a slide rail system in all liquid process tanks for a portable dewatering pump.
- Provide air compressors and compressed air distribution systems in each building or pump gallery.
- Upgrade HVAC system in the Administration Building and Operations Building.
- Provide an overhead crane and testing bench in New Maintenance Building Garage.
- Replace emergency generator.
- Expand the laboratory in the Administration Building.
- Provide pressure regulator on incoming potable water line.
- Upgrade roof drainage system on each building.
- Demolish the abandoned Burrville Wastewater Treatment Plant.

The recommended capital improvement project costs are summarized in Tables ES-2 and ES-3. Total project capital costs include an allowance of 20% for Contractor Overhead and Profit, a 20% contingency for unaccounted for items and 5% for changes during construction. In addition, an allowance of 15% has been included for technical services during the design and construction phases along with a 1% allowance for financing, legal and administrative costs. The project cost information presented herein is based on 2012 costs and was inflated at 4% per year for two years to the assumed mid-point of construction (2014). The total project capital cost is estimated to be $51,300,000.

TABLE ES-2
CITY OF TORRINGTON WPCF FACILITIES PLAN
SUMMARY OF RECOMMENDED IMPROVEMENTS CAPITAL COST
(AUGUST 2012)

DESCRIPTION			COST
SITE WORK			$3,035,000
PROCESS PIPING			$1,451,000
SCREENINGS BUILDING MODIFICATIONS			$1,106,000
PRIMARY CLARIFIER #4 / INFLUENT & EFFLUENT BOXES			$1,362,000
SEPTAGE RECEIVING FACILITY			$465,000
SECONDARY TREATMENT MODIFICATIONS			$2,893,000
NEW SECONDARY CLARIFIER #6			$1,240,000
TERTIARY TREATMENT FACILITY			$4,329,000
EFFLUENT / DISINFECTION FACILITIES			$728,000
ODOR CONTROL SYSTEMS			$624,000
NEW SLUDGE HANDLING FACILITIES			$1,386,000
MAINTENANCE / EQUIPMENT STORAGE / GARAGE			$1,750,000
LABORATORY FACILITIES EXPANSION			$275,000
ROOF DRAINAGE SYSTEM, EACH BUILDING			$100,000
MISCELANEOUS BUILDING REPAIRS			$327,000
		SUBTOTAL	$21,071,000
SPECIALS			$505,000
HVAC/PLUMBING			$1,780,000
INSTRUMENTATION			$750,000
ELECTRICAL			$2,725,000
		SUBTOTAL	$5,760,000
SUBTOTAL, GENERAL CONSTRUCTION			$21,071,000
GENERAL CONTRACTOR OH&P AND GENERAL CONDITIONS	20.0%		$4,214,000
SUBTOTAL, SUBCONTRACTORS			$5,760,000
GENERAL CONTRACTOR MARKUP	7.5%		$432,000
BONDS & INSURANCES	2.0%		$630,000
UNIT PRICE ITEMS (Ledge Excav., Additional Materials, etc.)	2.0%		$421,000
SUBTOTAL, CONSTRUCTION COSTS			**$32,528,000**
PROJECT MULTIPLIER, DESIGN CONTINGENCY, 20%	1.20		$39,033,600
TOTAL 2014 CONSTRUCTION COST (2 Yrs @ 4% INFLATION).	1.08		**$42,200,000**

TABLE ES-3

CITY OF TORRINGTON WPCF FACILITIES PLAN

TOTAL ESTIMATED PROJECT COST

(AUGUST 2012)

PROJECT COMPONENT		**ESTIMATED COST**
CONSTRUCTION ESTIMATE		$42,200,000
CONSTRUCTION CONTINGENCY	**5.0%**	$2,110,000
TECHNICAL SERVICES	**15%**	$6,330,000
VALUE ENGINEERING		$150,000
LEGAL/ ADMINISTRATIVE/FINANCING	**1.0%**	$420,000
ENGINEER'S ESTIMATE OF PROBABLE PROJECT COST		$51,300,000

One implementation approach would be to construct all of the recommended improvements as a single project. However, the facility currently does not have any major compliance problems, is well maintained and operated, and appears to be able to currently achieve its phosphorus removal requirements. Therefore, another approach would be to prioritize the improvements for implementation in a phased approach to match available funding. Based on discussions with the City of Torrington, an implementation plan was developed for the recommended improvements to be completed in a phased approach as part of a long-term capital improvement plan. The breakdown of the proposed implementation phases are summarized as follows:

- Phase 1:
    - Preliminary Treatment Improvements
    - Septage Receiving Improvements
    - Preliminary Treatment Odor Control System
    - Construction of fourth Primary Clarifier and Primary Clarifier Odor Control System.
    - Secondary Treatment Improvements including conversion to Four-Stage Bardenpho Process, modifications to existing Final Settling Tanks and construction of Final Settling Tank No. 6 and associated pumping systems.
    - Solids Handling Improvements and Solids Handling Odor Control System.
    - Implantation of the improvements to the existing buildings

  - Demolition of the Burrville facility
  - WPCF and Pump Station security improvements
- Phase 2:
  - Installation of the tertiary treatment system
  - Installation of tertiary treatment influent pumps station

The total construction cost of the two phase construction approach would be $43,800,000 (as opposed to $42,200,000 for a single construction project). If the City decided to implement the Torrington WPCF upgrade project in two separate phases, the construction cost would be approximately $1.6 million more than implementing the entire project in one phase.

SECTION 1

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SECTION 1

INTRODUCTION

1.1 INTRODUCTION

The City of Torrington owns and operates a secondary Water Pollution Control Facility (WPCF) designed to treat an average daily (ADF) flow of 7.0 million gallons per day (mgd) and a peak hourly flow of 20.0 mgd. The WPCF discharges to the Naugatuck River.

The WPCF originally constructed as a primary treatment plant in 1935, was upgraded to provide secondary treatment and sludge processing facilities in 1970. There have been additional modifications to the facility since the 1970 upgrade. The most recent process related improvements were completed in 1994, when nitrification and improved disinfection facilities were provided. A Regional FOG Receiving Facility was constructed and became operational at the Torrington WPCF in 2010.

Current and upcoming regulatory requirements, particularly concerning nitrogen and phosphorus reduction will require the WPCF to meet more stringent effluent limits. The operation and maintenance of the sludge handling system components is also a focus of this evaluation. In addition, there are architectural, structural, mechanical, instrumentation and electrical components that will need to be addressed to ensure that the facility meets its future requirements and to provide long term reliable operation of the WPCF.

The goal of the study is to evaluate the long-term upgrade needs of the WPCF including:

- Improvements to cost-effectively meet nitrogen reduction goals.
- Phosphorus reduction strategies to meet the interim goals currently being drafted by the CT DEEP.
- Equipment and systems upgrade requirements to ensure long-term reliability.
- Improvements to the septage handling facilities to accommodate regional disposal needs.

Lastly, the City of Torrington is interested in evaluating the overall effectiveness and efficiency of the energy use of their wastewater facilities, specifically the Water Pollution Control Facility and 7 of their largest pump stations.

1.2 STRUCTURE OF THE FACILITIES PLANNING STUDY

The Facilities Planning Study Report presents a summary of the evaluations completed, the findings and conclusions drawn from these evaluations, and the recommended improvements to ensure continued regulatory compliance. The report is divided into several sections, including this Introduction. A brief discussion of the contents of the subsequent sections is presented below.

Section 2 - Discharge Standards and Regulatory Requirements

The Torrington WPCF discharges to the Naugatuck River. The discharge is regulated under a National Pollutant Discharge Elimination System (NPDES) permit, administered by the State of Connecticut Department of Energy and Environmental Protection (DEEP). Discharge limitations in the current NPDES permit are provided to maintain the present and future water quality classification of the Naugatuck River. In addition, the Torrington WPCF is also subject to the *General Permit for Nitrogen Discharges* (Nitrogen General Permit). Section 2 presents the discharge limitations for the Torrington WPCF as well as a discussion on the WPCF's compliance with current requirements.

More recently, the DEEP has published an effluent phosphorus limit for the Torrington WPCF. It is anticipated that when the Torrington WPCF NPDES permit is renewed, it will contain a compliance schedule to meet this phosphorus limit.

Section 3 - Wastewater Flow and Loads

The Torrington Water Pollution Control Facility (WPCF) receives flows primarily from the City of Torrington with some additional flow received from the Towns of Litchfield and Harwinton. A portion of the influent organic loading, although a relatively small portion of the influent flow, is from trucked-in wastes such as septage and grease tank pumpings from the City and surrounding region.

To establish the basis for evaluation of the Torrington WPCF, projections of future wastewater flow and loadings have been made. Section 3 presents a summary of the current and projected future flows and loadings which are used as the basis of the facilities planning evaluation.

Section 4 - Existing Facilities and Operation

Section 4 presents an assessment of the existing facilities and current operating conditions. This assessment sets a baseline for subsequent evaluations of the existing liquid process and solids disposal facilities, including all ancillary support systems. The existing physical and operating parameters were summarized to facilitate the determination of their capability to meet current and future regulatory requirements at projected future flows and loads as well as their long term reliability.

Section 5 - Evaluation of Existing Primary Treatment Process Facilities

Alternatives for the upgrade of the liquid process facilities that are components of the primary treatment system were developed and are presented in Section 5. The evaluation is based on the current condition of the WPCF and an evaluation of the capacity of the existing facilities to meet current and future regulatory limits.

Section 6 - Evaluation of Existing Secondary Liquid Process Facilities

Alternatives for the upgrade of the secondary liquid process facilities, including the disinfection and post aeration facilities, were developed and are presented in Section 6. The evaluation is based on the current condition of the WPCF and an evaluation of the capacity of the existing facilities to meet current and future regulatory limits for the Basis of Design.

Section 7 – Phosphorus Removal Evaluation

Section 7 presents the evaluation of phosphorus reduction alternatives for the Torrington WPCF based on their current draft Total Phosphorus average seasonal load cap of 17.29 pounds/day, which equates to a concentration of 0.4 mg/l based on the flows utilized by CT DEEP (2001-2007). The Total Phosphorus limit is anticipated to be given to Torrington in their upcoming NPDES permit renewal and is also considered to be an interim limit, i.e. future NPDES permits may require WPCFs to achieve even lower limits.

Section 8 - Evaluation of Sludge Disposal Facilities

Wright-Pierce originally completed a sludge disposal study for the Torrington WPCF in 2002. The objectives of the study were to evaluate the anticipated future cost of liquid versus dewatered sludge disposal, options for dewatering of the thickened sludge, and other potential improvements to the existing solids handling facilities. The evaluation of the sludge handling process has been updated with current costs and quantities, in conjunction with the Basis of Design, and summarized in Section 8 as part of the Facilities Study.

Section 9 - Evaluation of Ancillary Items

In addition to the evaluations of the liquid and solids handling unit processes and equipment, evaluations of ancillary components related to the operation of this facility were completed as part of the facilities study. This section includes discussions on: odor control, staffing, building systems and treatment plant / pump station security issues.

Section 10 - Energy Audit

An Energy Audit was performed for the Torrington WPCF and the seven (7) largest pump stations located in the collection system. Section 10 summarizes the evaluation work and objectives of the audit, as well as any resulting recommendations. The full Audit Report is appended to this Facility Plan.

Section 11 - Recommended Plan

Based on the evaluations discussed in the Sections above, a recommended plan for upgrading and expanding the Torrington WPCF was developed and is presented in Section 11.

Section 12 - Environmental Impact Assessment

As part of the facilities planning process, direct impacts of the recommended plan to air and water quality, floodplains, coastal zones, wetlands, farmlands, aquifer protection zones, historical and archaeological areas, and endangered species must be assessed. This assessment is presented in Section 12.

A variety of efforts have been performed to develop the components of the plan listed above. An evaluation of the plant was originally conducted with regard to all disciplines (i.e. structural, process, mechanical and instrumentation engineers and architects). Components of the previous study were re-evaluated and summarized in this plan. This was accomplished through on-site observations and discussions with plant staff. The interviews aided in evaluating both the current conditions as well as the anticipated future needs of the facility. The plant personnel were key participants in the evaluation and they were instrumental in providing insight into current operations and assessment of possible alternatives to improve operations.

SECTION 2

SECTION 2

DISCHARGE STANDARDS AND REGULATORY REQUIREMENTS

2.1 CURRENT EFFLUENT DISCHARGE LIMITATIONS

2.1.1 NPDES Discharge Permit

The City of Torrington Water Pollution Control Facility (WPCF) discharges to the Naugatuck River. The effluent limitations, monitoring requirements, and other conditions of the facility operation are established in the National Pollutant Discharge Elimination System (NPDES) permit, which is administered by the State of Connecticut Department of Energy and Environmental Protection (DEEP). The current permit (Permit ID CT0100579) was issued on August 23, 2006 and expired on August 11, 2011, approximately five years from the date of issuance. Discharge limitations in the current NPDES permit are provided to maintain the present and future water quality classification of the Naugatuck River. A copy of the current NPDES permit is included in Appendix A. The Torrington WPCF effluent standards are summarized in **Table 2-1**.

TABLE 2-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
NPDES EFFLUENT DISCHARGE LIMITATIONS

Parameter	Limitation
Flow[1]	7 mgd Average Daily
BOD_5	30 mg/l Average Monthly[2] 50 mg/l Maximum Daily
TSS	30 mg/l Average Monthly[2] 50 mg/l Maximum Daily
pH	6 - 9 S.U.
Fecal Coliform	< 200/100 ml 30-day geometric mean <400/100 ml 7-day geometric mean
Escherichia Coli	< 126/100 ml 30-day geometric mean <410/100 ml Maximum Daily
Chlorine Residual	0.050 mg/l Maximum Daily 0.100 mg/l Instantaneous
Dissolved Oxygen	> 5.0 mg/l Instantaneous
Copper	0.632 kg/d Average Monthly 1.269 kg/d Maximum Daily

Notes:
1. Minimum, maximum, and total flow for each day of discharge and the average daily flow for each sampling month shall be recorded and reported.
2. Minimum average monthly percentage removal is 85%.

The Naugatuck River is classified as "water quality limited" from the City of Torrington to the Housatonic River. Therefore, the WPCF is subject to effluent ammonia nitrogen limits. These limits were established by the DEEP for the protection of Class B ambient water quality in the Naugatuck River. The ammonia nitrogen limits are seasonal and vary monthly. The average monthly limits for ammonia nitrogen are listed in **Table 2-2**.

TABLE 2-2
CITY OF TORRINGTON WPCF FACILITIES PLAN
AMMONIA NITROGEN DISCHARGE LIMITATIONS

Month	Limitation
January	N/A
February	N/A
March	N/A
April	12.7 mg/l
May	6.8 mg/l
June	3.7 mg/l
July	1.7 mg/l
August	1.7 mg/l
September	1.7 mg/l
October	3.1 mg/l
November	N/A
December	N/A

2.1.2 Nitrogen Discharge Limitations

To reduce the occurrence of hypoxia (low dissolved oxygen conditions) in Long Island Sound, Connecticut and New York have established a Total Maximum Daily Load (TMDL) for nitrogen. The TMDL quantifies the maximum amount of nitrogen that can be discharged to Long Island Sound to meet water quality goals within the Sound.

Each Water Pollution Control Facility in Connecticut has been assigned a Waste Load Allocation (WLA) as part of the *General Permit for Nitrogen Discharges* (Nitrogen General Permit). The Nitrogen General Permit specifies how much total nitrogen each facility is permitted to discharge. The WLA is an annual mass loading of total nitrogen expressed in pounds per day. To achieve the goals of the TMDL, approximately a 64% reduction in the total nitrogen discharged from Publicly Owned Treatment Works (POTWs) is necessary. The

TMDL for nitrogen entering Long Island Sound must be achieved by 2014. Discharge limits have been included for each facility in the Nitrogen General Permit. These limits are reduced annually until the final limit in 2014, which was developed based on each facility's proportionate share of the TMDL nitrogen loading based on their 1997 to 1999 average daily flow rate.

As part of the Nitrogen General Permit development, a baseline for nitrogen loading of 680 lbs/day was established for the Torrington WPCF. The baseline was determined from an average of the effluent flows from 1997 through 1999 and an effluent total nitrogen concentration of 15.4 mg/l. Based on a nitrogen reduction of 64% of the baseline, the fully implemented WLA for the Torrington WPCF is 248 lbs/day. The WLA implementation schedule and limits for the Torrington WPCF, as included in the Nitrogen General Permit are presented in **Table 2-3**. A copy of the Nitrogen General Permit is included in Appendix A.

TABLE 2-3
CITY OF TORRINGTON WPCF FACILITIES PALN
DISCHARGE LIMITS FOR TOTAL NITROGEN

Year	2009	2010	2011	2012	2013	2014
Total Nitrogen (lbs/day)	292	283	273	260	254	248

Facilities covered by the Nitrogen General Permit are considered in compliance if:

a) the facility's annual mass loading of total nitrogen is less than or equal to the discharge limit set forth in the permit; or

b) the facility has secured equivalent nitrogen credits equal to the amount the facility exceeded the permitted annual discharge limit.

As a means of obtaining the nitrogen reduction goals, the DEEP has initiated a nitrogen credit exchange program. Facilities that discharge less total nitrogen than the limit established in the Nitrogen General Permit are considered in compliance and are credited for the amount of nitrogen removed beyond the limit. The DEEP is required to purchase all excess equivalent nitrogen credits generated by facilities in compliance. The DEEP in turn sells the necessary number of equivalent nitrogen credits to facilities not otherwise in compliance.

The equivalent nitrogen credits generated by a POTW are determined by applying an equivalency factor to the actual differential between the facility's annual mass loading of total nitrogen and the discharge limit. The equivalency factor takes into account the attenuation of nitrogen within the receiving waters before it reaches Long Island Sound. The Torrington WPCF has an equivalency factor of 0.60. Therefore, for every pound of nitrogen below or above the discharge limit, 0.60 pounds of equivalent nitrogen credits would be bought or sold.

The price of an equivalent nitrogen credit is established each year by the DEEP. For the year 2010, the price was set by the Nitrogen Credit Advisory Board at $4.59 per equivalent nitrogen credit. The DEEP has estimated that nitrogen credits may increase to approximately $8.00 per equivalent nitrogen credit by 2014.

2.1.3 Phosphorus Discharge Limitations

In anticipation of the United States Environmental Protection Agency (EPA) and the State of Connecticut DEEP mandating the reduction of phosphorus in the state's waters, the City of Torrington and their Water Pollution Control Authority (WPCA) has proactively elected to evaluate and plan for the need to remove phosphorus at the Torrington WPCF.

The state's phosphorus reduction program was initially published by the DEEP in June 2009 in draft form, as outlined in the document *Nutrient Reduction Strategy for Inland Fresh Waters: Phosphorus.* The program was designed to limit phosphorus in all wastewater discharges where the potential exists for the particular discharge to contribute to eutrophication of the receiving stream. The DEEP had indicated that the Torrington facility would fall into the Best Management Practice (BMP) category of "Medium" or "Moderate" and as such, would receive a near-term average phosphorus limit based on an effluent concentration of 0.7 mg/l. This limit was developed based on the WPCF flows, loads and various other contributing data (i.e. nutrients from land area associated with agriculture, urban, forest, etc.) associated with the 2001 to 2007 time period.

The 0.7 mg/l limit was applied to the average daily flow from the study period (5.18 MGD), which yielded a seasonal load allocation of 30.27 pounds per day. At the Torrington WPCF's current and future average daily design flows of 5.50 MGD and 6.31 MGD, this equates to a seasonal concentration average of 0.66 mg/l and 0.57 mg/l, respectively. According to the original Fact Sheet Issued by the DEEP as part of the *Nutrient Reduction Strategy for Inland Fresh Waters: Phosphorus* document, the Torrington WPCF average phosphorus concentration (2001-2007) was 1.68 mg/l, with a current average phosphorus loading (2001-2007) of 64.73 lbs/day, well above the proposed load allocation.

The EPA did not agree with DEEP's initial evaluation of phosphorus removal requirements and directed the DEEP to develop phosphorus removal limits based on a more specific water quality assessment. As a result, the DEEP revised the strategy by using best available science to identify phosphorus enrichment levels in waste receiving streams that would adequately support aquatic life uses. The methodology focuses on significant changes in stream algae as the key aquatic life response to excess phosphorus loading. The EPA approved the methods used to develop this strategy in a letter dated October 26, 2010, as an interim strategy to establish water quality based phosphorus limits in municipal WPCF's NPDES permits until numeric nutrient criteria can be established in Connecticut's Water Quality Standards (WQS). In March 2011 the DEEP published the revised interim phosphorus removal limits for many WPCFs throughout Connecticut. The March 2011 document, see Appendix A, assigned a proposed permit phosphorus load of 17.29 lbs/day to the Torrington WPCF; this is the anticipated limit that will be included with the next NPDES Permit Renewal. **Table 2-4** below is a summary of the anticipated phosphorus concentration discharge limit at current and future flows.

TABLE 2-4
CITY OF TORRINGTON WPCF FACILITIES PLAN
DISCHARGE LIMITS FOR TOTAL PHOSPHORUS

	Current Average	Design average
Flow (MGD)	5.5	6.31
Total Phosphorus (mg/l)	0.37	0.32

As part of this facilities planning study, biological, chemical and tertiary treatment systems were evaluated in order to determine their individual and/or combined applicability to achieving the anticipated phosphorus limits for the Torrington WPCF. This study assessed each alternative system with regard to various factors, including treatment efficiency/performance, the effect each process would have on side-stream processes, capital and operational costs, and any hydraulic implications.

The ultimate objective of this evaluation is to determine a recommended plan that will allow the Torrington WPCF to meet the proposed compliance schedule for phosphorus removal, while considering alternatives that include additional enhanced nitrogen removal capabilities.

2.2 COMPLIANCE WITH EXISTING DISCHARGE LIMITATIONS

The Torrington WPCF generally operates in compliance with the current discharge permit limits. There have been relatively few permit violations over the last five years and none have resulted in the need for any regulatory action. Permit violations that have occurred are presented in **Table 2-5** below.

TABLE 2-5
CITY OF TORRINGTON WPCF FACILTIES PLAN
PERMIT VIOLATIONS

Year	Days/Months of Violation	Violation
2007	June	Ammonia as Nitrogen High
	April	Average Monthly Flow >7 MGD
	4/24, 4/25, 7/10	Dissolved Oxygen <5 mg/L
	4/16, 7/31, 8/18, 8/20-21, 8/25, 8/27-28, 9/3-4, 9/8-12, 9/16-18, 9/20-28, 9/30, 10/1-2, 10/14-16, 10/22, 11/5, 11/16, 11/18, 11/23, 11/25	pH < 6.0
2008	Feb, Mar, Dec	Average Monthly Flow >7 MGD
	May, 5/1	Chlorine Residual High
	3/5, 4/16-17	Dissolved Oxygen <5 mg/L
2009	April, 4/27, 5/18, 7/31	Chlorine Residual High
	4/22-23	Dissolved Oxygen <5 mg/L
2010	June	Ammonia as Nitrogen High
	March	Average Monthly Flow >7 MGD
	9/14	Chlorine Residual High
	April	Copper Exceedance
	4/22	Dissolved Oxygen <5 mg/L
2011	Mar, April, May, Sept, Oct	Average Monthly Flow >7 MGD

Note: Months indicate monthly average value violation; dates indicate daily average violation.

SECTION 3

SECTION 3

WASTEWATER FLOWS AND LOADS

3.1 INTRODUCTION

The Torrington Water Pollution Control Facility (WPCF) receives flows primarily from the City of Torrington. Additional flow is received from portions of the Towns of Litchfield and Harwinton. A significant portion of the influent organic loading, although a relatively small portion of the influent flow, is from trucked-in wastes such as septage and grease tank pumpings.

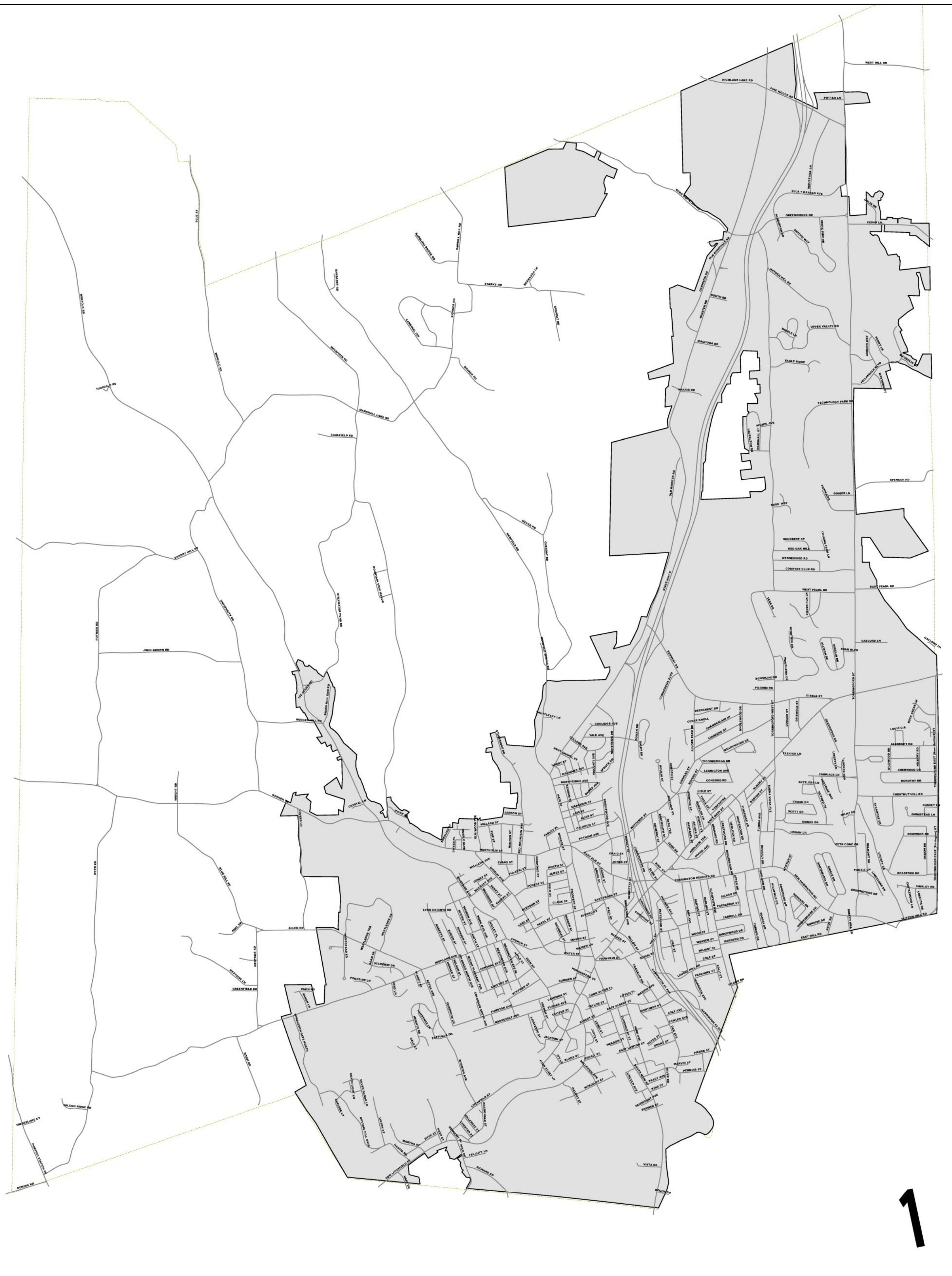
The existing historical influent flow and loading conditions from all waste streams was evaluated for average, minimum, maximum and peak influent values. These values have been used as the basis for determining the future design flows and loads.

Future design flows were projected based on the anticipated total population for the 20-year planning period (2015-2035). The future population was estimated from Census data and then coordinated with the City's Plan of Development and Sewer Service Area. The sewer service area for the City of Torrington is presented in **Figure 3-1**. This map represents the area of Torrington that currently is sewered or planned to be sewered

The existing influent loading conditions for the WPCF were utilized as the basis for projecting the future loading conditions associated with the design flows. The evaluation of the Torrington WPCF and all the unit processes was based on the Design Year Flow. This section is supplemented by a detailed flow and load technical memorandum which is included in Appendix B.

3.2 SEWER SERVICE AREA MAP

The City of Torrington's sewer service area map was prepared as part of the 2007 Torrington Facility Plan Draft. The sewer service area was developed and compared to the 2005 to 2010 State of Connecticut Office of Policy and Management (OPM) Conversation and Development



Sanitary Sewer Service Area

City of Torrington WPCA

Adopted: March, 7 2005
Amended: October 27, 2008
Effective: October 27, 2008

SCALE 1:12,000

0 0.5 1 Miles

-- Coord. System and Datum: CT SPCS NAD83 US Survey Ft
-- All Data City of Torrington 2008

DATE 10/27/08

Map by the City of Torrington Engineering Department

City of Torrington WPCF Facilities Plan

Sanitary Sewer Service Area Official Map

PROJ NO: 12411A	DATE: 03/26/2012	FIGURE: 3 - 1

WRIGHT-PIERCE Engineering a Better Environment

Policies Plan (C&D Plan) for consistency. The City adopted their Sewer Service Area in March 2005 and it was amended and made effective in October 2008. **Figure 3-2** shows the existing water and sewer service areas as provided in the City of Torrington Plan of Conservation and Development; this map was adopted by the City and became effective January 2010. The information from this figure was also provided to the State of Connecticut OPM for use in the Initial Draft of the 2013-2018 Conservation and Development Policies Plan for Connecticut. Presuming no changes are made to the draft 2013-2018 C&D Plan, the City of Torrington's Sewer Service Area map will be consistent with the C&D Plan.

Areas not designated as being sewered, or designated for future sewers, are considered Sewer Avoidance Areas. The City of Torrington has designated all areas outside of the Sewer Service Area as Decentralized Wastewater Management Areas. With the exception of when sewers would be the best solution to a water pollution problem caused by the failure of multiple subsurface disposal systems, the WCPA will not permit an extension of sewer infrastructure to serve individual properties outside of the Sewer Service Area.

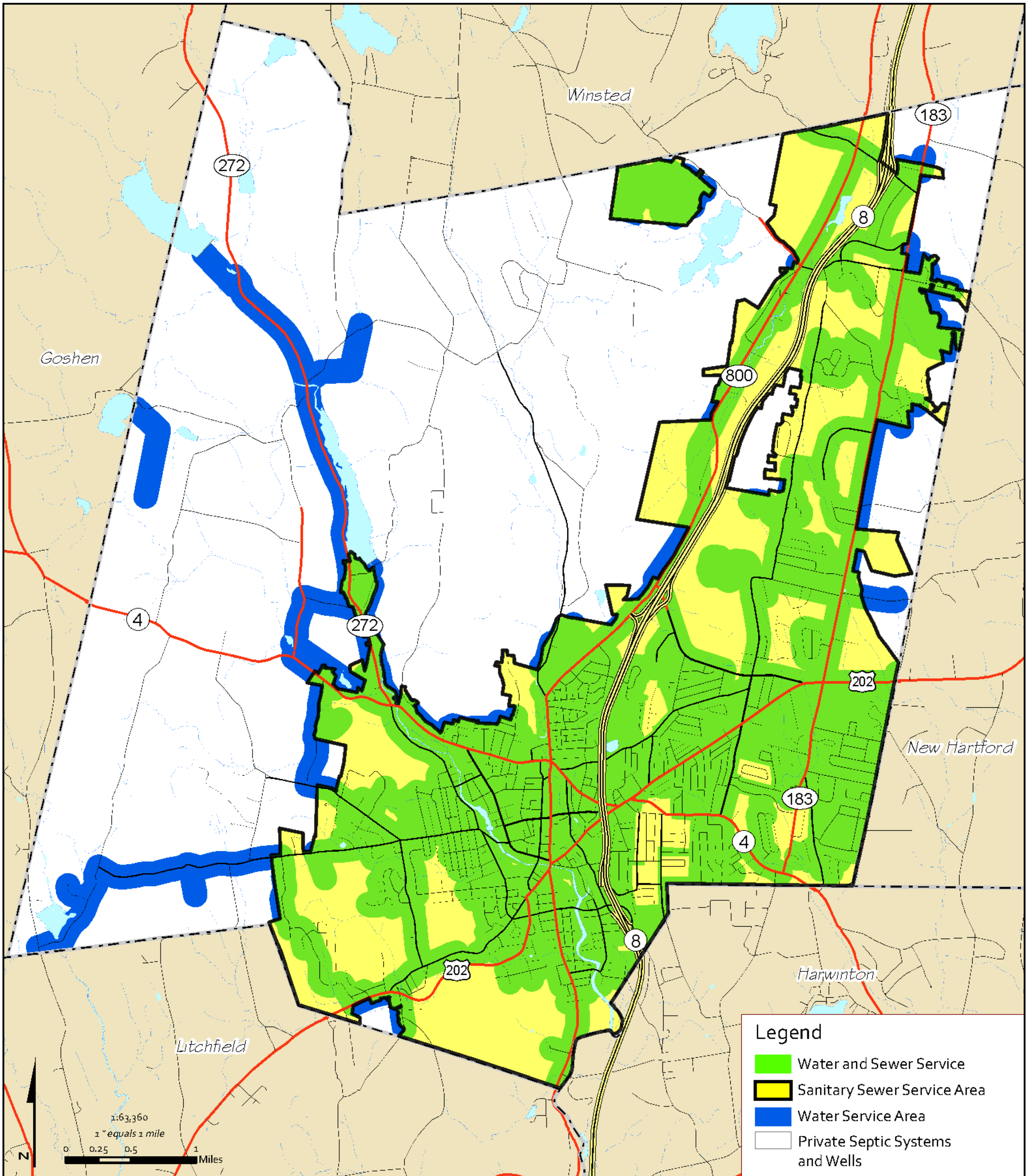
3.3 CURRENT FLOWS AND LOADS

Current influent wastewater flows and loads have been established based on facility operating data for the 58-month period from January 2007 through October 2011 (Analysis Period). The specific waste streams that make up the Torrington WPCF influent include the following:

- Sanitary flows through the Torrington wastewater collection system.
- Trucked-in septage.
- Decant from the grease receiving facilities.
- Internal recycle streams at the facility including supernatant from the gravity thickener and filtrate from the gravity belt thickener.

Note that the grease flows were analyzed for the period from 2007 to 2011 based on the availability of data.

Flow-proportioned composite samples were collected for influent BOD_5 and TSS analyses using an automatic sampler located in the screenings building. The collection of samples at this



City of Torrington WPCF Facilities Plan Update

Utilities Map

PROJ NO: 12411A	DATE: 03/26/2012	FIGURE:
WRIGHT-PIERCE Engineering a Better Environment		3 - 2

location included all raw sewage from the collection system but did not include septage, grease decant, gravity belt thickener filtrate, or gravity thickener supernatant.

The current influent wastewater characteristics were developed by evaluating the historic operating data over the analysis period for annual average, maximum month and maximum day flows and loads (BOD_5, TSS and TKN) and for peak-hour flows. Data for influent septage and grease were added to these values to develop the current wastewater characteristics. A summary of the existing influent characteristics is presented in **Table 3-1**.

TABLE 3-1
CITY OF TORRINGTON WPCF FACILITY PLAN
SUMMARY OF CURRENT FLOWS AND LOADS
INFLUENT DATA - JANUARY 2007 THROUGH OCTOBER 2011

	Min Day[1]	Annual Average	Max Month	Max Day[2]	Hydraulic Peak
EXISTING - Raw Influent					
Flow, mgd	2.84	5.50	11.60	21.94	22.50
Flow, mgd (98th percentile)				12.70	15.50
BOD5, lb/d		5,377	8,228	9,802	
TSS, lb/d		5,058	8,357	10,034	
TKN, lb/d		991	1,189	1,604	
EXISTING - Septage					
Flow, mgd		0.006	0.014	0.032	
BOD_5, lb/d		145	305	709	
TSS, lb/d		327	690	1,603	
TKN, lb/d		16	34	79	
EXISTING - Grease Decant					
Flow, mgd		0.003	0.003	0.007	
BOD_5, lb/d		66	79	160	
TSS, lb/d		43	52	105	
TKN, lb/d		2	2	5	
EXISTING - Raw Influent (Including Septage and Grease)					
Flow, mgd	2.84	5.51	11.62	21.98	22.50
Flow, mgd (98th percentile)				12.74	15.50
BOD5, lb/d		5,588	8,613	10,671	
TSS, lb/d		5,429	9,099	11,741	
TKN, lb/d		1,009	1,226	1,688	

Notes:
1. Minimum Day based on 3rd percentile value
2. Existing Maximum Day BOD_5 and TSS Loadings based on 98th percentile values

In addition to the analyses typically performed by the WPCF, supplemental sampling and analyses were performed as part of a wastewater characterization program. A copy of the wastewater characterization sampling plan and the results of the sample analyses are provided in Appendix C.

3.4 FUTURE FLOW AND LOAD PROJECTIONS

To establish the basis for evaluation of the Torrington WPCF, projections of future wastewater flow and loads have been made for the 20-year planning period (2015-2035). The projected flows and loads were determined based on an evaluation of current flows and loading parameters and adding allowances for anticipated residential, commercial, institutional and industrial growth as well as increases in trucked-in septage and grease loadings.

This section of the Facilities Study Report summarizes the results of the flow and load projections evaluation. A technical memorandum, presented in Appendix B, provides the details of the evaluation.

3.4.1 Projected Population Growth within the City of Torrington

Based on 2010 U.S. Census data, the average population associated with the existing flow data utilized for the evaluation period was approximately 36,383. It has been estimated, based on a review of U.S. Census data, pertinent sewer use/billing reports and discussions with the City of Torrington, that approximately 85% of the total population is located within the sewered service area. The remaining 15% of the City's population is served by on-site septic systems. Therefore, the average sewered population during the period of time which flows were evaluated is estimated to be approximately 30,925.

Population data based on the year 2010 Census data, along with projected population data for the year 2030 from the Connecticut State Data Center and University of Connecticut, were used to determine the current and future population within the Torrington Sewer Service area. As stated previously, approximately 85% of the total population is currently located within the sewered service area. It is assumed this value will remain unchanged in the future and 85% of the total projected population for the City of Torrington will be located within the sewered service area. Utilizing the estimated 2030 sewer population projections, extrapolating to the year 2035, it was estimated that the future sewered population of the City of Torrington would be approximately 40,307, an increase of approximately 9,382 people over the 2010 Census data. No allowances are included for unplanned connections of additional unsewered areas.

Growth and/or flow projections for the sewered areas outside of the City of Torrington that flow to the Torrington WPCF are as outlined in the respective sections below.

3.4.2 Inter-Municipal Agreement with Litchfield

An allowance for 0.112 mgd to the Town of Litchfield was assumed for allocated flows up to the limit in the current agreement. Under the existing inter-municipal agreement, the Town of Litchfield can discharge an average daily flow of up to 150,000 gpd to the Torrington WPCF. The current influent flow attributed to Litchfield is approximately 38,000 gpd. This represents 25 percent of their allocation.

At this time, it is uncertain as to whether Litchfield would actually need or desire to maintain this additional flow allocation which represents a sewered population increase of approximately 2,543 people, based on a per capita flow rate of 85 gpcd. Assuming that Litchfield would ultimately use their full allocation, and further assuming that there will not be an increase in their allocation over the planning period, the flow projections for the Torrington WPCF includes an allowance of 112,000 gpd to account for the remaining allocated flow from the Town of Litchfield.

3.4.3 Inter-Municipal Agreement with Harwinton

There is an allowance for 0.021 mgd to the Town of Harwinton for allocated flows up to the limit in the current inter-municipal agreement. Under the existing inter-municipal agreement, the Town of Harwinton can discharge an average daily flow of up to 77,000 gpd to the Torrington WPCF. The current influent flow attributed to Harwinton is approximately 56,000 gpd. This represents approximately 73 percent of their current allocation.

3.4.4 Commercial & Industrial Growth

Based on discussions with the City of Torrington, there is no significant commercial or industrial growth anticipated in the sewered service area. The base wastewater flow generation factor of 85 gpcd includes an allowance of 10 gpcd for commercial, institutional and industrial growth associated with the increased residential population. No specific additional increases in commercial or industrial flow are included in the flows projections.

3.4.5 Septage

Based on anticipated population growth in the unsewered areas within the City of Torrington and the surrounding towns that discharge septage to the Torrington WPCF, by the year 2035 an increase in septage loadings of approximately 12.4%, or 790 gpd, is anticipated based on receiving septage five days per week.

3.4.6 Grease

Additional grease trap pumping wastewater is anticipated to be received at the Torrington WPCF due to the recent issuance of the *General Permit for the Discharge of Wastewater Associated with Food Preparation Establishments*. As described in Appendix B, it is estimated that the decant volume associated with handling increased grease trap pumping could be approximately 2,300 gpd based on a five day per week operating schedule.

3.4.7 Summary of Projected Flows

Based on the growth assumptions, a design base wastewater flow generation rate of 85 gpcd for the increase in the population, plus the allowance for increases in septage and grease loadings, the anticipated increase in average daily flow is projected to be approximately 0.80 mgd by design year 2035. The design base wastewater flow generation rate of 85 gpcd is based on industry standards of 75 gpcd for residential/domestic flow, and an allowance of 10 gpcd for commercial and industrial flow.

Future maximum month and maximum day flows and loads were developed based on the projected future average daily flow multiplied by a peaking factor. The ratio of current maximum month to annual average and maximum day to annual average were used as the peaking factors as described in the technical memorandum in Appendix B. A summary of the projected increases in residential, septage and grease decant flows and loads are presented in **Table 3-2**.

TABLE 3-2
CITY OF TORRINGTON WPCF FACILITY PLAN
FUTURE INFLUENT FLOW AND LOAD PROJECTIONS BASIS OF DESIGN (YEAR 2035)

	Min Day[1]	Annual Average	Max Month	Max Day[2]	Hydraulic Peak
EXISTING - Raw Influent (Including Septage and Grease)					
Flow, mgd	2.84	5.51	11.62	21.98	22.50
Flow, mgd (98th percentile)				12.74	15.50
BOD5, lb/d		5,588	8,613	10,671	
TSS, lb/d		5,429	9,099	11,741	
TKN, lb/d		1,009	1,226	1,688	
PROJECTED INCREASE IN FUTURE RESIDENTIAL					
Flow, mgd	0.41	0.80	1.68	3.18	3.26
BOD_5, lb/d		1,595	1,876	3,002	
TSS, lb/d		1,595	2,064	3,190	
TKN, lb/d		319	375	600	
ADDITIONAL SEPTAGE					
Flow, mgd		0.001	0.002	0.004	
BOD_5, lb/d		23	49	113	
TSS, lb/d		41	86	200	
TKN, lb/d		2	4	10	
ADDITIONAL GREASE RECEIVING					
Flow, mgd		0.002	0.003	0.006	
BOD_5, lb/d		57	69	138	
TSS, lb/d		36	44	88	
TKN, lb/d		2	2	10	
TOTALS					
Flow, mgd	3.25	6.31	13.31	25.17	25.76
Flow, mgd (98th percentile)				15.93	18.76
BOD_5, lb/d		7,263	10,607	13,925	
TSS, lb/d		7,101	11,293	15,219	
TKN, lb/d		1,332	1,607	2,308	

Notes:
1. Minimum Day based on 3rd percentile value
2. Existing Maximum Day BOD_5 and TSS Loadings based on 98th percentile values

3.5 BASIS OF DESIGN

A summary of the current wastewater flows and loads along with the future projected wastewater flows and loads to be used as the basis of evaluation and Basis of Design for the Torrington WPCF is presented in **Table 3-3**.

TABLE 3-3
CITY OF TORRINGTON WPCF FACILITY PLAN
CURRENT AND PROJECTED INFLUENT WASTEWATER FLOWS AND LOADS

	Current Period				Design Year			
	Flow (mgd)	BOD (lb/day)	TSS (lb/day)	TKN (lb/day)	Flow (mgd)	BOD (lb/day)	TSS (lb/day)	TKN (lb/day)
Annual Average	5.51	5,588	5,429	1,009	6.31	7,263	7,101	1,332
Max. loading 30-Day MA[1]	8.27	8,613	8,930	1,981	9.95	10,607	11,120	2,360
Maximum Day								
100th percentile	21.98	-	-	-	25.17	-	-	-
98th percentile	12.74	10,671	11,741	1,688	15.93	13,925	15,219	2,300
Maximum Hour								
100th percentile	22.50	-	-	-	25.76	-	-	-
98th percentile	15.50	-	-	-	18.76	-	-	-

1. Maximum loading 30-Day MA Flow, TSS and TKN is based on the flows and loads that occurred for the same time period as the max 30-Day MA BOD loading value.

3.6 INFILTRATION AND INFLOW

The City of Torrington WPCF has a permitted design flow rate of 7.0 mgd. While the WPCF receives only 5.51 mgd on an annual average basis, the maximum month, maximum day and peak hour flows presented in **Table 3-3** show that the Torrington WPCF receives significant wet weather flows. As described in Appendix B, it is estimated that the Torrington WPCF receives approximately 2.87 mgd of infiltration on a seasonal basis. When groundwater levels are high and rain-induced infiltration occurs, infiltration flows in excess of 4 mgd are seen at the WPCF. In addition, peak inflow rates of 16 to 18 mgd have been experienced recently at the WPCF.

These inflow rates have increased substantially in recent years as the peak inflow rates were only estimated to be 10 to 15 mgd in 2007.

The City of Torrington is responsible for a sanitary sewer collection system that includes approximately 230 miles of sewer lines and 14 pumping stations. As stated above, the collection system experiences significant increases in flows both seasonally due to high groundwater levels and during and immediately following rainfall events. The City of Torrington has completed an Infiltration and Inflow Analysis and Sewer System Evaluation Survey of the East Main Street and New Harwinton Road Pump Station Service Areas. The results of these evaluations are presented in the *Phase I Infiltration and Inflow Analysis Report* and the *Phase II Sewer System Evaluation Study Report* prepared by Cardinal Engineering in 2004.

The service areas studied under this project included approximately 40 miles of sewer lines or less than 20% of the overall collection system. The Executive Summary of the *Phase II Sewer System Evaluation Study Report* (Cardinal, 2004) identified approximately 1.48 mgd of infiltration that could be cost-effectively removed. In addition, approximately 1.5 million gallons per year of inflow, out of an estimated total of 19.3 million gallons per year of inflow, was identified as being able to be cost-effectively removed.

While the City continues to budget and plan for the recommended improvements included in the Cardinal study, it should be noted that this study evaluated 9 of 13 drainage basins. The City of Torrington includes budget in their Capital Improvement Plan (CIP) to evaluate the remaining 4 drainage basins for both public and private sources of infiltration and inflow. Due to the size of the collection system, this is anticipated to be conducted as a multi-year investigation with on-going rehabilitation work as problem areas are identified.

SECTION 4

SECTION 4

EXISTING FACILITIES AND OPERATION

4.1 INTRODUCTION

The Torrington Water Pollution Control Facility (WPCF) is an activated sludge wastewater treatment plant designed to treat an average daily flow (ADF) of 7.0 million gallons per day (mgd) and a peak hourly flow of 20.0 mgd. The WPCF discharges to the Naugatuck River. The WPCF was originally constructed as a primary treatment plant in 1935, and was upgraded to provide secondary treatment and sludge processing facilities in 1970. There have been additional modifications to the facility since the 1970 upgrade. The most recent treatment improvements were in 1994, when nitrification and improved disinfection facilities were provided.

All of the wastewater is conveyed from the WPCF via a 54-inch diameter trunk sewer commonly referred to as the Central Interceptor. The Central Interceptor sewer discharges to a siphon inlet structure (S-140) located on the west side of the Naugatuck River. Three inverted siphons (two 16-inch and a 24-inch) passing under the Naugatuck River transport the wastewater to a siphon outlet chamber located at the head of the WPCF site. The main portion of the Central Interceptor was close-circuit televised (CCTV) in 2012 to determine the condition of the infrastructure. All sections of the Central Interceptor were found to be in good condition.

Once at the WPCF site, the flow enters the headworks area for preliminary treatment. The following major process systems exist at the Torrington WPCF:

- preliminary treatment (including septage handling and grease receiving);
- primary treatment;
- secondary treatment
- disinfection;

- effluent discharge; and
- solids handling.

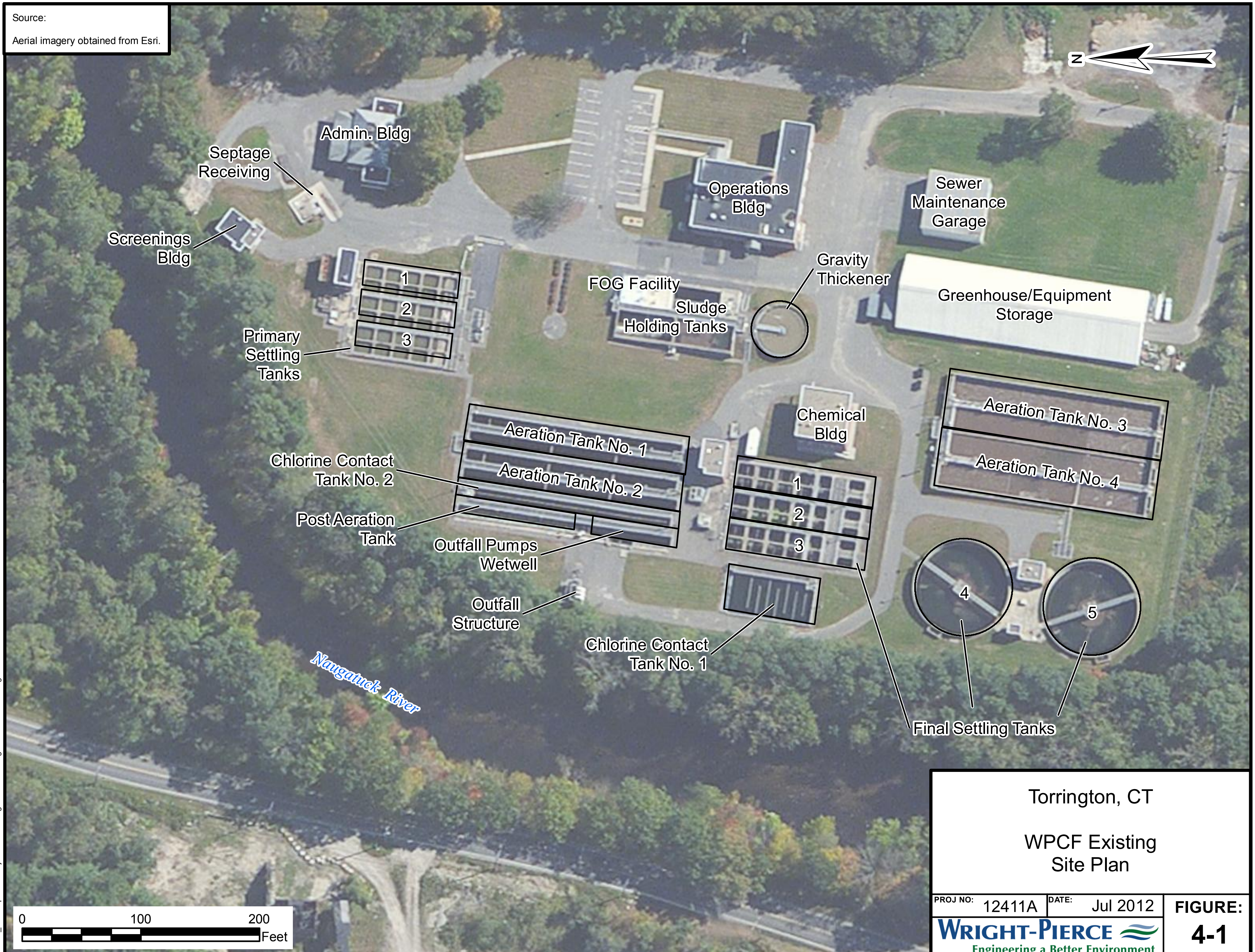
A site plan showing the existing buildings and tankage is presented in **Figure 4-1** and a description of each existing component is provided in the remaining portions of this section.

4.2 CURRENT OPERATING CONDITIONS

The current operating parameters for the various process units were established during the previous facilities planning process completed in 1990 and 1991. Wastewater and sludge Process Flow Diagrams for the treatment plant are illustrated in **Figures 4-2 and 4-3,** respectively. A summary of the current design criteria for each treatment process and/or component is presented in **Table 4-1**.

Although the Torrington WPCF typically receives an average daily flow less than the design capacity, flows increase to above the design average capacity during wet weather periods. Current BOD_5 and TSS loadings are also below the original unit design criteria. As is the case with the influent flows, periodic increases in the influent BOD_5 and TSS can impact the operation of the facility.

The plant is currently meeting the effluent permit limits for BOD_5 and TSS, as well as requirements for effluent disinfection and ammonia nitrogen removal. The current plant flows, loadings, and compliance to discharge requirements are discussed in detail in Section 2 of this report.



Torrington, CT

WPCF Existing Site Plan

PROJ NO: 12411A DATE: Jul 2012

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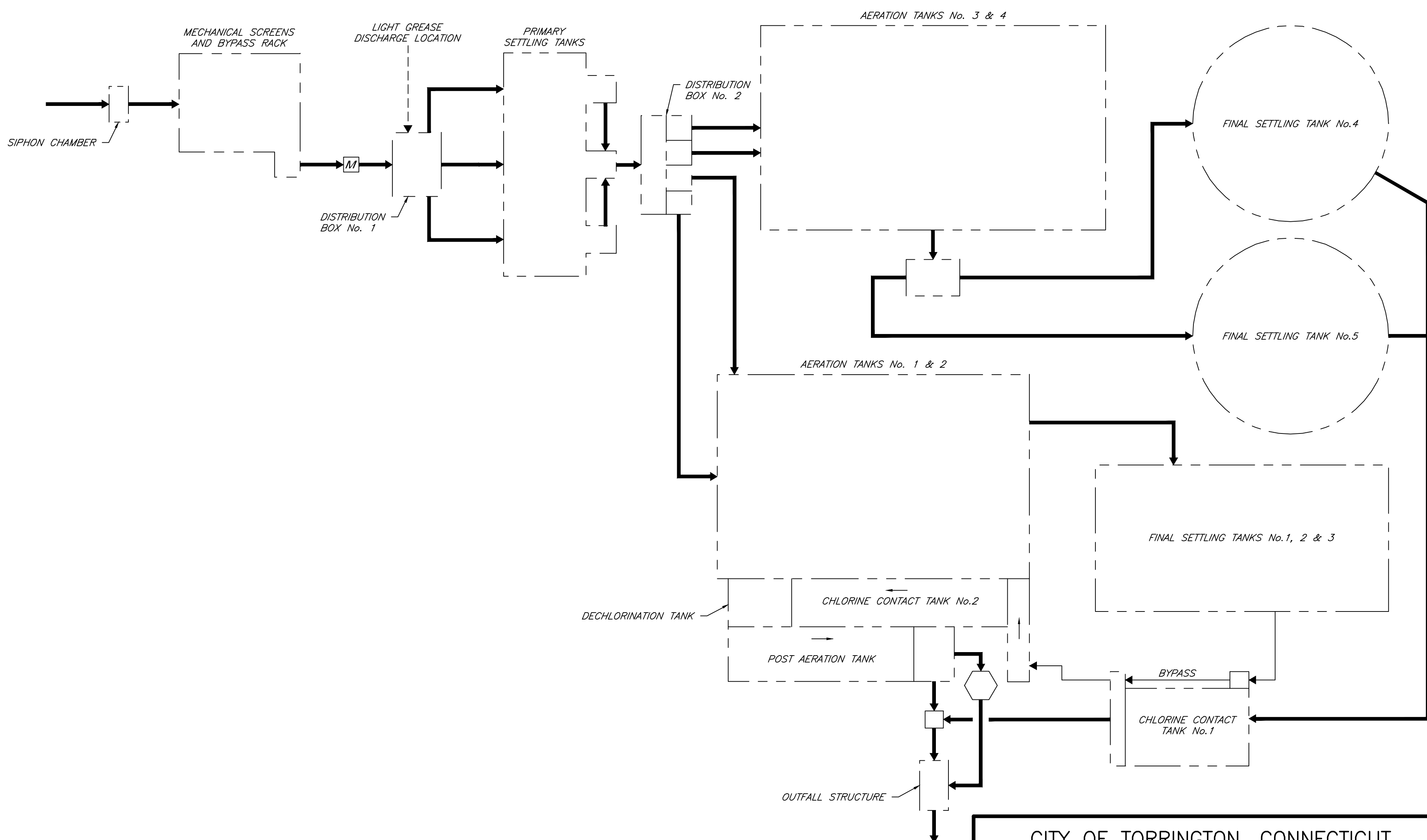
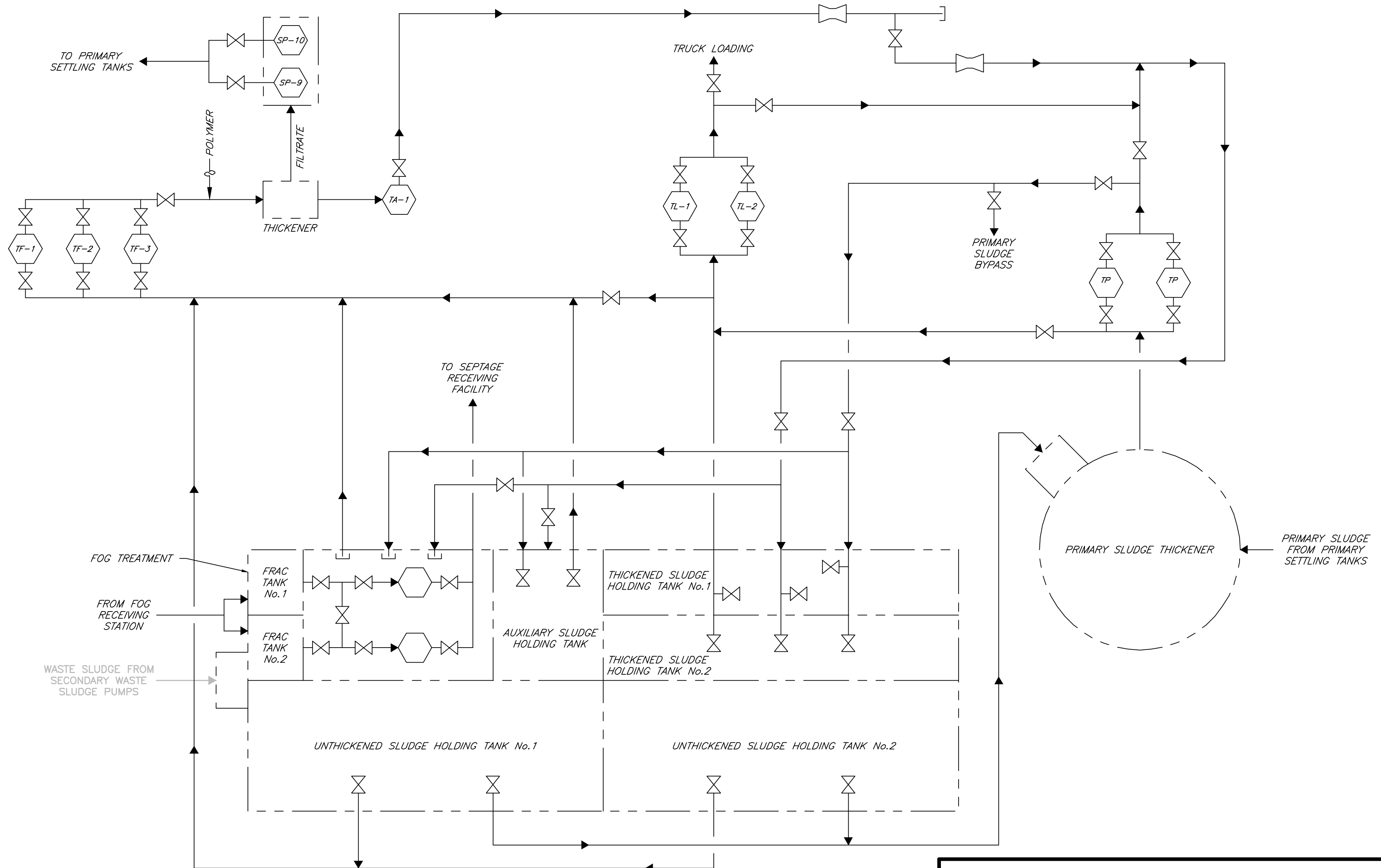
FIGURE: 4-1



CITY OF TORRINGTON, CONNECTICUT WATER POLLUTION CONTROL FACILITY UPGRADE WASTEWATER PROCESS FLOW DIAGRAM			
PROJ NO: 12411A	DATE: APR 2012		FIGURE:
WRIGHT-PIERCE Engineering a Better Environment			4-2



CITY OF TORRINGTON, CONNECTICUT

WATER POLLUTION CONTROL FACILITY UPGRADE

SLUDGE PROCESS FLOW DIAGRAM

PROJ NO: 12411A	DATE: APR 2012	FIGURE:
WRIGHT-PIERCE Engineering a Better Environment		4-3

TABLE 4-1
CITY OF TORRINGTON WPCF FACILITY PLAN
SUMMARY OF CURRENT DESIGN CRITERIA

Treatment Process	Criteria	
Mechanically – Cleaned Bar Screens		
Number of Units	2	
Channel width, ft.	3	
Bar spacing, inches	¾" clear opening	
Flow Measuring Device		
Type	Magmeter	
Diameter, inches	24	
Maximum Flow, mgd	25	
Primary Settling Tanks		
Number of Tanks	3	
Tank Dimensions, ft		
Length	96	
Width	24	
Side water depth	8	
Surface overflow rates, gal/day/sq ft		
Average flow (1)	1,010	
Peak Flow (2)	2,890	
Sludge Collection	Chain & Flight	
Primary Sludge Pumps (GP-1, GP-2, GP-3, GP-4)		
Number of pumps (including standby)	4	
Type	Recessed Impeller	
Total capacity with one pump out of service, gpm	1,500	
Grit Removal Equipment (Primary Sludge)		
Number of cyclone degritters	1	
Number of grit classifiers	1	
Aeration Tanks	Tanks 1 & 2	Tanks 3 & 4
Number of Tanks	2	2
Tank dimensions, ft		
Length	180	174.5
Width	30	45
Side water depth	13	13
Total hydraulic detention time, hours		
Average flow (1)	8.8	
Peak Flow (2)	3.1	
Total solids retention time days (3)	16	16
MLSS concentration, mg/l	3,500	3,500
F/M (3)	0.14	0.14
Organic loading, lb BOD/day/1,000 cu ft (3)	30	30

TABLE 4-1 (Continued)

Treatment Process	Criteria	
Aeration Equipment		
Number of blowers (including standby)	4	
Type of diffuser	Fine Bubble	
Total blower capacity (with 3 units in service), scfm	11,100	
Discharge pressure, psig	7	
Blower motor HP	200	
Sodium Bisulfite Feed System		
Chemical storage capacity, gal	4,000	
Number of tanks	2	
Number of pumps	2	
Pumping rate per pump, gph	8	
Final Settling Tanks	Tanks 1, 2, & 3[1]	Tanks 4 & 5
Number of Tanks	3	2
Type	Rectangular	Circular
Length, ft	120	--
Width, ft	25	--
Diameter, ft	--	80
Total Surface area, sq ft	9,000	10,000
Side water depth, ft	8	14
Total Surface overflow rates, gal/day/sq ft		
Average flow (1)	310	420
Peak Flow (2)	890	1,200
Sludge Collection	Chain & Flight	Rotating Rake
Return Sludge Pumps	RS-1, 2, 3 & 4	RS-5, 6 & 7
Number of pumps (including standby)	4	3
Type	Centrifugal	Centrifugal
Pump capacity, gpm	1,100	2,150
Percent of average plant design flow (1)	50 - 100	50 - 100
Motor HP	7.5	15
Waste Sludge Pumps	WS-1 & 2	WS-3 & 4
Number of pumps (including standby)	2	2
Type	Centrifugal	Centrifugal
Pump capacity, gpm	100	140
Motor HP	2	2
Chlorine Contact Tanks		
Volume of Chlorine Contact Tank #1, gal	168,000	
Length, ft	70	
Width, ft	35	
Side water depth, ft	9	

[1] Currently not in service

TABLE 4-1 (Continued)

Treatment Process	Criteria
Chlorine Contact Tanks (Continued)	
Volume of Chlorine Contact Tank # 2, gal	170,000
Length, ft	169
Width, ft	13.7
Side water depth, ft	9.8
Total detention time at peak flow (20 MGD), min	24
Chlorination Chemical	Liquid Sodium Hypochlorite
Dechlorination	Liquid Sodium Bisulfite
DeChlorination Facilities	
Volume of Dechlorination Tank	12,100
Length, ft	15
Width, ft	11
Side water depth, ft	9.8
Total detention time at peak flow, min	0.9
Dechlorination Chemical	Liquid Sodium Bisulfite
Post Aeration Facilities	
Air requirement	300 scfm
Type	Diffused Air
Effluent DO at peak flow, mg/l	5.0
Volume of post-aeration tank, gal	102,000
Length, ft	101
Width, ft	15
Side water depth, ft	9
Detention time at peak flow, min	7
Outfall Pumps (OP-1, OP-2, OP-3)	
Number	3
Type	Axial-Flow
Capacity per pump, gpm	11,800
Motor, HP	125
Outfall Pumps Wetwell	
Volume, gal	160,000
Length, ft	70.5
Width, ft	34
Depth, ft	8
Unthickened Sludge Holding Tanks	
Number of tanks	2
Mixing	Coarse Bubble Aeration
Volume per tank, gal	125,000
Length, ft	49.5
Width, ft	22.5
Depth, ft	15
Available Storage Capacity (Average), days	4

TABLE 4-1 (Continued)

Treatment Process	Criteria
Thickened Waste Sludge Holding Tank	
Number of tanks	2
Mixing	Mechanical Mixers
Volume per tank, gal	18,800 / 14,900
Length, ft	14
Width, ft	12 / 9.5
Depth, ft	15
Available Storage Capacity (Average), days	4
Auxiliary Sludge Holding Tank	
Number of tanks	1
Mixing	Coarse Bubble Aeration
Volume per tank, gal	106,000
Length, ft	42
Width, ft	22.5
Depth, ft	15
Sludge Mixing Blowers	
Number of blowers	2
Type	Positive Displacement
Capacity per blower, scfm	1,300
Blower Speed, rpm	2,050
Discharge pressure, psig	5.3
Primary Sludge Thickener	
Number	1
Dimension, ft	
Diameter	40
Sidewater depth	10
Solids loading, lb/day/sq ft	9.8
Thickened Primary Sludge Pumps (TP-1, TP-2)	
Number of pumps/Type	2/Double Disk, Positive Displacement
Capacity per pump, gpm	170
Motor HP	5
Gravity Belt Thickener (Secondary Sludge)	
Number	1
Solids loadings, lb dry solids/hour @ %solids feed	940 @ 0.5% - 2,250 @ 1.5%
Belt width, meters	1.5
Gravity Belt Thickener Feed Pumps (TF-1, TF-2, TF-3)	
Number of pumps/Type	3/Double Disk, Positive Displacement
Capacity per pump, gpm	170
Motor HP	5

TABLE 4-1 (Continued)

Treatment Process	Criteria
Thickened Waste Sludge Pumps (TA-1)	
Number of pumps/Type	1/Rotary Lobe, Positive Displacement
Capacity, gpm	70
Motor HP	5
Thickened Sludge Truck Loading Pumps (TLP-1, -2)	
Number of pumps/Type	2/Rotary Lobes
Capacity per pump, gpm	400
Motor HP	15
Septage Transfer Pump (ST-1)	
Number of pumps/Type	1/Double Disk, Positive Displacement
Capacity, gpm	170
Motor HP	5
Septage Receiving Station	
Number of tanks	1
Volume, gallons	15,000
Length, ft	20
Width, ft	20
Height, ft	11.75
Mixing	Mechanical
FOG Receiving Station	
Number of tanks	2
Volume per Tank, gallons	12,380
Length, ft	18'
Width, ft	12'
Height, ft	Sloped Floor
Mixing	Mechanical
Number of Mixers	2
HP	5
Level Sensors	2 Ultra Sonic
FOG Transfer Pumps	
Number of pumps	2
Type	Rotary Lobe Pumps
Capacity, gpm	400
Motor HP	25
Flow Meter	Doppler Type
TSS Sensor	Hach TSS Inline Analyzer

(1) Design average flow = 7.0 mgd
(2) Design peak hourly flow = 20.0 mgd
(3) Based on maximum monthly loading

4.3 PRELIMINARY TREATMENT

The preliminary treatment facilities include coarse mechanical screening, septage receiving, grease receiving, and flow measurement; each of these are discussed separately below.

4.3.1 Mechanical Screening

The influent wastewater is screened to remove gross suspended materials such as rags, sticks, leaves, rocks, fecal material and other debris. If not removed from the influent flow, these materials could interfere with or damage downstream processes and equipment.

Two self-cleaning, catenary-type mechanical bar screens, manufactured by Envirex, Inc., were installed as part of the 1970 plant upgrade. The mechanical bar screens are located in the Screenings Building, which is National Electric Code (NEC) rated as Class I, Division 1, Group C and D.

Mechanical Bar Screens

All electrical equipment in the building is rated as "explosion-proof". Electrical controls for the screening equipment are housed in the motor control center located in the Electrical Room of the Screenings Building.

Each bar screen is mounted in a three foot wide flow channel. A bypass channel with a manually cleaned bar rack, with 1-inch bar spacing, is located between the two primary flow channels. Inlet and outlet slide gates allow for isolation of each flow channel. Each bar screen is rated for a maximum flow of 10 mgd at a maximum water depth of 4.62 feet.

During normal operation, wastewater flows by gravity through both mechanical bar screens. Debris collected on the bar screens is mechanically removed and discharged into a small container. The screens alternate operation on 15-minute cycles and both screens are operational when the plant flows exceed 10 MGD. The screened material is collected and manually transported to a dumpster at a lower elevation within the Screenings Building. The screenings in the dumpster are then hauled, using a front-end loader, to a larger roll-off container located adjacent to the septage receiving station.

4.3.2 Septage Receiving

The Torrington WPCF serves as a regional septage receiving facility. The septage receiving station accepts trucked-in septage generated within unsewered areas of Torrington and surrounding communities for disposal at the plant.

Septage Receiving Facility

The septage receiving station, constructed as part of the 1994 plant modification, is located adjacent to the screenings building. The septage receiving facilities consist of an unloading pad, storage tank, mixer, and an influent channel containing quick disconnect couplers and a manually-cleaned bar screen. Although not located at the receiving station, another integral component of the system is the septage transfer pump which is located in the primary settling tank gallery.

The septage receiving station is designed for unloading one truck at a time. Two quick disconnect couplers, one 4-inch and one 6-inch, are provided to facilitate the unloading operation. Septage from the trucks passes through a manually cleaned coarse bar screen with 1-inch clear opening. The concrete storage tank has a nominal capacity of 15,000 gallons. Two manually-operated shear gate valves are located inside the storage tank on the septage pump suction pipe. The shear gates are located at different elevations within the tank. The shear gate valves are normally closed. The gate valve at the higher operating level is opened when pumping septage to the screenings building effluent channel. The gate valve at the lower elevation serves primarily as a tank drain and is opened to completely drain the tank when it is removed from service.

A vertical shaft mixer, manufactured by United Equipment Technologies, is located at the center of the septage holding tank. The mixer has a 50-inch diameter impeller and is driven by 7.5 hp reversing motor. The mixer can be operated in HAND or AUTO mode. In HAND mode, the mixer is operated by the Start/Stop pushbuttons located in the control station at the mixer. In AUTO mode, the mixer is operated by a 24-hour timer. The mixer is not intended for continuous operation. Generally, the mixer is operated to ensure the contents of the septage storage tank are completely mixed and any solids and grit are resuspended prior to the operation of the septage transfer pump.

The septage transfer pump conveys the septage from the storage tank, through 6-inch suction and discharge piping, to the effluent channel downstream of the mechanical bar screens. The unit is a double disk positive displacement pump manufactured by Penn Valley. It is equipped with a 5 hp motor, and has a design operating point of 120 gpm @ 40 ft TDH.

The septage receiving facilities were designed to accept septage during normal plant operating hours and discharge it to the WPCF during the night or early morning. This mode of operation would balance the septage loading and provide supplemental organic loading to the activated sludge system during periods when the influent flow is typically lower.

Interconnected piping and valves on the septage transfer pump discharge allows for alternative operations. These alternatives allow the septage to be pumped from the holding tank to:

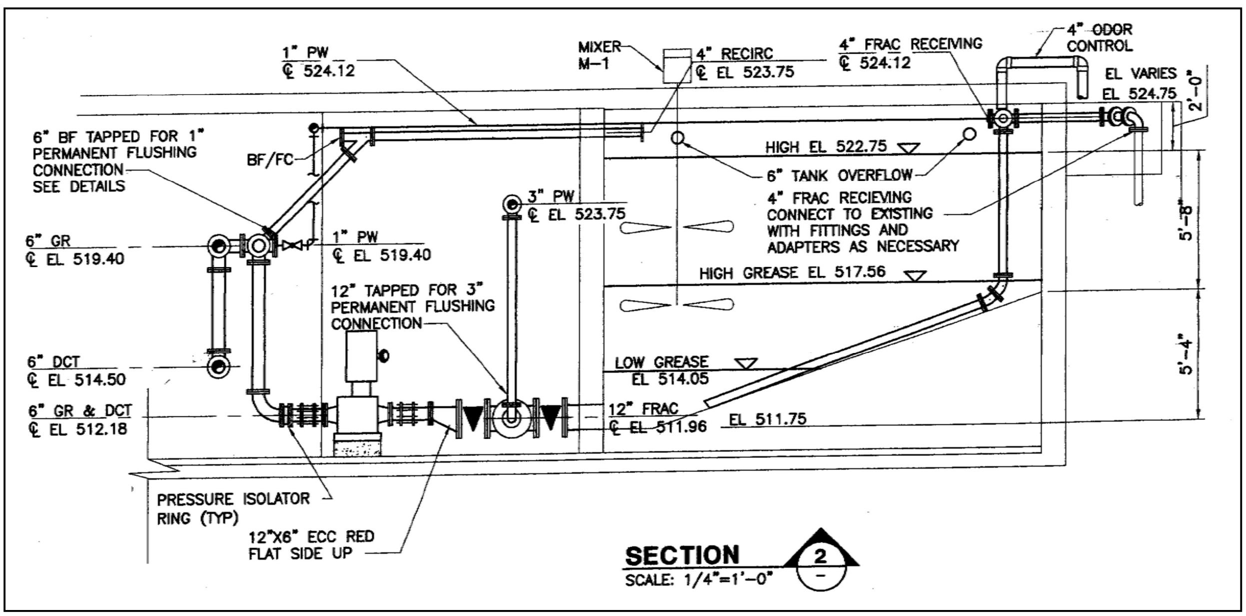
- the mechanical bar screen effluent channel,
- the thickened waste sludge holding tanks,
- the auxiliary sludge holding tanks.

Septage can also be off-loaded from the septage trucks into the 8-inch pipe located on the influent pipe downstream of the siphon outlet chamber.

4.3.3 Grease Receiving

The Torrington WPCF has served as a regional grease receiving facility since 1999. Grease loads are received at the plant on a regular basis from 18 communities served by the Torrington Area Health District (TAHD). The facility also receives varying amounts of grease deliveries from other communities throughout State. Following the promulgation of the *General Permit for Wastewater Discharges Associated with Food Preparation Establishments* by the CTDEEP, the grease loads are expected to increase steadily over time. Torrington evaluated and studied the potential for additional treatment of the grease, which culminated in the construction of the Torrington Regional FOG Receiving Facility in 2010.

The primary component of the grease receiving facilities are two completely enclosed concrete tanks that operate as fractionalization tanks (frac tanks), each with a working volume of nearly 12,400 gallons. Adjacent to these tanks, a pump room was constructed for the transfer pumps, piping and all controls related to the operation of the FOG system. Also included in the facility was a carbon style odor control canister to help mitigate odors on an interim basis until the full plant upgrade was constructed.



FOG Treatment Facility

Grease arrives at the plant by truck and is discharged to the scum pumping station adjacent to Primary Settling Tank No. 1. The scum pumping station is equipped with a submersible chopper pump. From the scum pumping station, grease can be discharged to the frac tanks or directly into the head of the primary settling tanks.

A plant operator samples the grease from each truck and makes a general determination on the thickness. Thicker grease is pumped to frac tank where the grease can thicken further, while lighter grease loads are discharged to the head of the primary settling tanks. Grease that is discharged to the primary settling tanks floats and is removed from the tanks using the scum collection system.

After the Grease is thickened, the decant water is pumped out of the frac tanks to the septage receiving tank, and is then metered into the front end of the treatment plant with incoming septage flows. Two rotary lobe pumps manufactured by Boerger are used to transfer the decant water. An inline Suspended Solids Analyzer is utilized to detect the transition between the clear water decant layer and thickened grease layer in the FOG material. Once the transition between these two layers is detected, the rotary lobe pumps are shut down to prevent transferring thickened grease to the septage receiving tank. Thickened grease is hauled off site to the Synagro Incinerator facility in Waterbury, Connecticut for processing.

4.3.4 Influent Flow Measurement

The influent magnetic flow meter is located downstream of the mechanical screens adjacent to the primary settling tank gallery. The meter measures the combined influent wastewater and septage flows. The installation of the flow meter is not ideal; a pipe enlargement and 90° bend combination is located immediately downstream of the meter and approximately three feet upstream of the meter. Generally, a flow meter requires a significant run of straight pipe upstream and downstream of the location to allow the flow profile to stabilize. Any turbulence or uneven flow velocity will affect the accuracy of the flow meter, causing the meter reading output to fluctuate to false highs and lows. As a general rule, a flow meter should be installed five to ten pipe diameters downstream, and two to five pipe diameters upstream of any bend, tee, junction, or change in pipe diameter.

4.4 PRIMARY TREATMENT

The primary treatment system is comprised of the following components:

- influent and effluent distribution;
- primary sedimentation;
- scum collection and disposal;
- sludge collection and pumping; and
- grit removal

4.4.1 Influent and Effluent Distribution and Primary Sedimentation

The combined influent wastewater and septage enters through the bottom of the influent distribution box (Distribution Box No. 1) via a 30-inch gravity pipe from the Screenings Building.

The influent distribution box is a concrete structure approximately 7'-0" x 7'-0" in area. Two weir openings are located on each of the side walls. The openings on the south, east, and west walls provide flow distribution to Primary Settling Tanks (PST) Nos. 1, 2, and 3 respectively. The weir openings on the north wall of the distribution box provide for primary settling tank bypass to Distribution Box No. 2, and a process drain between Distribution Box No. 1 and the Primary Sludge Gravity Thickener. Slide gates are provided to isolate and control flow to each tank.

Primary Settling Tanks

Three rectangular primary settling tanks provide primary sedimentation. The tanks were constructed as part of the 1970 plant upgrade. Each tank consists of two longitudinal bays and a cross collection channel. The tanks are 96 feet long, and each longitudinal bay is 12 feet wide. The design side water depth is 8 feet. The design surface overflow rates are 1,010 gpd/sq. ft at average flow (7.0 mgd) and 2,890 gpd/sq.ft at peak flow (20.0 mgd). Effluent from the primary settling tanks flows over horizontal v-notch weirs into the effluent troughs and discharges through the effluent channel to the aeration tank influent distribution box (Distribution Box No. 2). The aeration tank influent distribution box was constructed as part of the 1994 improvements. The purpose of the aeration tank influent distribution box is to distribute the primary effluent to the activated sludge system.

4.4.2 Scum Collection

The primary settling tanks are also equipped with surface skimming devices used to remove scum, grease and other floatable debris from the liquid surface. An adjustable slotted scum trough is provided at the effluent end of each primary settling tank bay to facilitate scum removal. To remove floatables from the primary settling tanks, the scum trough is manually rotated, allowing the scum to enter the pipe while limiting the amount of wastewater that can enter. Scum flows by gravity to the scum pumping station.

The scum pumping station is equipped with a mechanical mixer and a submersible chopper pump. Scum from the pumping station can be pumped to a truck for offsite disposal, or can be pumped to the sludge holding tank or sludge thickener or grease frac tank.

4.4.3 Primary Sludge Collection and Pumping

Each primary settling tank bay is equipped with longitudinal mechanical sludge collectors that continually scrape settled solids on the bottom of the tank. The sludge is collected in sludge hoppers where cross collectors direct it to grit pump suction pipe inlets for removal.

Recessed Impeller Grit Pumps

The longitudinal sludge collectors consist of fiberglass flights attached to a plastic chain at 10-foot intervals. The longitudinal sludge collectors operate continuously at a design speed of 2 fpm. The collector mechanism is driven by a 0.5 hp, helical-bevel, electric gear motor manufactured by Eurodrive. A single drive unit operates both longitudinal collectors in each bay of a settling tank.

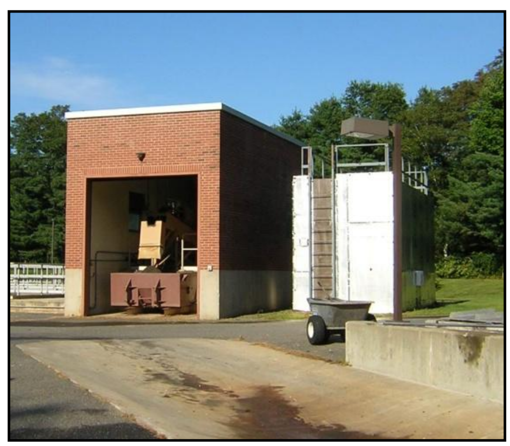
The sludge cross collector is located at the influent end of each primary settling tank. Each cross collector is an 18-inch helical screw, driven by a 0.5 hp, helical-bevel, electric gear motor. The cross collectors move the sludge towards the drive end where the grit pump suction inlets are located.

Four recessed impeller grit pumps, manufactured by Wemco, are provided to convey primary sludge from the settling tanks to the grit separation equipment. The grit pumps are located in the Primary Pump Gallery. The pumps currently installed are from the 1970 upgrade. Modifications were made as part of the 1994 improvements to provide greater operational flexibility. During normal operation, each primary settling tank is served by a separate grit pump. This leaves the fourth pump to serve as a back-up. The piping and valving configuration of the grit pumps allows for any three of the four pumps to be operated concurrently.

Each grit pump has a design capacity of 200 gpm at 25 ft TDH. The pumps are powered by 15 hp motors with variable frequency drives.

4.4.4 Grit Handling

Grit removal is performed downstream of the headworks and is removed from the wastewater after settling with the primary sludge. Thus, because this WPCF is not equipped with grit removal facilities prior to primary settling, the primary sludge contains grit materials, including particles of sand, gravel, and other mineral matter, and minimally putrescible organics such as coffee grounds, egg shells, fruit rinds, and seeds.



Grit Facilities

This grit is removed from the primary sludge to prevent excessive wear on solids handling equipment and to facilitate disposal. A combined grit classifying and grit washing type grit removal system is located in the upper level of the Primary Pump Gallery.

The combined system is a Conanda Grit Washing Plant system manufactured by Huber. The cyclone/wash plant has a design capacity of 250 gpm. The system is driven by a 1 hp totally enclosed motor, operating on 460 volt, 3 phase, 60 cycle electrical service.

The grit cyclone and wash plant are typically operated in the AUTO mode. In this mode of operation, the drive starts automatically whenever the pumps are turned on by the timer. The system is also capable of operation in HAND mode, whereby it is controlled by START-STOP pushbuttons.

Washed and dewatered grit collected in the classifier is discharged to a dumpster located inside the building. The grit is then hauled, using a front-end loader, to a larger roll-off container located adjacent to the septage receiving station where it is combined with screenings. Decant from the grit wash plant is combined with the degritted primary sludge from the grit cyclone and flows by gravity to the gravity thickener.

It should be noted that the existing grit removal system does not currently have any redundancy; there is only one grit cyclone/wash. Should this grit system fail or be taken out of operation for service, there is no method for removing grit from the primary sludge.

4.5 SECONDARY BIOLOGICAL TREATMENT

Wastewater from the primary settling tanks flows via gravity to the secondary biological treatment system. The secondary treatment system is comprised of two separate treatment trains. The first process train includes Aeration Tanks Nos. 1 and 2, Final Settling Tanks Nos. 1, 2 and 3, and Distribution Box No. 3. These process components were constructed during the 1970 plant upgrade.

The second process train, consisting of Aeration Tanks No. 3 and No. 4, Final Settling Tanks Nos. 4 and 5, and Distribution Box Nos. 4 and 5, were constructed as part of the 1994 improvements. Other components of the activated sludge system include centrifugal blowers, return activated sludge pumps, and waste activated sludge pumps.

4.5.1 Effluent Distribution Box

Effluent from the primary settling tanks is discharged to Distribution Box No. 2. This distribution box was constructed as part of the 1994 improvements. Distribution Box No. 2 is designed to split the wastewater proportionally between the four activated sludge tanks. The proportional split is controlled by weirs with differing hydraulic parameters. Weirs No. 1 and No. 2 regulate flow to Aeration Tanks Nos. 2 and No. 1 respectively. These weirs are 2.8 feet long. Weirs Nos. 3 and 4 are 4.0 feet long and regulate flow to Aeration Tanks Nos. 3 and 4 respectively. All four distribution weirs are fixed and are set at the same elevation. Manually operated weir gates are provided immediately downstream of the weirs. The primary function of the weir gates is to isolate flows to individual aeration tanks. Finer adjustments of the flow rate to the individual aeration tanks can be accomplished by altering the elevation of the corresponding weir gates.

4.5.2 Aeration Tanks

Aeration Tanks Nos. 1 and 2 each have a usable volume of approximately 516,000 gallons. Each tank is approximately 177 feet long, 30 feet wide, and has a side water depth of 13 feet. Each aeration tank has two passes and is segmented into six compartments using baffle walls with bottom openings. The tanks are designed to operate in either a plug flow or step-feed mode. An influent channel extends along the entire 177 foot length of each aeration tank. Three weir gates are provided in each influent channel to provide flexibility in the wastewater feed location along the length of the first-pass of each aeration tank.

Aeration Tanks Nos. 3 and 4 each have a volume of approximately 780,000 gallons. Each tank is approximately 174.5 feet long, 45 feet wide, and has a side water depth of 13.3 feet. The tanks are arranged, and have operational flexibility, similar to Aeration Tanks Nos. 1 and 2.

Aeration Tank No. 3

The influent channel for Aeration Tanks Nos. 3 and 4 is located in the center of each tank. Six weir gates along the influent channel allow for feeding primary effluent into any of the six compartments rather than just into the first three compartments as in Aeration Tanks Nos. 1 and No. 2.

Each aeration tank is also equipped with a submerged internal recycle pump. The purpose of the pump is to convey mixed liquor from the last compartment of the aeration tank to the first compartment. An effluent spray system is provided in the last compartment of each aeration tank to knock down foam created in the aeration system.

The effluent from Aeration Tanks Nos. 1 and 2 flows over a 9-foot long effluent weir, into the aeration tank effluent channel. The effluent channel distributes the flow to Final Settling Tanks Nos. 1, 2, and 3. Stop plates partition the channel into separate sections, each feeding a separate final settling tank. Stop Plates Nos. 1 and 2 bisect the effluent weir for Aeration Tank No. 1. Stop Plates Nos. 3 and 4 bisect the effluent weir for Aeration Tank No. 2. Flow distribution to the final settling tanks is controlled by the removal and placement of the stop plates.

The effluent from Aeration Tanks No. 3 and 4 discharges into a common channel running along the inlet side of both tanks. The channel continues around the outside wall of tank No. 4, to Distribution Box No. 5. Prior to Distribution Box No. 5, polymer can be pumped from the Chemical Building and added to the wastewater stream. Distribution Box No. 5 splits the aeration tank effluent proportionally to Final Settling Tanks Nos. 4 and 5.

4.5.3 Distribution Box No. 3 and No. 4

Distribution Box No. 3 is located between Aeration Tanks Nos. 1 and 2 and Final Settling Tanks Nos. 1, 2, and 3. Return activated sludge from the final settling tanks is pumped to the distribution box.

Distribution Box No. 4 is located at the influent end of Aeration Tanks Nos. 3 and 4. Return activated sludge from Final Settling Tanks Nos. 4 and 5 is pumped to the distribution box.

The distribution boxes are designed to split the return sludge to each respective aeration tank. This is accomplished by equal length weirs. Stop plates, located behind the weirs, are provided to isolate the flow of return sludge when an aeration tank is taken out of service.

4.5.4 Final Settling Tanks

Final Settling Tanks Nos. 1, 2 and 3 were initially constructed during the 1970 upgrade and modified as part of the 1994 improvements but are not currently in use. Each tank consists of two longitudinal bays and a cross collection channel. The tanks are approximately 120 feet long, and each longitudinal bay is 12 feet wide. The design side water depth is eight feet. The design average and peak surface overflow rates are 310 gpd/ft^2 and 890 gpd/ft^2, respectively.

Circular Final Settling Tank

Influent enters each settling tank through a dedicated 24-inch diameter pipe. Sluice gates located at the upstream end of each pipe, and slide gates in the aeration tank effluent channel, are used to isolate flow to the final settling tanks.

Final Settling Tanks Nos. 1, 2 and No. 3 are equipped with chain and flight style longitudinal mechanical sludge collectors that continually scrape settled solids on the bottom of the tank. The sludge is collected in sludge hoppers where helical screw cross collectors direct it to sludge pump suction pipe inlets for removal. The sludge collectors were refurbished as part of the 1994 improvements.

Final Settling Tanks Nos. 4 and 5, constructed as part of the 1994 upgrade, are 80-foot circular concrete tanks. The tanks have a design side water depth of 14 feet and design average and peak surface overflow rates of 420 gpd/ft2 and 1,200 gpd/ft2, respectively.

Final Settling Tank Overflow Structure

Influent enters the settling tanks via a vertical pipe center feedwell. Each tank is equipped with an influent baffle and Stamford-type density current baffles on the effluent launder to improve solids settling and reduce the potential for short-circuiting. Each settling tank has a sludge collector mechanism and a scum skimmer assembly. Settled sludge is conveyed to a center sludge hopper by the sludge collection mechanism. Sludge is pumped from the sludge hopper by either return or waste activated sludge pumps. The scum skimmer assembly conveys scum to a scum box where is it is discharged to a scum well. Clarified effluent discharges over a peripheral V-notched weir trough assembly. The effluent from the final settling tanks flow by gravity to Chlorine Contact Tank No. 1.

A buried 12-inch diameter perforated piping system encircles both of the circular final settling tanks. The pipe ties into the storm drain system on the south-west corner of the facility. Presumably, the perforated piping is to provide groundwater dewatering around the tanks. Each tank also has a ten foot wide concrete overflow structure connecting the tanks to the perforated pipe groundwater collection system. The elevation of the concrete overflow structure is approximately one foot higher than the tanks effluent weir.

4.5.5 Aeration System

The aeration system is designed to provide air to all four aeration tanks and the post aeration tank. The aeration system consists of centrifugal blowers, air distribution pipes and valves, aeration grids, flexible membrane diffusers, and a control system. Process air is provided by four centrifugal Hoffman blowers located in the basement of the Operations Building. Each

Aeration Blowers

blower has a maximum rated capacity of 3,700 scfm at 7.9 psig at an inlet air temperature of 100° F. A butterfly valve is provided on the intake line for each blower. This valve can be throttled to provide inlet control of the blowers. All four blowers discharge to a common air header. Separate lines branch off from the air header to distribute air to each of the aeration tanks and the post aeration tank. The discharge rate to each tank is measured by air flow sensors in the distribution lines.

Each air line to the aeration tanks and post aeration tank has a motorized butterfly valve for air balancing. The amount of air directed to each tank is controlled based on the level of dissolved oxygen (DO). A DO sensor is located in each of the aeration tanks and the post aeration tank.

The aeration tanks for both secondary treatment trains utilize a fine bubble diffused aeration system. Each aeration tank has six air distribution grids, one for each compartment. The air distribution system grids employ horizontal aeration piping with circular membrane fine bubble diffusers.

In addition to the balancing valves located on the header to each tank, a motorized butterfly valve is located on each drop pipe to each of the twenty-four individual compartments. The purpose of this valve is to shut off air flow to individual compartments.

4.5.6 Return Activated Sludge (RAS) Pumps

Four return activated sludge pumps (RS-1, RS-2, RS-3, and RS-4) are located in Secondary Pump Gallery No. 1 to serve Aeration Tanks No. 1 and No. 2, and Final Settling Tanks No. 1, No. 2, and No. 3. The RAS pumps discharge to Distribution Box No. 3. The pumps are also capable of dewatering Final Settling Tanks No. 1, No. 2, and No. 3, but not the aeration tanks.

Secondary Pump Gallery No. 2

Three return activated sludge pumps (RS-5, RS-6, and RS-7) are located in Secondary Pump Gallery No. 2 to serve Aeration Tanks No. 3 and No. 4, and Final Settling Tanks No. 4 and No. 5. The RAS pumps discharge to Distribution Box No. 4. The pumps are capable of dewatering Aeration Tanks No. 3 and No. 4, and Final Settling Tanks No. 4 and No. 5.

In each process train, one return activated pump is dedicated to one final settling tank. A standby pump is provided for each system. The piping layouts allow for any one pump to be taken out of service. A magnetic flow meter is provided on the discharge pipe of each pump.

All of the pumps are equipped with an AC variable frequency drive (VFD). The VFDs have a selector switch for Automatic or Manual operation. During Automatic operation, the pump discharge is paced to the plant influent flow meter. In the Manual mode of operation, the pump speed is controlled by a speed control potentiometer.

4.5.7 Waste Activated Sludge (WAS) Pumps

Two waste activated sludge pumps (WS-1 and WS-2) are located in Secondary Pump Gallery No. 1, and two waste activated sludge pumps (WS-3 and WS-4) are located in Secondary Pump Gallery No. 2. These two sets of pumps convey waste sludge from Final Settling Tanks No. 1, 2, and 3 and Final Settling Tanks No. 4 and 5, respectively, to the sludge thickening process. The waste activated sludge and secondary scum is pumped to Waste Sludge Holding Tanks No. 1 and No. 2 and to Auxiliary Sludge Holding Tank No. 1.

4.5.8 Disinfection and Effluent Discharge

Effluent from the final settling tanks is disinfected with chlorine seasonally, May 1st through September 30th, prior to discharge to the Naugatuck River. Following disinfection, the effluent flow is dechlorinated and then aerated in a post-aeration system to meet effluent permit limits for dissolved oxygen. In addition, during periods of high river level, an effluent pumping system is available. Each of these unit processes is discussed separately below.

<u>Chlorination and Dechlorination</u>

Effluent from the final settling tanks is routed to the chlorine contact chamber for disinfection. The chlorine contact chamber consists of two tanks operating in series. Chlorine Contact Tank No. 1 has a capacity of approximately 168,000 gallons. The overall dimensions of the tank are 70 feet long by 35 feet wide, with a side water depth of nearly 9 feet at peak flows. The tank contains baffle walls to create a serpentine flow pattern. The baffle walls reduce the length to width ratio by creating a flow pattern with an equivalent length of approximately 325 feet and an equivalent width of approximately 7 feet 6 inches (43:1). The baffles work to eliminate short-circuiting and increase the total detention/contact time.

Chlorine Contact Tank No. 2 was constructed as part of the 1994 improvements by modifying a portion of one of the existing aeration tanks. Chlorine Contact Tank No. 2 has an approximate capacity of 170,000 gallons. The tank has three passes that are approximately 169 feet long and 4 feet 6 inches wide (nearly 40:1), with a side water depth of nearly 9 feet at peak flows.

Changes to the chlorination and dechlorination process were made following the plant improvements completed in 1994. Disinfection is accomplished using liquid sodium hypochlorite. Sodium hypochlorite is stored in a 4,000 gallon, above-grade polyethylene storage tank located in the Administration Building, adjacent to the Septage Receiving

Station. The sodium hypochlorite solution is metered into a plant water carrier system and discharges to a chlorine mixing chamber at the inlet of Chlorine Contact Tank No. 1. There are currently no provisions for automated chlorine monitoring; the chemical feed pumps are paced to the plant flow.

Sodium Hypochlorite System

Dechlorination of the plant effluent is required prior to discharge. Liquid sodium bisulfite is used for dechlorination. The sodium bisulfite is stored in the Chemical Building. Two above-ground polyethylene storage tanks, each with a capacity of 2,000 gallons, are provided. The dechlorination feed system is similar to the chlorination system. The sodium bisulfite is metered to a plant water carrier system. The dechlorination solution is conveyed to the end of Chlorine Contact Tank No. 2.

Disinfection takes place on a seasonal basis. During the period when disinfection is not required by the WPCF's discharge permit (October through April), Chlorine Contact Tank No. 1 can be bypassed. The wastewater flow can completely bypass both chlorine contact tanks, which also bypasses the post-aeration system. Alternately, the wastewater flow from Chlorine Contact Tank No. 1 can be directed to Chlorine Contact Tank No. 2, or directly to the plant outfall structure. It is not expected that the disinfection season will be changed in the next NPDES permit reissuance.

The chlorine contact tanks provide approximately 73 minutes of detention time at the average daily flow of 7.0 MGD and approximately 24 minutes of detention time at the current peak design flow of 20.0 MGD. The highest flow that can be detained for the full 30 minutes as prescribed by *Technical Resource 16 (TR-16) Guides for the Design of Wastewater Treatment Facilities* is 16.1 MGD.

From May 2007 through end of September 2011, during the disinfection season, the Torrington WPCF only observed instantaneous and / or total flows above 16.1 MGD between

August 28th - 29th, 2011 and September 7th - 9th, 2011. The August 2011 peak flows occurred as a result of Hurricane Irene on the 28th of August, 2011. The September peak flows are correlated with rain events that left a total of 5 inches of rain in 3 days. During these high flow events, the NPDES permit was not violated for fecal coliform counts.

Post-Aeration

A post-aeration system is provided to increase the dissolved oxygen content of the plant effluent to the permit limit of 5 mg/l. The post-aeration system is directly downstream from the chlorination and dechlorination system. The post-aeration was constructed as part of the 1994 improvements by modifying a portion of one of the existing aeration tanks and has an approximate capacity of 102,000 gallons. The tank is approximately 101 feet long, 15 feet wide, and has a side water depth of 9 feet.

Post-Aeration & Effluent Discharge

Post-aeration air is supplied from the aeration blowers. The air flow rate is controlled by the aeration control system and a dissolved oxygen probe located at the effluent end of the post-aeration tank. The post-aeration diffusers are of the same type and size as the activated sludge diffusers. The effluent from the post-aeration tank flows via a 36-inch diameter pipe to the Outfall Structure, then ultimately to the Naugatuck River.

Outfall Pumps

The final treated effluent from the Torrington WPCF is normally discharged to the Naugatuck River by gravity flow. During periods of extreme wet weather and seasonal flooding, the elevated water level in the Naugatuck River necessitates pumping of the effluent. This is accomplished with an outfall pumping system.

Outfall Pumps

As part of the 1994 plant upgrade, a portion of the concrete tank, previously used as an aeration tank, was partitioned to create an outfall pump wetwell. The wetwell is equipped with three 125 hp submersible, vertical, open volute axial flow type outfall pumps (OP-1, OP-2, and OP-3) manufactured by M&W Pump Corporation. The rated capacity of each pump is approximately 11,800 gpm at 33 feet TDH. The existing pumps are operational; however, they are 18 years old and approaching the end of their useful life. Due to the age of the system, the existing pumps have had costly seal failures and bearing failures.

Each pump and motor is housed in a 30-inch diameter discharge column. The top end of the discharge column is covered with a plate bolted to the unit. Pump maintenance and inspection requires removal of the bolted plate cover. Although lifting lugs are provided on the pumps, access to the pumps for maintenance, inspection, or removal is difficult, and presents many safety issues.

4.6 SOLIDS HANDLING FACILITIES

The solids handling process includes gravity thickening, gravity belt thickening, thickened and unthickened sludge storage, and sludge pumping.

4.6.1 Primary Sludge Thickening and Pumping

Degritted primary sludge from the grit cyclone and wash water from the grit classifier is conveyed to the gravity thickener. The gravity thickener tank and pumping equipment were constructed as part of the 1970 plant upgrade.

The gravity thickener is a 40-foot diameter tank, equipped with a center-supported, rotating sludge collector mechanism. The design side water depth is 10 feet. The design solids loading rate is 9.8 lbs/day/sq. ft.

During the winter months, the gravity thickener is typically used to co-thicken primary and secondary sludge. Generally, combined primary and secondary solids do not achieve as high a solids concentration as straight primary sludge using the gravity thickening process. However, the Torrington WPCF operations staff reports no detrimental impact during co-thickening operations. During the warmer months, when the facility is not co-thickening the primary and secondary sludge in the gravity thickener, the secondary sludge is thickened via the gravity belt thickener.

Gravity Thickener

The sludge collector mechanism pushes the thickened sludge towards a collection hopper, located at the bottom center of the tank. Pickets on the sludge collector mechanism enhance the sludge thickening by helping to release water and gas trapped in the sludge. The pickets also help prevent bridging of the sludge solids.

The thickener supernatant passes over weirs mounted along the upper edge of the tank, and is conveyed to Distribution Box No. 1 upstream of the primary clarifiers. The supernatant generally has high concentrations of BOD_5 and TSS.

The thickened primary sludge is pumped from the gravity thickener by two 4-inch Double-Disc positive displacement pumps (TP-1 and TP-2) manufactured by Penn Valley. There are in-line grinders on the suction side of each pump. Each pump has a rated capacity of 170 gpm at 30-ft TDH. The pumps are belt driven by a 5 hp, 1,200 rpm motor.

4.6.2 Secondary Sludge Thickening and Pumping

When not being co-thickened with primary sludge, secondary sludge is thickened on the gravity belt thickener (GBT). Other components of the secondary sludge thickening process include feed pumps, a thickened sludge pump, and a polymer feed system. The gravity belt thickener and ancillary pumping equipment was installed as part of the 1994 upgrade.

The gravity belt thickener is located on the first floor of the Operations Building. The unit, manufactured by Klein, has an effective belt width of 1.5 meters. Gravity belt thickeners are typically sized based on the hydraulic loading per meter of belt width. The design hydraulic capacities of the gravity belt thickener at the Torrington WPCF are 200 gpm/meter belt width at a feed solids concentration of 1.5 percent, and 250 gpm/meter belt width at a feed solids concentration of 0.5 percent.

The gravity belt thickener consists of a single moving belt mounted on a system of rollers. The feed sludge is conditioned with polymer and uniformly distributed on the belt. The free water drains through the porous belt. Drainage is enhanced by plow-shaped chicanes which shift sludge sideways to continually expose areas of clean belt. Thickened sludge is discharged to a hopper. Filtrate and wash water are discharged to a wetwell, and pumped to Distribution Box No. 2 upstream of the aeration tanks.

The unthickened secondary sludge is pumped from the sludge storage tanks to the gravity belt thickener by positive displacement feed pumps. The three sludge feed pumps (TF-1, TF-2, and TF-3) are 4-inch Penn Valley Double-Disc pumps. The pumps are located in the basement of the Operations Building. The original design concept was based on two pumps to feed the gravity belt thickener, with the third pump serving as a back-up. Each pump has a rated capacity of 170 gpm at 32-ft TDH. Each pump is driven by a 5 hp, 1,200 rpm motor manufactured by Baldor Electric and is equipped with a variable frequency drive.

Thickened sludge from the gravity belt thickener discharge hopper is conveyed to the thickened sludge holding tanks using the thickened sludge transfer pump (TA-1). The thickened sludge transfer pump is a rotary lobe pump manufactured by Boerger. The pump has a rated capacity of 70 gpm at 100-ft TDH and operates between 40-80 gpm depending on the thickness of the sludge.

4.6.3 Sludge Storage

As part of the 1994 facility improvements, the existing aerobic digester tank was upgraded and divided into six compartments, creating six sludge storage tanks. The sludge storage tanks were compartmentalized to provide maximum flexibility in sludge processing and storage operations and to minimize the production of odors. The individual tanks can function as intermediate sludge storage before final processing, final storage before disposal, and emergency storage during plant upsets or disruptions in sludge processing operations. The sludge storage tanks are designated as follows and described further below:

- Unthickened Waste Sludge Holding Tanks (USHT No. 1 and USHT No. 2)
- Thickened Sludge Holding Tanks (TSHT No. 1 and TSHT No. 2)
- Auxiliary Sludge Holding Tank (ASHT No. 2)

<u>Unthickened Sludge Holding Tanks:</u>

The unthickened sludge holding tanks function as storage tanks for the waste activated sludge prior to thickening. Each tank is 49'-6" long and 22'-6" wide. The effective depth is 15'-0" which provides a storage volume of 125,000 gallons per tank. The tanks are equipped with a coarse-bubble aeration system to provide mixing. The aeration helps maintain the "freshness" of the sludge, preventing it from going septic and minimizing odors. Limited thickening of the secondary sludge is accomplished by shutting off the aeration, allowing the sludge to settle, and partially decanting the tanks.

<u>Thickened Sludge Holding Tanks:</u>

The thickened sludge holding tanks provide storage for thickened primary and/or secondary sludge. Each tank is 14'-0" long and has an effective depth of 11'-0". TSHT No 1 is 12'-0" wide, providing a storage volume of 13,825 gallons. TSHT No. 2 is 9'-6" wide providing a storage volume of approximately 10,945 gallons. The tanks have top-entering mechanical mixers to keep the thickened sludge in suspension. The tanks are designed to allow thickened primary sludge and thickened secondary sludge to be stored separately. However, in current operations, the two sludges are combined for storage.

Auxiliary Sludge Holding Tank:

The auxiliary sludge holding tank provides a source of additional storage. The tank is not generally used as part of the normal operation of the facility. Its function is to provide additional storage for either unthickened or thickened sludge during shutdowns for equipment maintenance or emergency situations. Tank dimensions are 42'-0" long and 22'-6" wide. The effective depth is 15'-0" which provides a storage volume of 106,000 gallons. The tank is equipped with a coarse-bubble aeration system to provide mixing and aeration.

It should be noted that one of the compartments, Auxiliary Sludge Holding Tank No. 1, was converted to a FOG receiving tank during a 2009 upgrade and is no longer available for sludge storage.

4.6.4 Truck Loading Facilities

The thickened sludge is transferred from the thickened sludge storage tanks to tanker trucks for off-site disposal using two Boerger pumps. The pumps are each rated for 400 gpm at 46 feet of TDH. A 6,500 gallon truck can be loaded in 30 to 90 minutes depending on the liquid level in the thickened sludge storage tank and the solids concentration of the thickened sludge.

Because additional fees are charged if truck loading takes greater than one hour, this can be problematic. To facilitate effective pumping, the level in the thickened sludge storage tanks was maintained at greater than 50% of tank depth and thickening was limited to an average value of 6% solids concentration.

Truck Loading Facilities

The existing truck loading station is located adjacent to the Operations Building. There is no spill containment for the truck loading area. Any spillage during the loading operations would discharge to the ground, and could potentially reach the stormwater drainage system.

SECTION 5

WRIGHT-PIERCE
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SECTION 5

EVALUATION OF EXISTING OF PRELIMINARY AND PRIMARY TREATMENT PROCESS FACILITIES

5.1 INTRODUCTION

Each existing liquid treatment process stream and related components were evaluated with regard to a) the current condition of the existing component / process; and b) for the ability of each component / process to meet both current and known future regulatory limits at projected flows and loads for the planning period. This section highlights the improvements needed for the Preliminary and Primary Treatment components of the Torrington Facility to continue to meet all of their regulatory limits, with the exception of Secondary Treatment (including nitrogen related nutrient removal requirements (i.e. TN, ammonia, etc.) and Tertiary Treatment related to the pending phosphorus removal requirements. Section 6 presents the evaluation of the Secondary Treatment processes, including disinfection and Section 7 presents the evaluation of phosphorus removal alternatives to meet the pending Total Phosphorus limit for this facility.

For each system evaluated, the existing conditions and operations are summarized, highlighting suggested process improvements. Various alternatives are presented and evaluated for applicability to the Torrington facility. The evaluations are based on both capital costs as well as operation and maintenance issues. For each process requiring an upgrade, a recommendation is presented.

The existing preliminary treatment facility includes mechanical screening located in the Screenings Building and separate septage receiving facilities. Grit is currently settled in the primary sedimentation tanks and removed from the primary sludge with a combined grit classifier/washer unit. The Torrington WPCF has recently replaced their grit classifier/washer unit and thus additional alternatives for upgrading this system have not be evaluated.

Several operational concerns are evaluated for the primary treatment system.

5.2 PRELIMINARY TREATMENT - MECHANICAL SCREENING

Coarse screening of the influent wastewater is accomplished by two self-cleaning mechanical bar screens. The screens are functional and are in good structural condition but are ineffective at adequately screening the wastewater. Debris in the wastewater either passes through the wide screen openings or is not effectively removed from the screen and reintroduced into the wastewater flow on the downstream side of the screens.

The screens also require extensive maintenance and operator attention. The most frequent maintenance issue is related to constant wear on the drive chains and shear pins failures. Because the screens are fixed in the channel, the screen has to be removed from service in order to perform any maintenance. Furthermore, access to components below the floor level is also constrained by limited access openings - there is only one 2'-0" x 3'-0" opening both upstream and downstream of the screens.

The raw screenings are odorous and difficult to handle. The screenings have relatively high moisture content and contain fecal and other organic matter. Operator handling of the screened material is required at least twice prior to ultimate disposal, representing an inefficient use of the operator's time. The screenings material is co-mingled with the grit material and disposed of at a landfill. Costs for hauling and disposal of the screenings have been volatile during the past few years. Furthermore, landfill capacity is becoming limited in the region.

The Department of Energy and Environmental Protection (DEEP) will permit the disposal of wastewater screenings as Municipal Solid Waste (MSW) if the screenings have undergone grinding, washing, and dewatering. Therefore, a common element to all screenings alternatives considered in this evaluation is the provision for grinding, washing and dewatering of screenings.

5.2.1 Development of Screening Improvements

Several alternatives were investigated to address the operational deficiencies, maintenance issues and disposal concerns related to the screening operation. The goals of the improvement alternatives are summarized as follows:

- Improving process control
- Reducing operation and maintenance costs
- Improving screenings removal efficiency and protecting downstream equipment
- Reducing odors associated with screenings
- Reducing screenings disposal costs
- Achieving compliance with DEEP regulations regarding screenings disposal as MSW
- Provide sufficient space and access for the plant staff to maintain the equipment

Alternative No. 1 - New Combined Channel Grinder and Wash Press

This alternative involves the replacement of the existing mechanical screens with a single piece of screening equipment designed to combine all of the functions - screening and immediate grinding, washing and compaction of the screenings. This type of unit is capable of grinding, washing, dewatering, and compacting influent screenings to the same quality as a stand-alone screenings grinder and wash press. As of the writing of this plan, only the proprietary Auger Monster/Screenings Washer unit as manufactured by JWC has the capability of performing these functions.

A disadvantage to these type units is that they are hydraulically limited to flows of approximately 6.5 MGD. Thus, for the Torrington facility, three (3) separate units could be installed, one in each of the existing channels; this would provide screening for the future maximum month flow condition and approximately the 98 percentile of the future peak flow. Thus, a fourth channel with manual bar rack would need to be constructed to accommodate hydraulic conditions greater than the maximum month flow.

The new units would be installed inside of the existing Screenings Building; extensive modifications to the influent channel and building would be required including a 10-foot by 20-foot extension at the northern end of the Screenings Building. This addition would provide for the space requirements to accommodate the new equipment. A separate 10-foot by 20 foot extension parallel to the two existing channels would be required for the construction of the fourth influent channel required to accommodate high flow conditions. Screenings from the new equipment would discharge into containers located inside of the Screenings Building. The concrete floor above the influent channels would be demolished and replaced with aluminum diamond plating with access panels as appropriate.

Alternative No. 2 - New Mechanical Screens and One Grinder/Wash Press

The second alternative would entail the replacement of the existing mechanical screens with two (2) new climber or step-type mechanical screens. These types of screen units have no, or few, mechanical parts below the water surface. This style screen reduces the mechanical wear and makes equipment maintenance much more convenient and operator friendly. The new screens would be installed in the same location as the existing mechanical screens. Depending on the selected screen model, the roof may need to be raised to accommodate the height requirements of the new screens.

This alternative would also require the installation of a belt or screw conveyor system to transport the screenings from each of the units to a single new grinder/wash press. An addition to the existing building, approximately 10-foot by 20-foot, would be required to house the discharge chute of the conveyor as well as the new screenings grinder/washer and associated discharge receptacle. The concrete floor above the influent channels would be demolished and replaced with aluminum diamond plating with access panels as appropriate.

Alternative No. 3 - New Screens and Two Grinder/Wash Presses

The third alternative is similar to second in that the existing mechanical screens would be replaced with new climber or step type screens. However, in this alternative, the new screens would be staggered in their respective channels approximately 8-feet apart allowing for the installation of two new screenings grinder/wash presses located on the discharge side of each screen. Again, the concrete floor above the influent channels would be demolished and replaced with aluminum diamond plating with access panels as appropriate.

As was the case with the second alternative considered, the roof of the Screenings Building may need to be raised to accommodate the height requirements of the new screens. Additional building modifications would include a 10-foot extension of the Screenings Building to the north and a 10-foot extension of the screenings building to the east, and extensive modifications to the influent channels. The expansion of the building footprint and the staggering of the new screens would provide adequate room to install the two new grinder/wash presses inside of the existing building.

5.2.2 Evaluation of Screening Alternatives

Both Capital and Operation & Maintenance (O&M) costs, including energy costs, for each of the three pretreatment alternatives were evaluated and a present worth analysis was developed. A summary of the present worth analysis is presented in **Table 5-1**. The Capital Costs represent only costs associated with the specific alternative presented and are not representative of the total cost to construct the alternative. Capital costs include the Contractor's costs associated with Overhead and Profit and General Conditions based on 15% of the estimated construction costs included in the analysis. The estimated alternative-specific Construction Costs also include a 15% design contingency and 5% construction contingency. This present worth of the O&M cost is based on 3%/year inflation and a 2% interest rate over a 20-year life cycle. Costs are based on 2012 dollars and do not include engineering or contingency.

TABLE 5-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
SUMMARY OF SCREENING ALTERNATIVES COSTS
(March 2012)

Alternative	Description	Capital Cost	NPW of Annual Costs
1	Combined Screen/Grinder/Wash/Press Units	$ 1,451,000	$ 3,305,700
2	Mechanical Screens, Single Grinder/Wash/Press	$ 1,553,400	$ 2,823,900
3	Mechanical Screens and Dedicated Grinder/Wash/Press	$ 1,610,800	$ 2,767,000

Alternative No. 1 has the lowest capital cost but has a greater present worth because the screening/grinder/wash units will operate continuously resulting in a greater overall energy cost. Alternative No. 2 has the lowest capital costs but a higher net present worth cost and does not effectively address the issue of the space restrictions within the screenings building. Alternative No.3 has the highest capital costs but the lowest net present worth costs of the three alternatives presented. Furthermore, installation of two grinder/wash/press units provides redundancy for that part of the equipment.

The advantages and disadvantages of each alternative are summarized below.

TABLE 5-2
CITY OF TORRINGTON WPCF FACILITIES PLAN
SUMMARY OF PRELIMINARY TREATMENT ALTERNATIVES
ADVANTAGES AND DISADVANTAGES

Alternative	Advantages	Disadvantages
1	• Lowest capital cost	• Greater present worth costs • Higher operating cost
2	• Minimizes influent channel modifications •	• Maintenance of belt conveyor • Only one wash press (no redundancy)
3	• Sufficient redundancy for all major pieces of equipment • Provides sufficient space to maintain all equipment • Lowest Present Worth	• Highest capital cost

5.2.3 Recommended Screening Improvements

Alternative No. 3 is the recommended alternative because it has the lowest net present worth and it provides additional space within the Screenings Building which will provide for easier maintenance of all equipment. In addition, this alternative provides for the use of two separate grinder/wash/press units and eliminates the need for a belt or shaftless screw conveyor.

Alternative 3 includes:

- Replacing the existing mechanical bar screens with screening equipment that operates more efficiently and is easier to maintain than the installed units.
- Installation of screenings handling equipment (grinding, washing, dewatering / press) equipment. A dedicated unit would be provided for each influent screen.
- Removing concrete flooring above the screen channels and installing aluminum diamond plating with integral access panels.
- Expanding the building footprint and ceiling height to provide sufficient access to all equipment for operation and maintenance activities.

5.3 PRELIMINARY TREATMENT - SEPTAGE RECEIVING

The Torrington staff has reported both equipment issues and operational related issues with the existing septage receiving facility, as summarized below.

Septage unloading had traditionally been conducted based on an honor system. For each load, the truck driver self-reported to the operations staff the volume of septage discharged. To eliminate or reduce the potential for abuse of this method of determining the size of each load, plant personnel fabricated a portable septage metering system. This system is used to measure septage deliveries with manual reading of septage volumes. This is considered an interim solution, since it requires considerable operator time when compared to an automatic metering system.

Septage is unloaded into the tank through a coarse bar rack; which is a relatively ineffective screening mechanism. Because the septage is pumped to the channel downstream of the plant influent mechanical bar screens, materials that pass through the coarse manually-cleaned bar rack at the septage receiving facility also enters the primary settling tanks and downstream unit processes.

The septage holding tank is difficult to empty and clean because the existing pumps cannot pull down all septage due to the long suction piping. In addition, the ineffectiveness of the manually-cleaned bar rack at the septage facility results in continuous housekeeping problems.

The existing mixer does not provide sufficient mixing of the septage tank contents. The mixer impeller is located too high in the tank to mix at lower tank depths and volumes and therefore only mixes a portion of the tank contents.

The septage pump is located in the primary settling tank pump gallery. This configuration requires an 80-foot suction pipe and an 80-foot discharge pipe which causes operational difficulties with the pump. These existing double disk positive displacement pump (manufactured by Penn Valley) is not able to pull thicker materials, like grease. Thus, when the loads consist of thicker material, this pump is not able to completely empty the septage receiving tanks.

5.3.1 Development of Septage Receiving Improvements

Several improvements were considered to address the operational issues associated with the septage receiving facility as outlined above. This included evaluating modifications to the following three (3) components of the system:

- metering and control system related alternatives;
- septage tank mixing alternatives; and
- septage pump alternatives.

Metering and Control System Modifications

Three alternatives were considered regarding automating the septage receiving metering and control system and are described as follows:

Alternative No. 1 - Install a New Liquid Level Monitoring System

This alternative would consist of installing a liquid level monitoring system, such as an ultrasonic level sensor, inside of the septage holding tank to measure the volume of septage discharged by the septage hauling trucks. A simple control system, such as the Milltronics MiniRanger Plus could be used to calculate the volume of septage discharged into the tank by converting the change in liquid level to gallons. The accuracy of this system would be plus or minus 200-300 gallons which is about 10% of the volume of a typical septage truck. To ensure proper recording of volume for billing purposes, the septage pump could not be allowed to operate during the unloading process. Locking the spetage pump out during truck unloading was not practical and this alternative was not considered viable.

Alternative No. 2 - Install a New Magnetic Flow Meter and Access Control Panel

This alternative would entail the installation of a magnetic flow meter and approximately 8-feet of piping from the existing 4-inch line in the bar screen channel to a new 4-inch quick disconnect positioned in the center of the existing septage unloading pad. All piping and valves would be installed in a heated aluminum enclosure to ensure proper operation over the winter months. The septage haulers would enter their code number or insert a key card into a control box mounted inside of the enclosure. Upon acceptance of the pin code/key card, a motorized ball valve would open allowing the truck to discharge through the flow meter and into the channel. The flow meter would record the volume of septage discharged from the truck for billing purposes.

This alternative has the following advantages:

- Greater accuracy in measured volume for large and small loads
- Does not require operator attention during truck unloading

This alternative has the following disadvantages:

- Higher installation cost
- Requires minor structural modifications
- Does not improve septage screening efficiency

Alternative No. 3 - Install a New Septage Pretreatment Unit

The third alternative would include the installation of a septage receiving unit, magnetic flow meter, and pin code/key card control panel. The septage haulers would enter their code number or insert a key card into a control box mounted inside the new septage building. Upon acceptance of the pin code/key card a motorized ball valve would open allowing the truck to discharge through the flow meter and into the septage receiving unit. The flow meter would record the volume of septage discharged from the truck for billing purposes. The septage receiving unit would provide fine screening and grit removal. Screenings and grit from the septage receiving unit would discharge into a container inside of the building and screened, de-gritted septage would flow by gravity into the existing septage holding tank.

As discussed above, the existing manual septage screen is inefficient and ineffective. Thus, if a new septage receiving facility is installed, screenings removal would be greatly improved.
This alternative has the following advantages:

- Greater accuracy in measured volume for large and small loads
- Does not require operator attention during truck unloading
- Removes screenings from septage stream

This alternative has the following disadvantages:

- Highest installation cost
- Requires structural modifications

Septage Tank Mixer Modifications

Two alternatives were considered regarding the replacement or modification of the septage tank mixer to allow for better mixing of the tank contents. Having the tank contents completely mixed would mean that a more homogenous waste stream would be introduced to the downstream process.

Alternative No. 1 - Modify the Existing Mixer

This alternative would involve replacing the existing mixer shaft and impeller with a new, longer shaft and improved impeller to better mix the contents of the tank. The existing mixer manufacturer, United Equipment Technologies, was contacted and indicated that new impeller designs have been developed that could significantly improve the mixing efficiency of this mixer.

This alternative has the following advantages:

- Lowest installation cost

This alternative has the following disadvantages:

- Lower efficiency
- Motor is approaching 20 years old

Alternative No. 2 - Install a New Mixer

This alternative would involve replacing the existing mixer with a new mixer or mixing system to more effectively mix the contents of the tank. The mixer would be controlled with a timer to reduce electrical costs and equipment wear.

This alternative has the following advantages:

- Replaces aged equipment
- Increases life of equipment
- Higher efficiency

This alternative has the following disadvantages:

- Higher installation cost

Septage Metering Pump Modifications

To increase the ability of the septage pumps to handle all flow conditions, two separate alternatives were considered - including the replacement or modification of the existing septage pump operation.

Alternative No. 1 - Modify Existing Pump Discharge Piping

This alternative would leave the existing septage pump in operation and extend the existing discharge piping to a new discharge point upstream of the mechanical screening equipment.

This alternative has the following advantages:

- Lowest installation cost
- Provides additional screening of the pumped septage material

This alternative has the following disadvantages:

- Requires long suction and discharge piping, causing a lower pumping efficiency and operational difficulties
- Increases screening operation in headworks
- Does not address the inability to completely empty the tanks

Alternative No. 2 - Install New Submersible Septage Pump

This alternative would involve the installation of a new submersible type chopper pump in the septage tank. The new pump would be mounted on slide rails to facilitate removal for maintenance. A new access hatch would be constructed in the tank over the new pump. A second pump could be purchased as a shelf spare or the existing septage pump and piping could be left in place as a back-up to the new system. New level controls and a timer would be installed to facilitate the operation of the new pump. This alternative also includes new pump discharge piping with the discharge point upstream of the mechanical screening equipment.

This alternative has the following advantages:

- Lowers suction elevation
- Submersible pump will have a higher efficiency
- Chops solids during pumping
- Aids in tank draining and cleaning process

This alternative has the following disadvantages:

- Higher installation cost

5.3.2 Evaluation of Septage Receiving Alternatives

Of the modifications and alternatives discussed above, the option to install a new stand-alone septage receiving unit would provide the most comprehensive benefit to the process. The septage could be screened at the dumping location versus collecting septage and pumping it back to the head of the plant for mechanical screening. This option also will not eliminate the need for a submersible pump or a septage tank mixer because the septage receiving unit effluent will flow by gravity to the existing septage storage tank. It would be recommended that the facility maintain the flexibility to store septage and meter it into the headwork influent flow during low flow periods.

Alternatively, two other options investigated included modifying the existing mixer in the septage tank so that contents would be mixed well. The existing mixer is not currently configured correctly to provide proper mixing in the existing septage holding tank. While replacing the impeller and shaft would improve the mixing, a more reliable long term approach would be to provide a new mixer or mixing system specifically designed to provide the necessary mixing requirements. Therefore, a new mixer was considered the most beneficial modification to implement at the septage receiving area provided that the first alternative above was not implemented.

A new mixer was evaluated in combination with the other modifications (metering, pumping, etc.) discussed above.

Option 1

Option 1 includes the installation of a new magnetic flow meter and access control panel at the receiving station, a new mixing system within the existing tank and modifying the existing septage pump's discharge piping.

Option 2

Option 2 includes the installation of a new magnetic flow meter and access control panel at the receiving station, installation of a new mixing system within the existing tank and installation of a new submersible pump within the septage tank.

Option 3

Option 3 includes the installation of a complete new stand-alone septage receiving pretreatment unit. The stand-alone pretreatment unit would screen the septage material, eliminating the need to pump the septage to the mechanical screens at the head of the plant. However, a submersible pump and mixer is still included in this option in order to maintain flexibility to meter septage into the influent flow during low flow/loading periods. The screened septage effluent would be fed by gravity to the existing septage storage tank to be metered into the headworks process for treatment with the influent wastewater flow. The screenings and grit material from the package unit would then be disposed of separately with the addition of a bag, cart, or container bay located at the discharge of the solids chute. The stand-alone septage receiving pretreatment unit would also grind, wash and dewater all screenings removed by the unit, in order to meet the DEEP requirements for disposal as MSW.

The alternative specific costs for constructing the septage receiving improvement options were evaluated and summarized in **Table 5-3**. These costs also include 15% for the Contractor's costs (OH&P and General Conditions) and design contingency as well as a 5% construction contingency.

TABLE 5-3
CITY OF TORRINGTON WPCF FACILITIES PLAN
COST COMPARISON OF SEPTAGE RECEIVING IMPROVEMENT OPTIONS

Description	Idividual Alternative Cost	Combined Costs	Contractor General Conditions OH&P	Design Contingency	Construction Subtotal	Construction Contingency (5%)	Total Option Specific Construction Cost
New Magnetic Flow Meter	$ 52,900						
New Submersible Septage Mixer	$ 24,900	$ 91,600	$ 13,740	$ 15,800	$ 121,140	$ 6,057	$ 127,200
Modifying Pump Discharge	$ 7,100						
Septage Bar Rack	$ 6,700						
New Magnetic Flow Meter	$ 52,900						
New Submersible Septage Mixer	$ 24,900						
New Submersible Septage Pump	$ 64,200	$ 155,800	$ 23,370	$ 26,900	$ 206,070	$ 10,304	$ 216,400
Modifying Pump Discharge	$ 7,100						
Septage Bar Rack	$ 6,700						
Installation of complete Septage Receiving System	$ 229,000			$ 56,100			
New Submersible Septage Mixer	$ 24,900	$ 325,200	$ 48,780		$ 430,080	$ 21,504	$ 451,600
New Submersible Septage Pump	$ 64,200						
Modifying Pump Discharge	$ 7,100						

Note: Option 3 assumes that the additional building square footage needed for this new receiving system will be accommodated in the new building additions for the recommended Screenings Alternate 3.

For ease of evaluation, the operational costs, including electrical cost and required staffing, was assumed to be essentially the same for each alternative, and the O&M cost differential between alternatives was assumed to be negligible. However, it should be noted that Option 3 would have a positive impact on O&M costs which helps to rationalize a higher capital cost alternative.

5.3.3 Recommended Septage Receiving Improvements

It is recommended that the Torrington Facility implement Option 3 because it fully addresses the noted operations issues identified with the existing septage receiving facilities. Option 3 provides the facility with a viable, long term solution. Although Option 1 and Option 2 have lower capital costs, Option 3 eliminates additional operation and maintenance of a Septage tank mixer and submersible pump in the future due to rag buildup and grit within the existing tank.

Option 3 would provide additional screening and grit removal, from the septage making this option the most efficient. This option also decreases additional wear and tear on the Headworks Screening equipment associated with Option 1 and 2.

Option 3 includes the following improvements:

- Installation of a complete septage receiving unit to grind, wash, dewater, and screen septage to enter downstream into the treatment process at the WPCF headworks;
- Installation of a complete Septage receiving unit that includes a control system to automatically meter and record the volume of septage discharged from each hauler with PIN or card access features for security; this would replace the current interim portable flow meter system;
- Elimination of the septage tank mixer, submersible chopper type septage pump, and septage pump piping and discharge; and
- Centralization of septage and WPCF headworks influent wastewater and equipment.

5.4 GRIT FACILITIES

Similar to many WPCF designed in the 1970s, Torrington's facility does not provide for separate grit removal ahead of the primary clarifiers. At the Torrington facility, grit settles in the primary clarifiers and is removed from the primary sludge using a cyclone separator and grit washing classifier. This method of grit removal has several disadvantages including:

- It requires a relatively high primary sludge pumping rate.

- The primary sludge concentration must be maintained at less than 0.5% solids for effective grit removal. This has a negative impact on solids handling processes.

In 2006 the Torrington WPCF piloted and purchased a Conanda Grit Washing Plant system, manufactured by Huber to replace the old/aged grit removal equipment.

Based on the new condition of the grit removal system and relatively smooth operation, it was determined that no additional grit removal improvements would be considered as part of this upgrade.

5.5 PRIMARY TREATMENT SYSTEM

The WPCF operations staff has identified several concerns with the existing three rectangular primary settling tanks (PST-1, 2 and 3) including:

- unbalanced influent distribution,
- insufficient hydraulic capacity under various operational scenarios, and
- failures with the sludge cross collectors.

5.5.1 Influent Distribution

Flow from the Screenings Building enters the primary settling tank influent distribution box vertically through the bottom of the structure. No baffling is provided inside the distribution box which provides uneven hydraulic conditions, distributing more flow to PST No.1. As a result of a higher wastewater flow rate through PST No. 1, the unbalanced influent distribution also results in greater sludge and grease accumulation in PST No.1.

From the distribution box, flow is conveyed to the each primary settling tank through a 24-inch influent pipe. The influent flow to an individual settling tank is then split to each bay. Diffuser boxes are located at the head of the primary settling tank bays. Typically inlet structures, such as the diffuser boxes, are utilized to dissipate the inlet velocities and evenly distribute the influent flow and solids. However, the diffuser boxes in the primary settling tanks at the Torrington facility have an opposite effect, since they are located directly above the sludge hoppers and have an open bottom and a closed top. The open area of each diffuser box, at an average flow of 7.0 mgd, results in a downward velocity of approximately 24 fpm. The downward influent flow velocity towards the sludge hopper likely causes some re-suspension of settled sludge. Also, the downward distribution of the influent flow could result in hydraulic short-circuiting in the tanks, reducing the efficiency of solids settling.

5.5.2 Hydraulic Capacity

The original design criteria provided for surface overflow rates of 1,010 gpd/sq. ft. at average flow (7.0 mgd) and 2,890 gpd/ft^2 at peak flow (20.0 mgd) with all three tanks in operation. The side water depth is eight feet.

The current design criteria for the primary settling tanks are on the higher end of a typical range of overflow rates. Typical overflow rates for rectangular primary settling tanks are 800 to 1,200 gpd/sq. ft. at average flow and 2,000 to 3,000 gpd/sq. ft. at peak flow. The original design criteria assumed operation of all tanks. Therefore, no redundancy is provided in the original design criteria for mechanical and/or operational shut-down, particularly during periods of high flows.

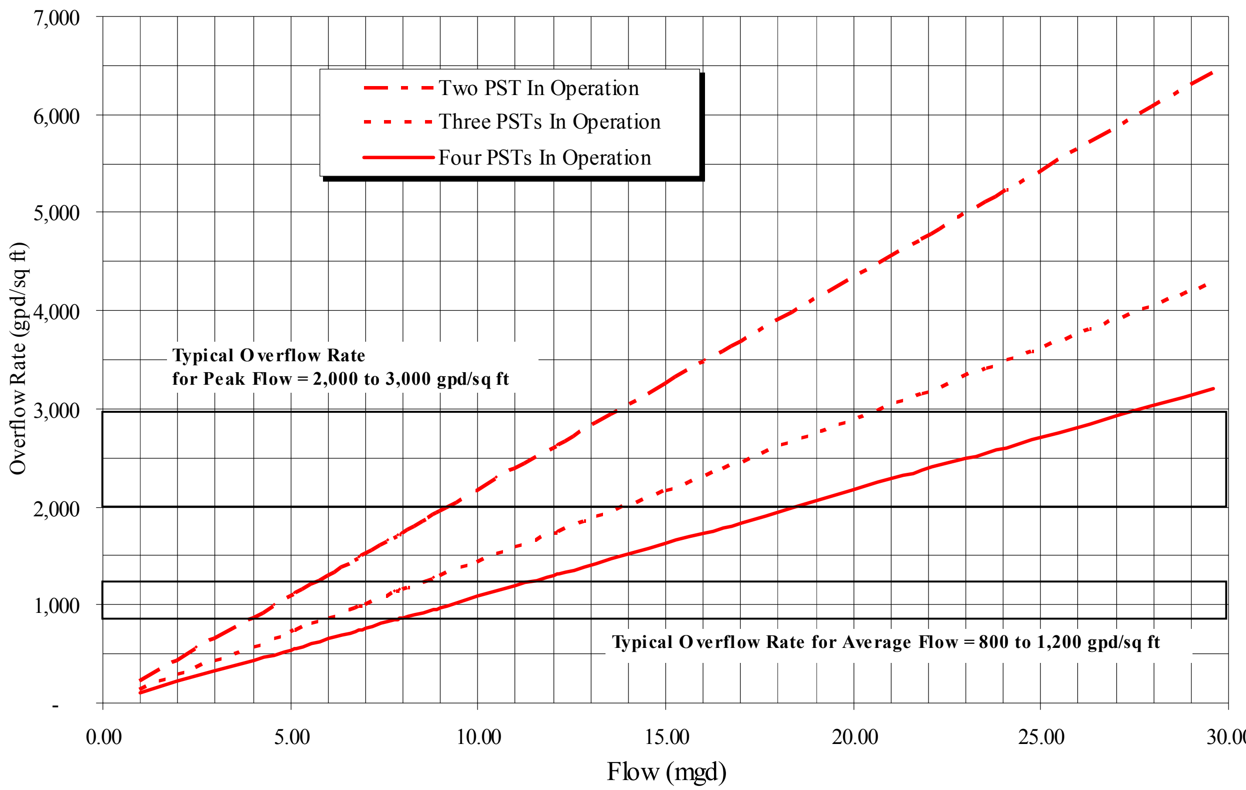
Table 5-4 provides a summary of overflow rates for the primary settling tanks for various flow parameters and number of operating units. This information is also presented graphically in **Figure 5-1**.

TABLE 5-4
CITY OF TORRINGTON WPCF FACILITIES PLAN
PRIMARY SETTLING TANK OVERFLOW RATES
AT VARIOUS FLOW AND OPERATING CONDITIONS

Flow Parameter	Flow (MGD)	Overflow Rates (GPD/SF) per Number of PSTs in Operation		
		2	3	4 (Future)
Current Average Daily Flow	5.5	1194	796	597
Design Year Average Daily Flow	6.31	1369	913	685
Permitted Monthly Average Daily Flow	7.0	1519	1013	760
Design Year Max. Month (98th percentile)	11.59	2514	1676	1257
Design Year Max. Month (100th percentile)	13.31	2888	1925	1444
Current Max Hourly Flow 98th percentile	15.5	3364	2242	1682
Design Year Max Hour Flow (98th percentile)	18.76	4072	2715	2036
Current Max Hour Flow (100th percentile)	22.5	4883	3255	2441
Design Year Max Hour Flow (100th percentile)	25.76	5591	3727	2795

These data illustrate that the primary settling tank overflow rate exceeds typical design criteria at the projected future maximum month and peak hour flows with only three (3) tanks in service. The surface area used to develop the overflow rates presented in **Table 5-4** assumes 100% of the tank length and volume is available. However, as discussed above, due to the configuration of the influent diffusers boxes, the effective length of the tanks is likely less due to short-circuiting. In addition, the effective length of the tanks is likely also reduced somewhat by the placement of the effluent trough structure. These perceived losses in effective length likely results in an increase in the overflow rate above those presented in **Table 5-4**.

FIGURE 5-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
PRIMARY SETTLING TANK OVERFLOW RATES FOR RANGE OF FLOWS AT VARIOUS OPERATING CONDITIONS



5.5.3 Sludge Cross Collector

Sludge cross collectors are located at the influent end of each primary settling tank. The cross collectors move the sludge towards the drive end of the tank, where grit pump suction inlets are located. The location of the drives, relative to the screw operation, has resulted in the accumulation of sludge and grit within the mechanical components of the drive system.

This accumulation of sludge and grit requires significant operator attention and repairs. Additionally, the drives have historically pulled out of the concrete wall. It is likely that the drives have pulled from the wall because of the increased torque force required by the mechanism to convey the sludge; which is then compounded by the presence of significant quantities of grit in the primary sludge.

5.5.4 Recommendations for Primary Treatment

The following improvements are recommended to address the deficiencies with the primary selling tanks:

- Install a fourth primary settling tank.
    - The configuration and dimensions of the new tank would replicate the existing tanks.
    - The effluent flow from the fourth primary settling tank would discharge into Distribution Box No. 2, along with the effluent flow from the other three existing primary settling tanks.
- Modify the existing (or construct a new) influent distribution box to improve flow split between the four primary settling tanks.
- Modify primary effluent distribution to ensure balance flow split each of the aeration tanks; relocate RAS discharge to primary effluent distribution box.
- Modify the influent baffles within each existing tank to dissipate inlet velocities and reduce hydraulic short-circuiting.
- Reconfigure the drive location for the primary sludge cross collectors. The cross collector drives should be placed on the wall opposite the grit pump suction so that sludge and grit are pushed away from the drive end of the screw.
- Provide automated scum removal.
- Provide for new primary sludge pumps with VFDs.

SECTION 6

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SECTION 6

EVALUATION OF EXISTING SECONDARY LIQUID PROCESS FACILITIES

6.1 INTRODUCTION

As mentioned in Section 5, each existing liquid treatment process stream and related components were evaluated with regard to a) the current condition of the existing component/process; and b) for the ability of each component/process to meet both current and known future regulatory limits at projected flows and loads for the planning period. This section highlights the improvements needed at the Torrington WPCF in order for the Facility to meet nitrogen related nutrient removal requirements (i.e. TN, ammonia, etc.).

Secondary Treatment alternatives were first screened to identify whether a particular alternative warranted further evaluation. For the biological processes that were selected to be evaluated further, the Secondary Treatment system (which includes the aeration system, activated sludge tanks and secondary clarifiers) was evaluated with the use of a dynamic computer simulation process model (BioWin) to indentify the best method for increased nitrogen removal to meet the Basis of Design. The evaluation included an assessment of the capacity of the existing unit processes to meet current and anticipated permit limits at projected future flows and loadings as well as the ability to reduce nitrogen discharges in accordance with the Nitrogen General Permit. The model was also used to evaluate the potential for biological phosphorus removal.

This section also evaluates the Disinfection process with regard to the existing and future Basis of Design.

6.2 TREATMENT PERFORMANCE WITH REGARD TO TOTAL NITROGEN

The WPCF staff is currently operating the newer secondary process train for secondary treatment, consisting of Aeration Tanks No. 3 and No. 4 and Final Settling Tanks No. 4 and No. 5. However, the design intent of the 1991 Facilities Report indicated that both process trains (original and new) would be utilized for treating the average and peak hourly design flows of 7.0 mgd and 20 mgd respectively.

The existing secondary system is currently performing very well with respect to BOD_5, TSS and ammonia removals. The existing secondary system does operate either with, or without, full nitrification on a seasonal basis as allowed by the current effluent permit limits. The existing process is operated such that it removes a portion of the influent total nitrogen, reducing the amount of credits to be purchased.

Aeration Tanks No's 3 and 4 each include an internal recycle pump to return nitrified mixed liquor (nitrate) back to the front of the aeration tanks to achieve denitrification (i.e. loosely identified as a Modified Ludzack Ettinger Process). The initial portion of each aeration tank does not include baffle walls and until recently, was mixed using the fine bubble aeration system at reduced air flow rates. Therefore, ideal anoxic conditions were not achieved. The existing process has been shown to produce an effluent total nitrogen concentration in the 5.0 to 8.0 mg/l range with a total nitrogen (TN) discharge of approximately 280 lbs./day (average flow rate of approximately 5.0 MGD). Typically, the facility has been able to achieve lower TN levels during the warmer months.

It should be noted, the Torrington WPCF staff recently completed some intermediate improvements to the treatment process; including the installation of mixers within the anoxic zone. The effect of utilizing mixers within the first anoxic zone is currently being evaluated through supplemental sampling. All data reviewed and utilized in the evaluation summarized in this Section does not include data currently being collected after the recently installed mixers within the anoxic zones.

A review of the process data indicates that historically the facility does experience periods of low influent and effluent pH (in the 6.1 to 6.4 range). At this level, the activity of the nitrifying bacteria will be reduced. Historical data also indicates that during periods of low effluent pH during colder months, the facility has not achieved full nitrification (although not required to by permit) even with sludge retention times that would typically be associated with full nitrification. However, the facility has achieved full nitrification at the lower pH values during the warmer months (presumably due to increased nitrification rates corresponding with the warmer temperatures). The facility may be required to achieve complete nitrification during the winter season in order to meet the annual total nitrogen removal goal. This will require a supplemental alkalinity source to increase the pH during low pH conditions.

As discussed in Section 2 of this report, to achieve the permit limits included in the General Permit for Nitrogen Discharges (Nitrogen General Permit), the Torrington WPCF would need to reduce the annual nitrogen discharge to 248 lbs/day by 2014. Interim compliance limits were also established for each of the years 2002 through 2010, inclusive. The compliance limits are based on a 12-month average, over the calendar year. The total nitrogen limits for the Torrington WPCF, as required by the Nitrogen General Permit, are illustrated in **Table 6-1 and Figure 6-1**.

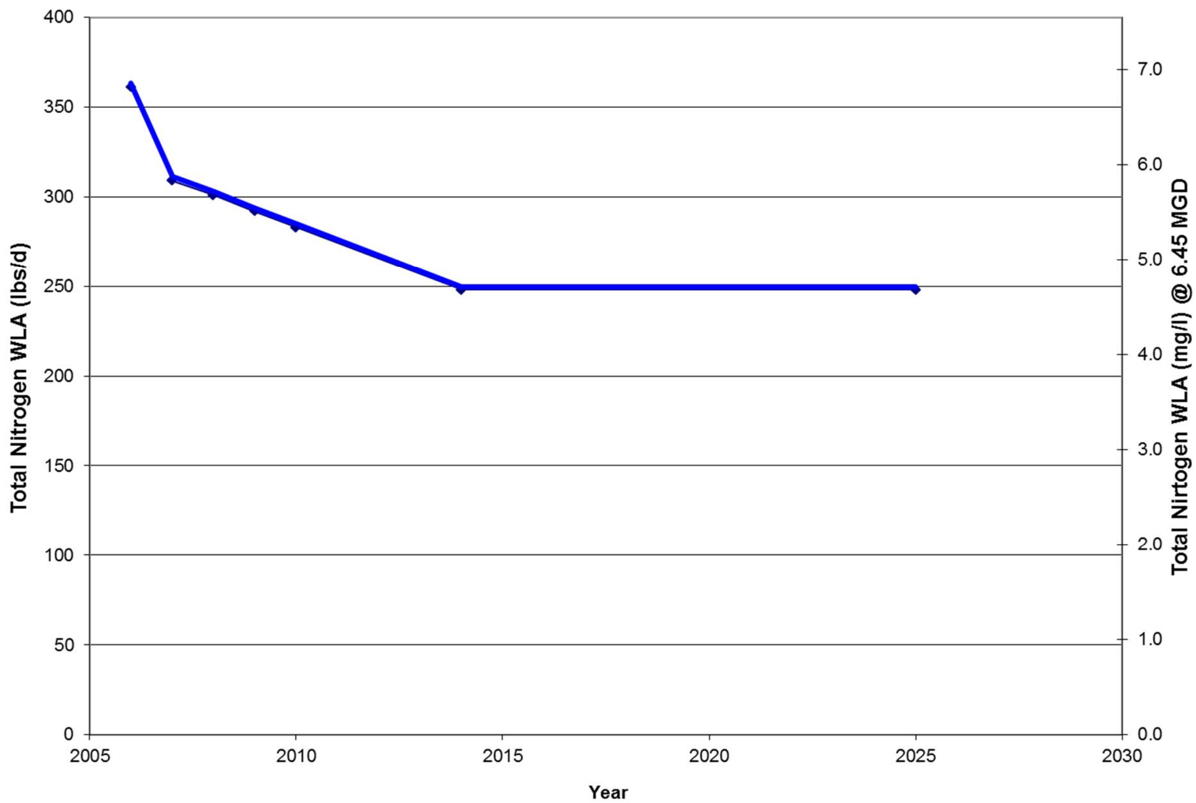
TABLE 6-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
DESIGN YEAR TOTAL NITROGEN DISCHARGE LIMITS

Year	2009	2010	2011	2012	2013	2014
Total Nitrogen (lbs/day)	292	383	373	260	254	248
Total Nitrogen (mg/l)*	5.5	5.4	5.2	4.9	4.8	4.7

*Based on the design average daily flow of 6.31 MGD

The Nitrogen General Permit defines a waste load allocation (WLA) for the Torrington WPCF which represents the amount of nitrogen discharged on a mass (pounds) basis for the entire year. **Figure 6-1** also identifies the resulting effluent nitrogen concentration based on an average daily flow of 6.31 mgd (design year average daily flow rate). The 2014 effluent TN limit of 248 lbs/d is equivalent to a TN concentration of approximately 4.7 mg/l at 6.31 mgd.

FIGURE 6-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
TOTAL NITROGEN DISCHARGE LIMITS



6.3 DEVELOPMENT OF SECONDARY TREATMENT ALTERNATIVES

Biological nutrient removal processes can be generally grouped into two categories; substrate level denitrification and endogenous level denitrification processes. A substrate level denitrification process can achieve effluent total nitrogen levels in the 5.0 to 8.0 mg/l range; and if a process is well operated, lower nitrogen level concentrations can be achieved. When combined with endogenous level denitrification, effluent total nitrogen levels of 3.0 to 3.5 mg/l are possible, depending on residual non-biodegradable nitrogen fractions. Typically, endogenous level denitrification only processes are not cost effective for the treatment of nitrogen. If effluent

TN levels less than 3.0 mg/l are required then more advanced non-biological processes may be required. The cost for treatment increases significantly from one group to the next.

It is believed that the level of treatment required at the Torrington facility to meet the WLA without the purchase of credits can be achieved with a well operated substrate level denitrification process.

6.3.1 Modified Ludzack-Ettinger (MLE) Process

The MLE process is a single stage system, consisting of an anoxic zone for denitrification, and an aerobic zone for carbon oxidation and nitrification. The denitrification anoxic zone is located at the influent end and typically occupies 30% of the total tank volume. Nitrification occurs in the aerobic portion of the tank, downstream of the anoxic section. A nitrate recycle system is used to return the high nitrate mixed liquor from the aerobic zone to the anoxic zone.

A limitation of the MLE process is that the aeration tank effluent contains some portion of nitrate that is not recycled back from the aeration zone to the anoxic zone, and therefore cannot be removed. This process can generally achieve an effluent total nitrogen concentration of approximately 5.0 to 6.0 mg/l. It can achieve lower effluent total nitrogen levels depending on the site specific conditions (high influent carbon to nitrogen ratio, low influent nitrogen concentration, etc.).

Typically the limiting factor is the amount of carbon present in the first stage anoxic zone and the lack of a secondary anoxic zone to remove the nitrates not recycled back to the initial zone (typically ¾ of the MLSS is recycled while ¼ continues on to the secondary clarifiers). Increasing the internal recycle will reduce the nitrate level until all the carbon is used up. Once this occurs, recycling additional MLSS back upstream will not increase the performance of the process.

As shown above in **Figure 6-1**, achieving full compliance with the general permit (i.e. no credit purchase), will require a treatment process that can achieve an effluent TN concentration of less than 4.7 mg/l. This level is just below the MLE process's normal treatment level but can be accomplished if the conditions are right. Therefore, the MLE process was retained for modeling and further evaluation.

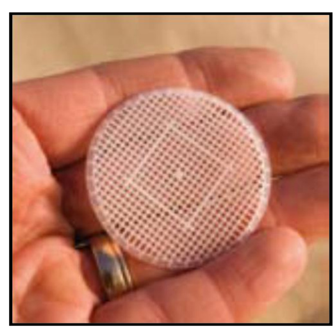
6.3.2 Four-Stage Bardenpho Process

The Four-Stage Bardenpho process is a single-sludge, four-stage process for nitrogen removal consisting of an initial anoxic zone, an aeration-nitrification zone, followed by a second anoxic zone and subsequent re-aeration zone. The initial anoxic zone and aeration-nitrification zone are essentially the same size as the zones developed for the MLE process. The secondary anoxic zone utilizes carbon released from endogenous decay as opposed to the influent carbon used in the initial anoxic zone. The denitrification rates within the second anoxic zone are much slower than the first zone and thus this anoxic zone is typically greater in volume than the first anoxic zone (unless a supplemental carbon source is added).

The four-stage process has demonstrated reliable performance and is capable of achieving total nitrogen limits lower than the MLE process. Typically, this process can achieve TN levels in 3.0 to 4.0 mg/l range. Given the fact that this process requires a secondary anoxic and post-aeration zone, the total volume and amount of equipment required will be greater than the MLE process. Thus, while it can achieve a higher level of nitrogen removal, the capital and operational cost of this alternative will be greater than the MLE process. The Four-Stage Bardenpho process was retained for modeling and further evaluation.

6.3.3 Integrated Fixed Film Activated Sludge (IFAS) Process

In an integrated fixed film activated sludge process (IFAS), media is added to the aeration tanks to provide sites for the fixed growth of bacteria. This allows for an increase in the mass of mixed liquor in the aeration basins with a corresponding increase in solids retention time. However, the suspended concentration of mixed liquor solids can remain the same or be even lower than without the media. Therefore, the solids loading rate on the secondary clarifiers may not



Sample IFAS Media

increase even though additional mixed liquor solids are provided.

The IFAS, through the increased density of bacteria within the aeration tanks, can provide more treatment per tank volume than the MLE or four-stage Bardenpho process. The media provided typically consists of free-floating plastic or a webbed material mounted to a fixed frame in the aeration tanks. The IFAS process was retained for modeling and further evaluation.

6.3.4 Denitrification Filter System

Another alternative considered for the Torrington Facility was the use of a denitrification filter as a tertiary step to remove any remaining nitrogen not removed in the existing biological system to the 2014 permitted limit. For this evaluation, a BIOSTYR denitrification filter system manufactured by Kruger was considered; this type of system is a biological aerated filter (BAF) that combines biological treatment and upflow filtration. An advantage to this style system is the compact footprint required. A disadvantage to this system is the capital investment required to construct the entire BAF process (tankage, equipment, etc.) and the storage system for supplemental carbon as well as the annual operational costs associated with supplemental carbon addition.

Typical BAF System

Because the capital investment for a denitrification filter was anticipated to be extreme, a cost comparison of the alternative-specific costs associated with the filter alternative and the Four-Stage Bardenpho process was performed to determine if the filter alternative was viable for future consideration at the Torrington WPCF. As can be seen in **Table 6-2**, the alternative specific costs required to construct a denitrification filter is approximately double that required to construct the necessary changes to the existing process to implement a Four-Stage Bardenpho process. Thus, the alternative of constructing a denitrification filter <u>was not</u> retained for further evaluation.

TABLE 6-2
CITY OF TORRINGTON WPCF FACILITIES PLAN
COST COMPARISON OF ALTERNATIVE-SPECIFIC SECONDARY TREATMENT

	Treatment Alternative	
	Four-Stage Bardenpho	Denitrification Filter
Subtotal Construction	$ 4,309,900	$ 9,463,800
Contractor's General Conditions / OH&P	$ 646,000	$ 1,420,000
Design Contingency (15%)	$ 743,000	$ 1,633,000
Construction Costs	$ 5,698,900	$ 12,516,800
Construction Contingency (5%)	284,945	625,840
Alternative Specific Construction Costs	**$ 5,983,800**	**$ 13,142,600**

6.4 DEVELOPMENT OF A PROCESS MODEL FOR EVALUATING SELECTED SECONDARY TREATMENT ALTERNATIVES

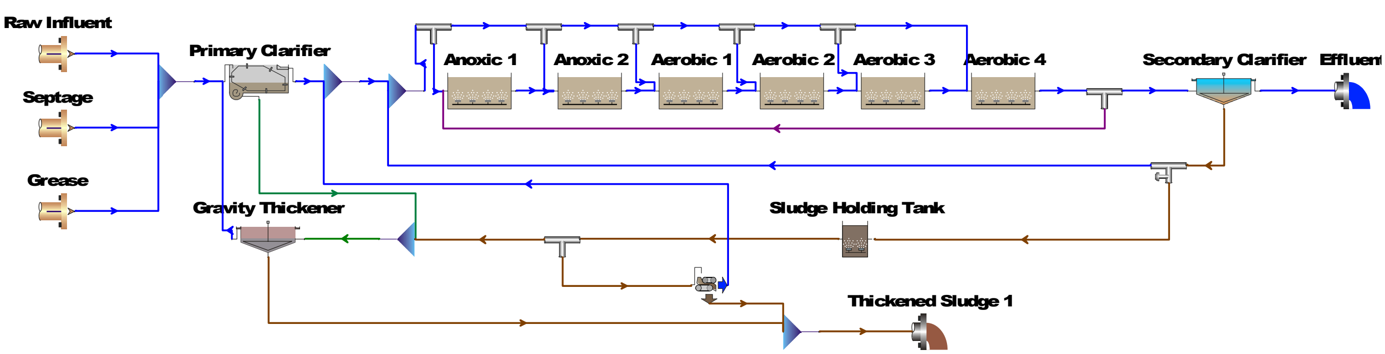
The BioWin simulator was used for process modeling of the activated sludge system to determine the process loading capacity, operating characteristics and ultimately the nutrient removal capacity of the secondary treatment process. BioWin was developed by EnviroSim Associates Limited and is based upon the IAWPRC 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.

Simulation of biological wastewater treatment performance requires the model to be set up in the following manner. A process flow schematic is created inside the model and the physical characteristics of the system components such as aeration tank volumes, settling dimensions, etc. are defined and associated with the model components (see **Figure 6-2**). Once the physical characteristics of the facility are defined, plant operating conditions are then applied to the model. The process of defining the operating conditions includes the following steps:

1. specifying flow rates for the plant influent, return activated sludge, and waste sludge;
2. specifying the dissolved oxygen concentration in the aerated zones; and
3. defining the composition of the influent streams

Note that for modeling purposes, only the newer secondary process train (Aeration Tanks No. 3 and No. 4 and Final Settling Tanks No. 4 and No. 5) was assumed to be in operation. The aeration tanks were operated to provide BOD_5 and nitrogen removal. The first two zones in each aeration tank were operated as anoxic zones. However, air leakage from diffusers installed in the first anoxic zone provided a continuous low level of dissolved oxygen. The process configuration used to model the current operation is presented in **Figure 6-2**.

FIGURE 6-2
CITY OF TORRINGTON WPCF FACILITIES PLAN
BIOWIN MODEL - PROCESS FLOW DIAGRAM



In order for the model to accurately predict specific process configuration responses to site specific influent conditions, the model was calibrated based upon existing data and conditions. Once calibrated, additional model runs are completed to verify the model, as discussed below.

Proper definition of the influent waste characteristics is paramount in achieving realistic predictions from the model simulator. To this end, a review of the influent wastewater composition was conducted to establish both the variability and composition of the wastewater. A wastewater characterization study was conducted, as presented in Appendix C, to supplement the historical data to aid in the model development.

6.4.1 Model Calibration

Once a model is created, it is calibrated utilizing actual monthly average plant data. The June and July 2001 data was selected for the initial model calibration performed in a previous version to this plan developed in 2005. Because the WPCF process is currently configured the same way it was in 2005, the previous Biowin Model was deemed acceptable for use in this evaluation. Utilizing the actual plant data, kinetic and stoichiometric parameters in the model were adjusted until a reasonable fit was achieved between the model output and the actual plant data. The kinetic and stoichiometric parameters determined during the calibration phase then became the basis for all subsequent computer simulations for all of the process configuration alternatives.

Table 6-3 shows the calibration model kinetic and stoichiometric parameter inputs for the June-July 2001 data in comparison with the BioWin defaults. All other parameters not identified in Table 6-2 were maintained at the BioWin default values.

TABLE 6-3
CITY OF TORRINGTON WPCF FACILITIES PLAN
MODIFIED CALIBRATION MODEL PARAMETERS

	BioWin Default Values	**Torrington Calibration Model Value**
Kinetic		
Heterotrophic Aerobic Decay Rate (Arrhenius value)	1.029	1.1
Heterotrophic Anoxic/Anaerobic Decay Rate (Arrhenius value)	1.029	1.1
Stoichiometric		
Heterotrophic Yield (aerobic)	0.666	0.68
Particulate Substrate COD/VSS	1.6	1.9
Particulate Inert COD/VSS	1.6	1.9

6.4.2 Model Verification

The second step to confirming the applicability of the model, is to verify the calibrated model. The model is verified by modeling a separate influent condition and comparing the result to the actual data. The February through March 2002 data was utilized in the model to verify the calibration. During the verification stage, the kinetic and stoichiometric parameters are not altered. The results of the verification model were then compared to the historical data. Because the model accurately predicted the operating conditions, effluent concentrations and characteristics of the facility during the verification period, it was assumed to be applicable for further use.

Results from both the calibration and verification modeling scenarios are presented in **Table 6-4**.

TABLE 6-4
CITY OF TORRINGTON WPCF FACILITIES PLAN
BIOWIN MODEL CALIBRATION RESULTS

	Calibration Phase		Verification Phase 2001	
	Actual Data	**BioWin Results**	**Actual Data**	**BioWin Results**
Influent (Raw)				
Flow Rate, mgd	5.01	5.01	4.59	4.59
BOD_5, mg/l	134	134	133	133
TSS, mg/l	140	140	118	118
TKN, mg/l	No Data	20	No Data	23
NH_3-N, mg/l	10.3	11	12.5	12.5
Temp, C	18	18	12	12
Primary Effluent				
BOD_5, mg/l	No Data	105	No Data	110
TSS, mg/l	No Data	93	No Data	76
Sludge, lbs/day	2,805	2,817	2,722	2,100
Aeration Tanks				
Volume, mgal	1.53	1.53	1.53	1.53
MLSS, mg/l	3,515	3,416	3,767	3,747
Sludge Residence Time, days	9.06	8.99	11.1	10.6
Waste Sludge, lbs/day	3,133	3,005	2,718	2,813
Plant Effluent				
BOD_5, mg/l	5.0	4.0	5.0	4.0
TSS, mg/l	4.0	5.0	4.5	5.0
TKN, mg/l	1.85	1.55	2.1	1.9
NH_3-N, mg/l	0.5	0.5	1.0	1.0

6.4.3 Modeling Conditions

Process modeling was conducted to determine the secondary treatment system's nutrient removal capacity based on the following conditions:

Design Year Annual Average Loading Conditions:

As previously stated, the Nitrogen General Permit limits the annual amount (pounds) of total nitrogen that can be discharged from the Torrington WPCF. For this analysis, the annual average loading condition was utilized to determine the average expected effluent total nitrogen concentration. Process modeling was conducted to achieve an effluent total nitrogen concentration less than 4.6 mg/l under average flow and loading conditions. Once the effluent TN is determined, it is expected that over the course of the year the average effluent condition would match the predicted model output. To account for real world dynamic conditions (i.e. non-linear response to variable influent loading conditions) in a steady state model run, the annual average loading condition was run at the minimum temperature (it is assumed that the process will perform better than predicted under warmer influent conditions).

Design Year Maximum Month Loading Conditions:
The Torrington facility is subject to effluent ammonia nitrogen limits. These limits are seasonal (effluent ammonia limit from April through October) and vary monthly (monthly average limit of 12.7 to 1.7 mg/l, depending on the actual month). The design year maximum month loading condition was utilized to verify that the proposed process configuration could achieve compliance with the seasonal ammonia nitrogen limit and to determine the 30-day peak sustained total nitrogen effluent concentration under design conditions. A high month total nitrogen effluent concentration can increase the overall annual average and cause the WPCF to surpass the effluent nitrogen discharge limit. Process modeling was conducted utilizing a minimum temperature (it is assumed that the process will perform better than predicted under warmer influent conditions).

The influent loading for both conditions outlined above are represented in Section 3, Table 3-2 for the current design year. For all modeling scenarios, it was assumed that FOG and septage would be discharged upstream of the primary clarifiers. The loading from the solids handling recycle was applied to both average and maximum month conditions. The resulting combined load to the primary clarifiers is presented in **Table 6-5**.

TABLE 6-5
CITY OF TORRINGTON WPCF FACILITIES PLAN
COMBINED DESIGH YEAR INFLUENT LOADING CONDITIONS

	Annual Average Loading Condition	**Maximum Month Loading Condition**
Flow (MGD)	6.34[1]	9.97[1, 2]
BOD		
(mg/l)	148	132
(lbs/day)	7,825	10,976
TSS		
(mg/l)	152	140
(lbs/day)	8,053	11,615
TKN		
(mg/l)	26	29
(lbs/day)	1,382	2,411
NH_3-N		
(mg/l)	14	16
(lbs/day)	748	1,307
Total Phosphorus		
(mg/l)	3.3	3.2
(lbs/day)	173	267
Temperature (C)	10	10

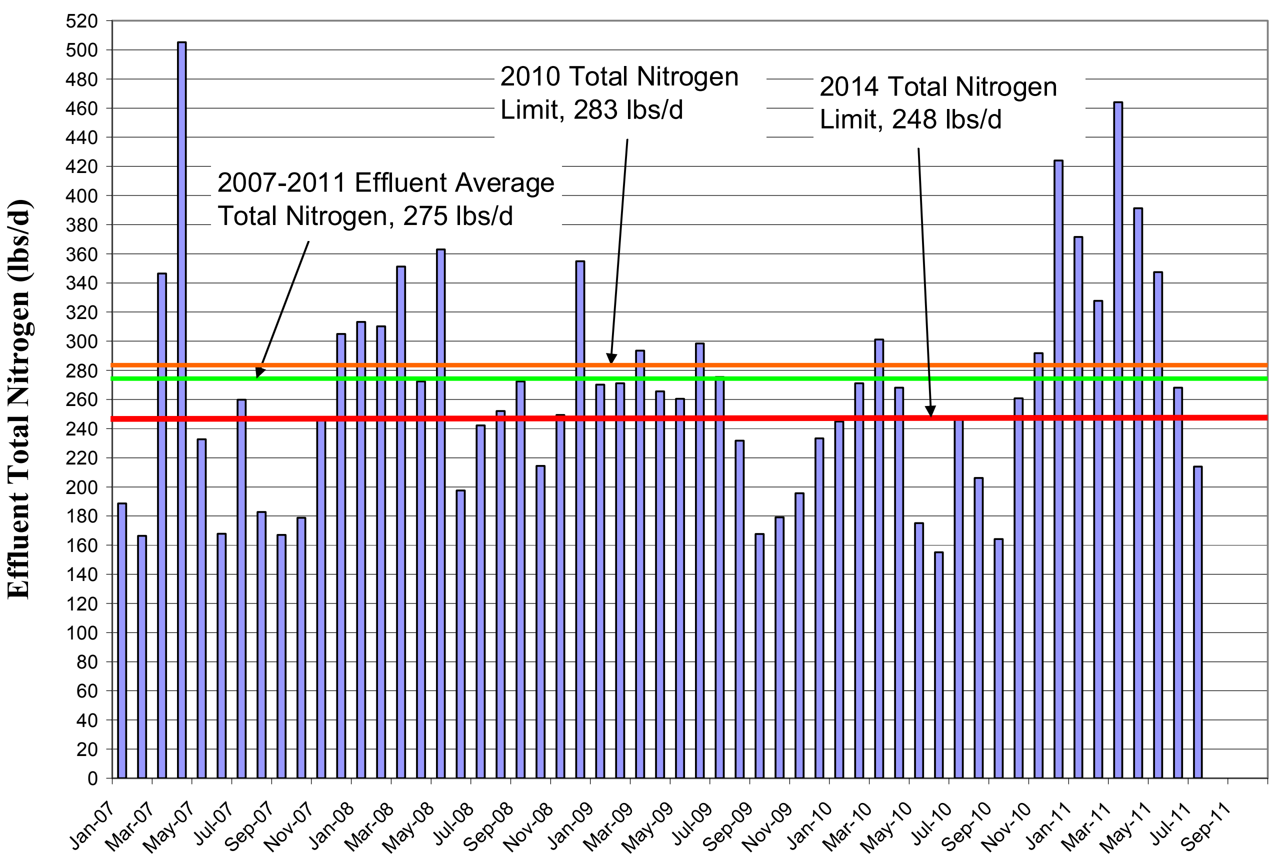
Notes:
1. Includes solid processing recycle loads
2. Flow corresponding to month that maximum organic loading occurred

6.5 EVALUATION OF SECONDARY TREATMENT ALTERNATIVES

The Torrington WPCF is currently achieving nitrogen removal within the two newer aeration tanks (Aeration Tanks No's 3 and 4). The WPCF does not currently use Aeration Tanks No. 1 and 2 for treatment.

The operating data collected from January 2007 - July 2011 shows the average effluent total nitrogen concentration was 6 mg/l at a flow rate of 5.5 mgd (275 lbs/d). This monthly average total nitrogen effluent load discharged from the Torrington facility during the January 2007 - July 2011 analysis period is shown graphically in **Figure 6-3**. Both the 2010 and 2014 Total Nitrogen Limits of 283 lbs/d and 248 lbs/day respectively are also presented. As indicated, the current operation meets the 2010 effluent nitrogen limit but does not meet the future 2014 effluent nitrogen limit.

FIGURE 6-3
CITY OF TORRINGTON WPCF FACILITIES PLAN
EFFLUENT TOTAL NITROGEN MONTHLY AVERAGE CONCENTRATION
(2007-2011)



There are interim improvements that can be done to the existing process in order to improve the treatment efficiency. These improvements include:

- using mechanical mixers instead of air in the anoxic zone (Recently implemented by WPCF Staff)
- increasing the internal recycle pump capacity to provide a better environment for denitrification and potentially decreasing the overall total nitrogen effluent concentration

6.5.1 Modified Ludzack Ettinger Process Alternative

The Torrington facility consists of four aeration tanks and four final settling tanks (only two aeration and two final settling tanks are in service). This process alternative for the design year conditions will require upgrading all of the existing aeration tanks to include a larger internal recycle system (an internal recycle system is currently installed in Aeration Tank Nos. 3 and 4, but does not have sufficient capacity for the required flow rate), installation of partition walls to segregate the anoxic zones and installation of a permanent non-aerated mixing system within the anoxic zone.

For evaluating the MLE process for the design year conditions, all influent loads (grease and septage included) would be discharged upstream of the primary clarifiers. Also, suspended solids and BOD_5 removal through the primary clarifiers were set in the model at the historical values of 50% (TSS) and 30% (BOD_5).

Design Year Annual Average Loading Conditions:

Under the design annual average flow rate entering the aeration tanks of 6.49 mgd (includes recycle flows), process modeling has indicated that the MLE process can achieve an effluent total nitrogen concentration of 5.0 mg/l (270 lbs/d) at a wastewater temperature of 10 degrees C, assuming all four aeration tanks are online. The model also predicted that approximately 20 gallons per day of supplemental carbon is needed to achieve this level of nitrogen removal. In addition, some alkalinity addition may be needed and will be confirmed during preliminary design efforts.

Design Year Maximum Month Loading Conditions:

Under the design max month loading condition, process modeling indicates that the MLE process can achieve an effluent total nitrogen concentration of 13.5 mg/l (1,157 lbs/d) at a wastewater temperature of 10 degrees C. Process modeling indicated that in order to achieve an effluent ammonia-nitrogen concentration of less than 1.0 mg/l at a wastewater temperature of 10 degrees C, approximately 6 hours of hydraulic detention time (HRT) within the oxic zone is required to achieve complete nitrification. The required 6 hours of

HRT was equivalent to approximately 2.48 million gallons of tankage, while a total volume of approximately 2.58 million gallons is available. Therefore, only 100,000 gallons of anoxic zone is available for denitrification, resulting in a much higher effluent nitrate concentration and overall total nitrogen concentration. The modeling also predicted that 20 gallons a day of supplemental carbon is needed for this flow condition, and that some alkalinity addition may be needed, especially at system start-up.

Warm Weather versus Cold Weather Operation:
The Torrington facility is not required to maintain nitrification during the winter months. However, for total nitrogen removal year-round nitrification is required. Process modeling indicated that during cold temperatures, maintaining complete nitrification may be difficult. The maximum sludge retention time achievable, while not overloading the secondary clarifiers, may result in partial nitrification during the coldest months of the year (i.e. wastewater temperature below 12 degrees C). To provide maximum treatment flexibility, it is recommended that the initial anoxic zone be subdivided into smaller zones. One of the smaller partitioned anoxic zones can be fitted with aeration, creating a "swing zone" for the ability to provide an increase in aerobic SRT during the winter months. This "swing zone" could also be aerated during the maximum loading conditions to ensure complete nitrification for permit compliance during the warmer months.

Process modeling results for the MLE process for the design year condition is presented in **Table 6-6**.

TABLE 6-6
CITY OF TORRINGTON WPCF FACILITIES PLAN
BASIS OF DESIGN: MLE PROCESS

	Design Year Annual Average Loading	Design Year Max Month Loading
Influent (Raw)		
Flow Rate, mgd	6.34[1]	9.97[1, 2]
Peak Day Flow Rate, mgd	18.8[3]	18.8[3]
Temperature, C	10	10
Anoxic Zone		
Volume, mgal	0.86	0.23
Hydraulic Residence Time, hrs	3.26	0.55
Sludge Residence Time, days	4.80	0.93
Volume of Methanol, gpd	20	20
Aerobic Zone		
Volume, mgal	1.82	2.45
MLSS, mg/l	3,227	3,243
F:M	0.11	0.11
Hydraulic Residence Time, hrs	6.89	5.90
Sludge Residence Time, days	10.60	10.24
Oxygen Demand, lbs/day	8,054	13,952
Internal Recycle Rate, mgd	18.90	24.83
System SRT, days	15.4	11.17
Final Effluent TN, mg/l (lbs/d)	5.0 (264)	14.17 (1,179)
Permitted Effluent TN, lbs/d	248	
Final Phosphorus TP, mg/l (lbs/d)	0.72 (38.1)	0.92 (76.5)
Permitted Phosphorus TP, lbs/d	17.29	
Primary Sludge, lbs/day	4,417	5,960
Waste Activated Sludge, lbs/day	4,453	6,029

Notes:
1. Includes solids processing recycle flows
2. Flow corresponding to month that maximum organic loading occurred
3. Based on the 98th percentile flows

The existing aeration system, as described in Section 4, has sufficient capacity to handle the average and peak day air demands. There are a total of four blowers, each with a capacity of 3,700 scfm. It is anticipated that at maximum day conditions, approximately 6,000 scfm will be required. Therefore, the existing blowers have sufficient capacity with one unit out of service to meet this demand.

However, upgrading the existing four aeration tanks to the MLE process would not allow the Torrington facility to meet the 2014 total nitrogen waste load allocation (WLA) under design year conditions without the need to purchase credits. If there are periods where nitrification cannot be maintained during cold-weather periods, it would increase the amount of credits needed.

Note that the MLE process will not sufficiently reduce phosphorus to the anticipated permit limits through the biological process. Alternatives for phosphorus reduction are discussed in Section 7.

6.5.2 Four-Stage Bardenpho Process Alternative

As previously discussed, the Four-Stage Bardenpho process is an extension of the MLE process; the Four-Stage Bardenpho process includes a secondary anoxic zone (post-denitrification) and a re-aeration zone. The advantage of the Four-Stage Bardenpho process (or any process incorporating a secondary anoxic zone) is its ability to achieve lower effluent Total Nitrogen values. The Bardenpho process alternative requires a greater aeration tank volume for full treatment than the MLE process. The Four-Stage Bardenpho process is typically more expensive to construct, due to the capital cost associated with the additional tankage, and operate due, to additional mixing requirements in the secondary anoxic zone than the MLE process, but achieves better effluent quality.

Process modeling was conducted to determine what additional aeration tank volume (above that needed for the MLE process) would be required to incorporate the Four-Stage Bardenpho process. Two alternative treatment scenarios where analyzed:

- **Scenario No.1**: Process modeling was conducted on the assumptions that the Four-Stage Bardenpho process would achieve an effluent total nitrogen concentration of 3.5 mg/l during the average loading conditions (cold weather). However, under the maximum month loading conditions, nitrogen removal performance would be sacrificed. The combined result would be a process that would produce an average effluent load of 248 lbs/day over the entire year.

- **Scenario 2**: Process modeling was conducted on the assumptions that the Four-Stage Bardenpho process would achieve an effluent total nitrogen concentration of 3.5 mg/l during the average loading conditions (cold weather). However, under the maximum month loading conditions, nitrogen removal performance would not be sacrificed. Instead, the Four-Stage Bardenpho process would be designed to achieve an effluent total nitrogen concentration less than 3.5 mg/l at the maximum month influent load condition. The combined result would be a process that would produce a maximum month effluent concentration of 3.5 mg/l. Currently, the Torrington WPCF does not have a monthly TN limit, so this scenario would be considered the complete build-out of this process due to a change in the Nitrogen General permit which would require meeting a monthly TN limit without the ability to purchase credits.

Process modeling, **for Scenario No.1**, indicates that the existing aeration tank volume of approximately 2.58 million gallons is sufficient to achieve an effluent total nitrogen concentration less than 3.5 mg/l at the annual average loading condition; the results are presented in **Table 6-7**. However, during the maximum month loading condition, the Four-Stage Bardenpho process would be overloaded under future flow and load conditions and subsequently is at risk for ammonia violations. To address nitrification concerns, the proposed 4-Stage Bardenpho process will need to have the ability to aerate the second anoxic zone (effectively turning the 4-Stage Bardenpho process into an MLE process). While the total nitrogen level will be significantly higher during this month, compliance with the monthly ammonia limit and annual average TN limit is still achievable.

TABLE 6-7
CITY OF TORRINGTON WPCF FACILITIES PLAN
BASIS OF DESIGN: FOUR-STAGE BARDENPHO PROCESS-IN EXISTING TANKS

	Design Year Annual Average Loading	Design Year Max Month Loading
Influent (Raw)		
Flow Rate, mgd	6.34[1]	9.97[1, 3]
Peak Hour Flow Rate, mgd[2]	18.80	18.80
Temperature, C	10	10
1st Anoxic Zone		
Volume, mgal	0.86	0.43
Hydraulic Residence Time, hrs	3.27	1.0
Volume of Methanol, gpd	0	50
Aerobic Zone		
Volume, mgal	1.39	2.16
MLSS, mg/l	4,069	4,010
Hydraulic Residence Time, hrs	5.26	5.20
Sludge Residence Time, days	11.0	10.8
Oxygen Demand, lbs/day	7,951	13,440
Internal Recycle Rate, mgd	19.0	29.8
2nd Anoxic Zone		(operated aerobically)
Volume, mgal	0.30	0
Hydraulic Residence Time, hrs	1.14	N/A
Volume of Methanol, gpd	200	0
System SRT, days	19.76	12.86
Final Effluent TN, mg/l (lbs/d)	3.5 (189.4)	8.57 (709)
Permitted Effluent TN, lbs/d	248	
Final Phosphorus TP, mg/l (lbs/d)	0.72 (38.1)	1.0 (83.1)
Permitted Phosphorus TP, lbs/d	17.29	
Primary Sludge, lbs/day	4,037	5, 967
Waste Activated Sludge, lbs/day	4,052	6,033

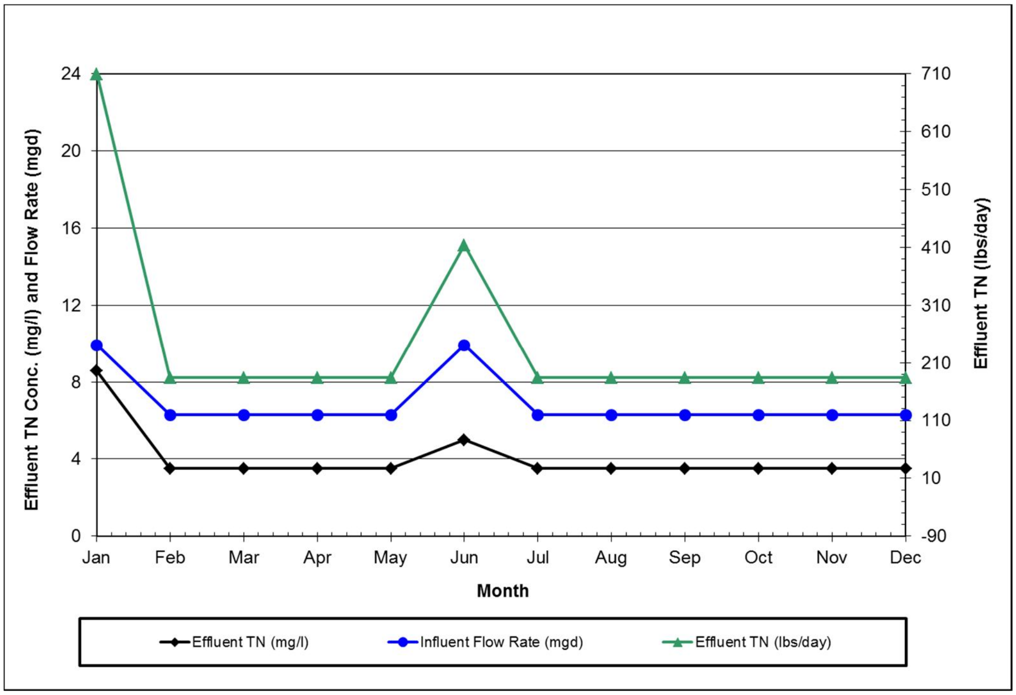
Notes:
1. Includes solids processing recycle flows
2. Flow corresponding to month that maximum organic loading occurred
3. Based on the 98th percentile flows

The proposed improvements include the ability to aerate the secondary anoxic zones. This feature may be required during maximum month low temperature conditions to maintain complete nitrification. Subsequently, an increase in the nitrogen effluent would be expected. The Four-Stage Bardenpho process can and will need to be operated to achieve approximately 3.5 mg/l effluent total nitrogen under all other loading conditions, ultimately complying with the annual nitrogen permit. Process modeling indicates that, at the annual average loading condition, the proposed process can achieve 3.5 mg/l effluent nitrogen regardless of the wastewater temperature. **Figure 6-4** graphically summarizes results of process modeling for the Four-Stage Bardenpho process installed in the existing aeration tanks with three circular secondary clarifiers for a typical 12 month period under future conditions.

The assumptions used in developing **Figure 6-4** include:

- January - Maximum month loading conditions occurred, commensurate with cold water (10° C). The proposed process would need to be operated as an MLE process resulting in an elevated effluent nitrogen concentration (8.5 mg/l).
- February through May – Annual average loading condition, commensurate with cold water (10° C used to give conservative effluent TN value). The proposed process would be operated as a Four-Statge Bardenpho process resulting in an effluent nitrogen concentration of 3.5 mg/l.
- June - Maximum month loading conditions occurred, commensurate with warm water (18° C). The proposed process would need to be operated as an MLE process resulting in an elevated effluent nitrogen concentration (5.0 mg/l).
- July through December - Annual average loading condition, commensurate with cold water (10° C used to give conservative effluent TN value). The proposed process would be operated as a Four-Stage Bardenpho process resulting in an effluent nitrogen concentration of 3.5 mg/l.

FIGURE 6-4
CITY OF TORRINGTON WPCF FACILITIES PLAN
HYPOTHETICAL 12 MONTH PERIOD – EFFLUENT TN QUALITY



Notes:

1. Max Month Conditions (9.93 mgd @ 10,100 lbs BOD_5/day)
2. Average Conditions (6.3 mgd @ 7,000 lbs BOD_5/day)

Process modeling, **for Scenario No.2**, indicates that the existing aeration tank volume of approximately 2.58 million gallons is not sufficient to achieve an effluent total nitrogen concentration less than 3.5 mg/l at the maximum month loading condition. Process modeling indicated that a total volume of 4.16 million gallons would be required for this Scenario. This additional volume of approximately 1.63 million gallons needed for Scenario No. 2 is essentially equal to the volume of Aeration Tanks No's 3 and 4.

The existing aeration system, as described in Section 4, has sufficient capacity to handle the average and peak day air demands. There are a total of four blowers, each with a capacity of 3,700 scfm. It is anticipated that at maximum day conditions, approximately 6,000 scfm will be required. Therefore, the existing blowers have sufficient capacity, with one unit out of service, to meet this demand.

In summary, upgrading the existing four aeration tanks to the Four-Stage Bardenpho process could allow the Torrington facility to meet the 2014 Total Nitrogen waste load allocation limits under design conditions without the need to purchase credits.

6.5.3 Integrated Fixed Film Activated Sludge (IFAS) Process Alternative

Although there is not sufficient volume within the existing infrastructure to construct a conventional Four-Stage Bardenpho process that would achieve an effluent total nitrogen concentration under cold weather max month condition that would meet the 2014 limit; implementing a Four-Stage Bardenpho process within the existing aeration tanks in conjunction with the use of an IFAS process was evaluated as an alternative. A smaller volume could be used for first-stage aerobic zone by incorporating IFAS media in this zone, thus allowing for the Four-Stage Bardenpho process to be implemented within an overall smaller tank volume.

An IFAS media manufacturer (Hydroxyl) was contacted during the preparation of the plan to assist in determining the feasibility of this alternative for use at the Torrington Facility. Preliminary analysis conducted by Hydroxyl indicated that modifying two of the four aeration tanks to operate in a four-stage Bardenpho process and utilizing IFAS media in the first-stage aerobic zone could achieve an effluent TN of less than 248 lbs/d. Therefore, the cost-effective analysis included an analysis of converting Aeration Tanks No's 3 and No. 4 to a Four-Stage Bardenpho process and adding an IFAS system. For purposes of this evaluation, the Hydroxyl system was utilized and included the necessary media, media retention screens and related appurtenances. Note that with an IFAS system such as Hydroxyl, a fine screen must be installed upstream of the aeration tanks to prevent clogging of the media. Thus, evaluation also includes the cost difference for the installation of a fine mechanical screen versus a traditional mechanical screen.

6.5.4 Do-Nothing Alternative

The Nitrogen General Permit currently allows the permittee to either achieve the Total Nitrogen Waste Load Allocation or purchase credits from the CT DEEP via the Nitrogen Credit Trading program to make up the differential between the actual annual Total Nitrogen discharged and the permitted limit. In the "Do-Nothing" option, the Torrington WPCF would continue to operate the facility as it currently is operating and they would achieve 'compliance' through annual, increasing, payments to the CT DEEP.

While the Nitrogen Trading Credit program is currently in place, the program may not be available indefinitely. Furthermore, the costs per credit have and will continue to increase over time. The Torrington Facility is currently anticipated to pay CT DEEP approximately $30,000 for this last year; in previous years Torrington had been getting paid by CT DEEP because they were below their Total Nitrogen.

6.6 EVALUATION OF SECONDARY CLARIFIERS

As stated previously, there are three rectangular clarifiers from the original construction and two 80-foot diameter circular clarifiers that were constructed in the early 1990's. Hydraulically, Aeration Tanks No’s 1 and 2 can only discharge to the three rectangular clarifiers and Aeration Tanks No’s 3 and 4 can only discharge to the two circular clarifiers. The water level in Aeration Tanks No. 1 and 2 is approximately 1-foot lower than the water level in Aeration Tanks No. 3 and 4.

In order to implement any of the three secondary treatment alternatives evaluated above, all volume within the four aeration tanks would need to be utilized. A conceptual flow process would be for all flow leaving the primary settling tanks to enter Aeration Tanks No’s 3 and 4. Flow would then exit from Aeration Tank No’s 3 and 4, and be conveyed to Aeration Tanks No’s 1 and 2. Then, from Aeration Tank No’s 1 and 2, flow would enter the existing circular secondary clarifiers. The following scenarios were analyzed:

- Scenario 1 - two clarifiers online;
- Scenario 2 - three clarifiers online; and
- Scenario 3 - four clarifiers online.

The treatment capacity of secondary clarifiers is evaluated based on the clarifier's surface overflow rate, solids loading rate and the state point analysis (an analysis that takes into account both the overflow and underflow rates, solids loading rate and thickening capacity of the clarifier).

Table 6-8 summarizes the surface overflow rate for each of the secondary clarifier scenarios based on the flow conditions presented. Ideally, a secondary clarifier's maximum surface overflow rate should be below 1,200 gal/ft^2 at peak hour flow. Thus, three secondary clarifiers are required to treat a peak hour flow rate of 16.5 mgd.

TABLE 6-8
CITY OF TORRINGTON WPCF FACILITIES PLAN
SECONDARY CLARIFIER EVALUATION

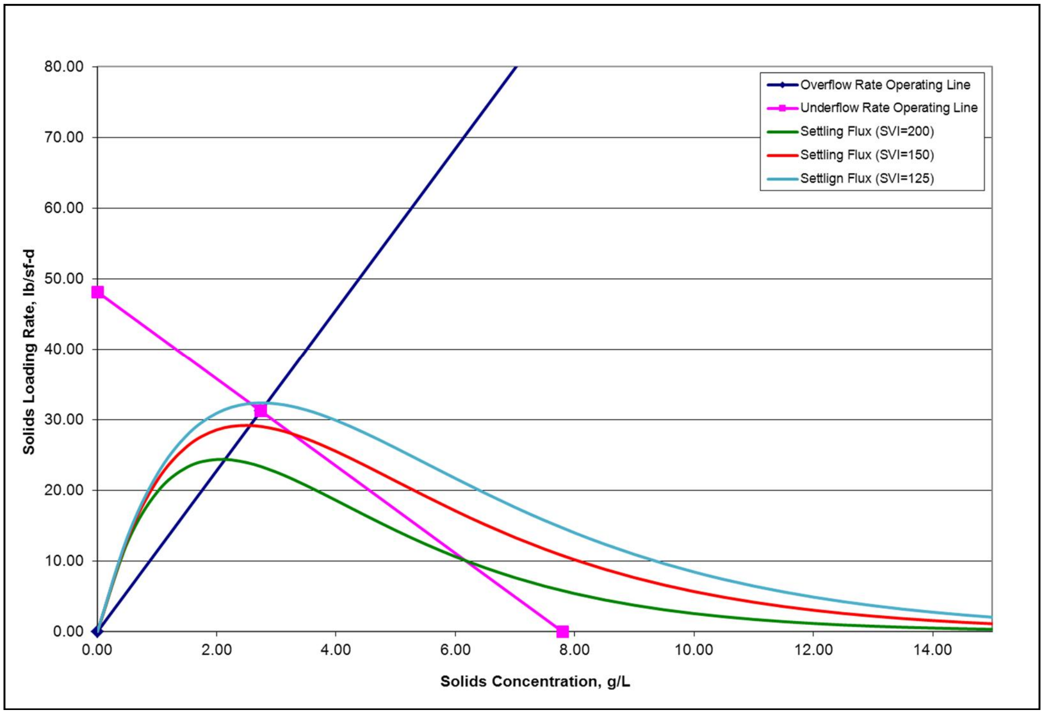
Scenario	Number of 80' Diameter Secondary Clarifiers Operating	Total Surface Area ft^2	SOR gal/ft^2	Flow mgd
1	Two	10,000	1100	11
2	Three	15,000	1100	16.5
3	Four	20,000	973	19.55

A state point analysis was conducted for the proposed Four-Stage Bardenpho process. This analysis was completed in order to determine the surface area required to properly settle out the projected mix liquor suspended solids (MLSS) concentration for the proposed Four-Stage Bardenpho process. Depending on the limited solids flux that can be transported to the bottom of the clarifier, if enough surface area is not provided for settling under peak flow conditions, the MLSS could overload the underflow capacity, buildup within the clarifier, potentially push to the surface and flow out of the clarifier via the effluent weir, causing a high TSS effluent concentration. The state point analysis graphically illustrates if the proposed clarifier system can adequately remove the mixed liquor or if the proposed process's operating MLSS will overload the settling capacity of the clarifier.

The State Point is graphically represented at the intersection of the Overflow Rate Operating Line and the Underflow Rate Operating Line. If a clarifier is operating within its settling parameters, the State Point will be shown below the Settling Flux Curve calculated for the clarifier. In addition, the Underflow Rate Operating line will also be below the Settling Flux Curve. If the State Point is shown above the Settling Flux Curve in any condition, the material will not settle in the clarifier but will flow out of the clarifier via the effluent weir. Similarly, if the Underflow Rate Operating line is shown above the Settling Flux Curve in any condition, the sludge blanket is projected to rise and also exit the clarifier via the effluent weir.

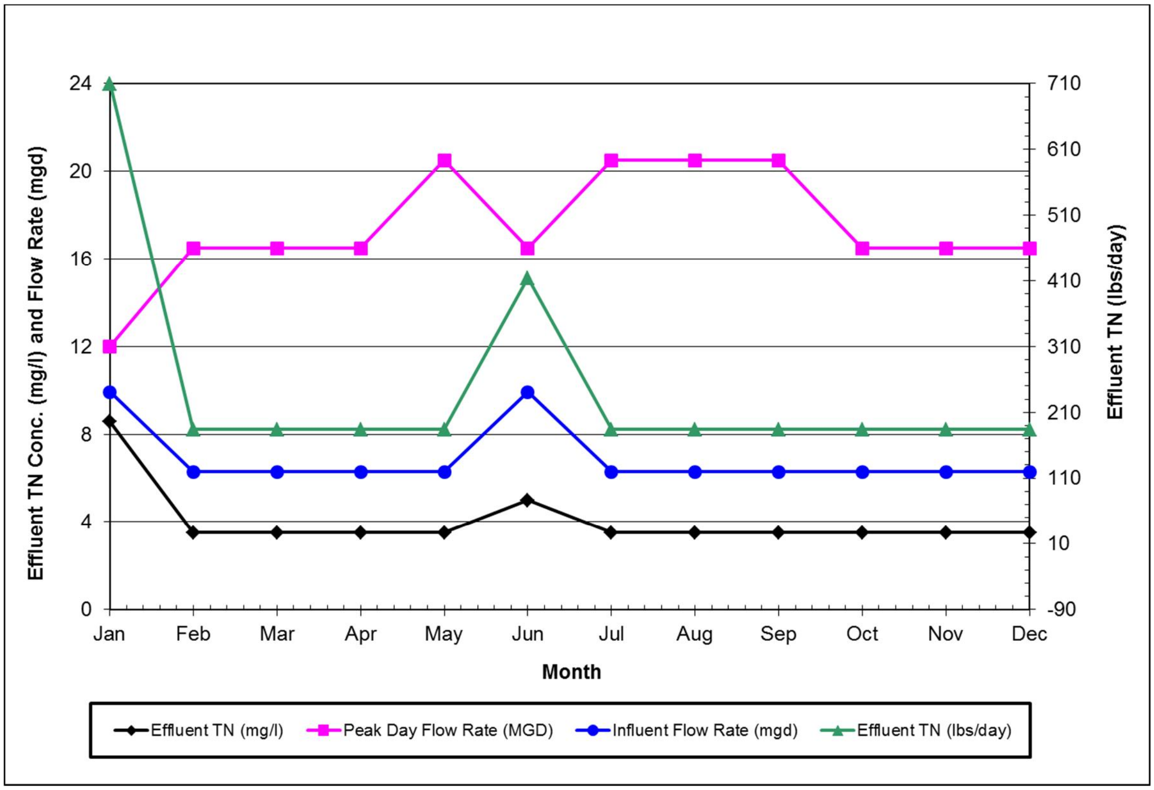
Figure 6-5 graphically summarizes the results of the state point analysis for the Four-stage Bardenpho process (existing four aeration tanks and three secondary clarifiers) at the annual average operating condition. The peak flow rate that a particular process can reliably treat is contingent upon a good settling sludge (represented as the sludge volume index (SVI) number) and the operating mixed liquor concentration.

FIGURE 6-5
CITY OF TORRINGTON WPCF FACILITIES PLAN
STATE POINT ANALYSIS – FOUR-STAGE BARDENPHO PROCESS
ANNUAL AVERAGE



Although, the state point analysis shown above in **Figure 6-5**, indicates three clarifiers would have the settling capacity for the average day flow and loading condition, the clarifiers need to be designed to handle peak instantaneous flows also. An additional state point analysis was conducted to determine the peak day flow capacity for the proposed Four-Stage Bardenpho process (existing four aeration tanks and three secondary clarifiers) for a typical 12 month period. As shown in **Figure 6-6,** the peak flow capacity varies month to month based on influent loading condition, wastewater temperature and subsequently the operating mixed liquor level.

FIGURE 6-6
CITY OF TORRINGTON WPCF FACILITIES PLAN
TYPICAL 12 MONTH PERIOD – EFFLUENT TN QUALITY

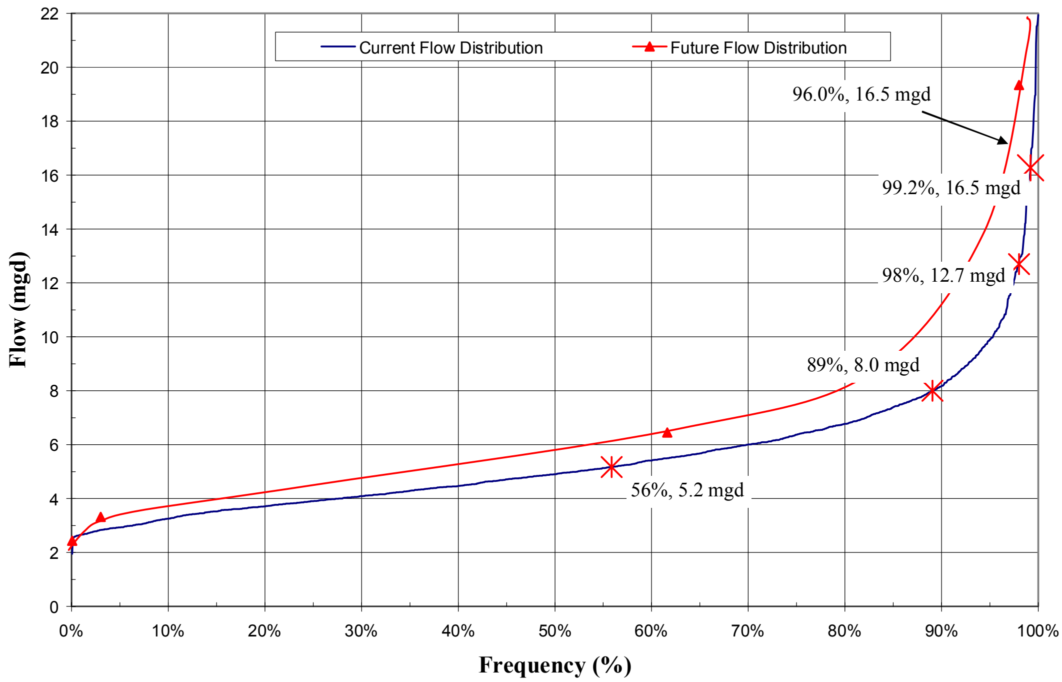


Notes:

3. Max Month Conditions (9.93 mgd @ 10,100 lbs BOD_5/day)
4. Average Conditions (6.3 mgd @ 7,000 lbs BOD_5/day)

A flow analysis was completed in order to estimate the risk of installing only one new secondary clarifier. Using the January 2007 to October 2011 plant data, a flow distribution chart was developed in order to determine what percentile of time the 16.5 mgd threshold would be under for both the existing and future projected flow conditions (refer to **Figure 6-7**).

FIGURE 6-7
CITY OF TORRINGTON WPCF FACILITIES PLAN
INFLUENT FLOW DISTRIBUTION



Under current conditions, flows less than 16.5 mgd occurs 99.2% of the time. If the future design flow conditions match the current flow distribution pattern, 16.5 mgd would be identified as the 96th percentile flow design rate. Although typical design criteria for a secondary clarifier are based on the 98th percentile design flow condition, the WPCF may elect not to construct a fourth new clarifier as part of the recommended upgrade. The WPCF may elect to construct one new secondary clarifier now, to replace the existing rectangular clarifiers, and provide space for the installation of an additional fourth clarifier in the future.

An advantage to installing the fourth clarifier now would be the additional redundancy / backup it provides should one of the remaining three clarifiers needed to be taken out of service during peak flow conditions. If the fourth secondary clarifier were constructed during this upgrade, it would be partially funded by a grant through the Clean Water Fund (CWF) Program. It should be noted that CWF grants are currently not funding projects that are being constructed to accommodate growth in a sewer system. Thus a disadvantage to not constructing the fourth clarifier during this upgrade would be that Torrington may not receive grant funding in the future to construct the clarifier when needed for performance reasons.

6.7 RECOMMENDATIONS FOR SECONDARY TREATMENT

6.7.1 Alternative 1 - MLE Process

The Modified Ludzack Ettinger (MLE) process was modeled for the Torrington WPCF and it was determined that this process cannot achieve Torrington's 2014 Total Nitrogen Waste Load Allocation. The Torrington WCPF can rely on this process as long as it can continue to purchase nitrogen credits (i.e. the nitrogen credit trading program). Modeling identified that the MLE process could achieve an effluent Total Nitrogen concentration of 5 mg/l (approximately 266 lbs/d), utilizing all four aeration tanks, approximately 20 lbs/d more than the 2014 permit limit of 248 lbs/d. It should be noted this process was modeled under winter conditions; the MLE process could achieve a concentration that would likely meet the 2014 permit limit during summer months. However, under a peak flow condition the process was projected to achieve a Total Nitrogen effluent concentration of 13.5 mg/l, equivalent to approximately 1,160 lbs/d. Should this occur during a maximum month loading condition, the annual average total nitrogen load measured in the effluent flow could be substantially affected, potentially increasing to 340 lb/d or more.

6.7.2 Alternative 2 - Four-Stage Bardenpho Process

The Four-Stage Bardenpho process can achieve Torrington's 2014 total nitrogen waste load allocation without the need to purchase nitrogen credits under most anticipated flow conditions. Recommended improvements include internal modifications to the existing aeration tanks, a new third secondary clarifier and a supplemental carbon system. Additional aeration tanks are not required based on the current nitrogen credit trading program (Scenario No.1). If Torrington was required to achieve TN permit compliance on a monthly versus, an annual average basis, then two additional aeration tanks would be required under future flow and loading conditions.

The proposed improvements include the ability to operate the secondary anoxic zones as swing zones. These zones may be required to be aerated during maximum month low, temperature conditions to maintain complete nitrification. Subsequently, an increase in the effluent nitrogen is expected. The Four-Stage Bardenpho process can achieve approximately 3.5 mg/l effluent total nitrogen under future annual average flows and loadings, even at minimum wastewater temperatures. **Figure 6-4** graphically summarizes results of the process modeling for a typical 12 month period. Specifically, if the future maximum month loading conditions occurred in January (commensurate with very cold water) the proposed process would need to be operated with the swing zones aerated (essentially an MLE process) resulting in an elevated effluent nitrogen concentration (8.5 mg/l). If that same future maximum month loading condition occurred in June (commensurate with warm water) the proposed process would still be able to operate as a Foure-Stage Bardenpho process with only a slightly elevated effluent TN concentration (5.0 mg/l). Process modeling indicates that, at the annual average loading condition, the proposed process can achieve 3.5 mg/l effluent nitrogen regardless of the wastewater temperature. Based on the information presented in Figure 6-4, under future flow and loading conditions, a Four-Stage Bardenpho in the existing aeration tanks with three secondary clarifeirs, could achieve an annual average effluent TN loading of 247 lb/d.

Similar to the MLE process, additional structures and piping will be required to divert and evenly split flows to each tank. The installation of larger internal recycle sludge pumps, mixers and diffusers in each anoxic zone will also be required.

In their current configuration, the newer circular final settling tanks are not hydraulically capable of accepting flows from the effluent end of Aeration Tank No's 1 and 2. The original two aeration tanks discharge to three rectangular settling tanks because of this hydraulic limitation. If the Four-Stage Bardenpho process is selected, it is recommended that the elevation of the effluent weirs in Aeration Tank Nos. 1 and 2 be raised so that the flow from Aeration Tank No's 1 and 2 can be diverted to the secondary clarifiers. The three rectangular secondary clarifier tanks would be abandoned in this alternative.

6.7.3 Alternative 3 - IFAS (Four-Stage Bardenpho) Process

Torrington could elect to upgrade the existing Aeration Tank volume to a Four-Stage Bardenpho process and add IFAS media to the first-stage aerobic zone of each tank. Based on the preliminary data provided by Hydroxyl, this option would allow the Torrington facility to meet the Nitrogen General Permit and may result in the potential to sell credits. However, the initial capital expenditure would be the greatest of the upgrade alternatives. The IFAS alternative would require the same equipment upgrades as the Four-Stage Bardenpho alternative, but also include the cost to purchase and utilize the IFAS technology. In addition, the IFAS process requires that fine screening is provided at the WPCF Headworks. Typically, IFAS manufacturers will require that screening down to 1/8th to 1/4 -inch be provided to prevent blinding of the media retention screens. Also, note that if it is necessary to take a tank off-line for maintenance, it will be necessary to transfer the media to an alternate location resulting in additional O&M costs associated with this alternative.

6.7.4 Alternative 4 - Do Nothing

Torrington could elect not to upgrade the existing activated sludge system and continue to purchase nitrogen credits. Note that for this alternative, process modeling indicated that the existing two circular final settling tanks did not provide sufficient capacity to handle peak flow events. Thus a third secondary clarifier is needed to replace the rectangular clarifiers that cannot be used because of the hydraulics limitations. In order to meet the design year peak instantaneous flow, Torrington will need to construct a fourth circular secondary clarifier. As outlined earlier, if construction of a fourth clarifier was not included in the upgrade, The City may not receive grant opportunity in the future for construction of the fourth clarifier in a subsequent project.

6.7.5 Secondary Treatment Recommendation For Nitrogen Removal

As discussed above, the Four-Stage Bardenpho Process and the IFAS Process were the only two evaluated process that could meet the future 2014 Total Nitrogen General Permit Discharge Limit without the need to purchase credits. Based on the state point analysis, a third and fourth clarifier is needed for both alternatives. The following Table is a summary of a present worth cost evaluation of these two processes. The annual operating costs includes power cost and chemical usage for supplemental carbon.

TABLE 6-9
CITY OF TORRINGTON WPCF FACILITIES PLAN
CAPTIAL AND PRESENT WORTH ALAYSIS – NITROGEN REMOVAL

Alternative	Capital	Annual Operating Cost	Present Worth
Four-Stage Bardenpho	$4,623,000	$316,000	$9,328,000
IFAS	$6,139,000	$359,000	$11,475,000

The Four-Stage Bardenpho Process was determined to be the most cost effective secondary treatment process that could meet the 2014 annual average effluent TN limit. As part of this alternative, Aerations Tanks 1, 2, 3 and 4 would be upgraded to include additional baffle walls to create / rearrange the existing zones to adjust the anoxic and aerated treatment volumes providing additional process felxibility. The installation of larger internal recycle sludge pumps, mixers and diffusers in each anoxic zone will also be required. Although the blowers were observed to have the needed capacity to meet future air demands for secondary treatment, the blowers are observed to be aged and approaching their useful life. As part of the preliminary design, it is also recommended to complete a cost evaluation to determine if it is beneficial to replace the blowers with newer, more effect equipment to ensure reliable future operation.

Process modeling indicated that the current configuration with two aeration tanks in service could achieve an effluent Total Nitrogen of approximately 8.3 mg/l on an annual average basis at a temperature of 14°C. This would yield an effluent Total Nitrogen load of 445 lbs/d, approximately 197 lbs/d over the 2014 permit limit of 248 lbs/d. This alternative would require the city to continue to purchase nitrogen credits. The Aeration Tanks No's 1 and 2 will need to be put into operation if the facility wishes to remove nitrogen during the winter period (cold temperature).

6.8 EFFLUENT DISINFECTION

The existing chlorine contact tanks provide a total volume of approximately 338,000 gallons. At the permitted peak flow of 20 mgd, the total contact time for chlorine disinfection is approximately 24 minutes. Design criteria for effluent disinfection provided in TR-16 states that, in the absence of site specific studies to demonstrate minimum contact time needed to achieve adequate disinfection, the minimum contact time at the peak flow rate should be 30 minutes. When the original facilities were constructed, the minimum contact time design basis was 15 minutes. As discussed in Section 4, the highest flow that can be detained for the full 30 minutes currently prescribed in TR-16 is approximately 16.1 MGD.

Currently, effluent disinfection is required on a seasonal basis. Based on a review of historic plant operating records from January 2007 through October 2011, the influent flows to the WPCF are greater outside of the effluent disinfection season. The flows and loads evaluation, summarized in Section 3 of this report, projects a future peak flow of 25.76 MGD and a future max day flow of 25.17 MGD, while the future maximum month and future annual average flows are both expected to be under the 16.1 MGD threshold (13.31 MGD and 6.31 MGD respectively). This projected flow is based on a review of 58 months of plant operating data and assumptions on future growth within the service area. The projected 98th -percentile maximum daily flow is approximately 15.93 MGD, and the projected 98th - percentile hydraulic peak flow is 18.76 MGD. At these peak flow rates, the detention times in the chlorine contact tanks would be approximately more than 30 minutes and 25 minutes respectively.

The table below summarizes the number of instances the maximum flow has exceeded 16.1 MGD during the disinfection season in the past 5 years:

TABLE 6-9
CITY OF TORRINGTON WPCF FACILITIES PLAN
DISINFECTION SEASONAL FLOWS GREATER THAN 16.1 MGD

Year	No. of Days with Peak Flow > 16.1 MGD
2007	0
2008	0
2009	0
2010	0
2011*	4

* Data through September 2011

Of these 4 instances when the peak flow was above 16.1 MGD, the effluent fecal coliform counts did not violate the NPDES permit; the geometric means stayed below the 30 day and 7 day limits of < 200 / 100 mL and < 400 / 100 mL respectively.

It should be noted as well that during disinfection season, there were no violations of the geometric mean limits in the NPDES permit with the exception of a "TNTC" (Too Numerous to Count) value reported on July 13th, 2009. The City of Torrington WPCF reported however that the staff had failed to run sufficient dilutions to obtain fecal count on that specific day. The laboratory standard of practice (SoP) was modified to ensure that sufficient dilutions were to be used to obtain counts from this date forward. Since 2009, a TNTC level has not been reported.

Even during separate occasions of large scale rain events (i.e. nearly 7 inches of rain on August 28th, 2011 associated with Hurricane Irene) where the fecal coliform counts were significantly higher than normal (315 / 100 mL), there were no 7 day or 30 day geometric mean violations, and as such, Torrington did not violate their permit.

The existing volume of the chlorine contact tanks provides 30 minutes of contact time up to 16.1 MGD. With the exception of three separate large scale rain events that occurred during the disinfection season between May of 2007 and September of 2011, the maximum flow that occurred at the Torrington WPCF was 15.0 MGD. Because of this, and the historical successful performance of disinfection at the facility, the existing chlorine contact tanks appear to provide sufficient capacity for disinfection throughout the entire season.

Class B surface waters are considered suitable for fish and other aquatic life and wildlife, recreation, navigation, and industrial and agricultural water supply. Because the Naugatuck River is designated as a Class B river, the Torrington WPCF is subjected to stringent treatment requirements, particularly with respect to chlorine residual. The Torrington WPCF has generally met the permit requirements for chlorine residual. However, the existing chlorine residual permit limits of 0.10 mg/l instantaneous and 0.05 mg/l maximum daily can be difficult to maintain.

The existing chlorination and dechlorination facilities are in good condition, and would require minimal, if any modifications to continue to provide reliable operation. It is recommended that effluent disinfection continue to rely on sodium hypochlorite disinfection and sodium bisulfate for dechlorination.

SECTION 7

SECTION 7

PHOSPHOROUS REMOVAL EVALUATION

7.1 INTRODUCTION

Recently, the CT Department of Energy and Environmental Protection (DEEP) along with the United States Environmental Protection Agency (EPA) has begun to mandate the reduction of phosphorus in Connecticut waters. The City of Torrington WPCF, upon renewal of their NPDES permit, will be subject to these new regulations in the near future. Even though the City of Torrington WPCF has yet to receive their renewed NPDES permit with a phosphorus removal compliance schedule, it is essential the WPCF Facilities Planning Study evaluates potential phosphorus treatment alternatives that will allow the facility to meet the future limit.

A summary of the state's phosphorus reduction program and how it will affect the Torrington WPCF can be found in Section 2 of this report. The proposed phosphorus limit expected to be included as part of the next Torrington NPDES permit reissuance is shown below in Table 7-1.

TABLE 7-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
EXPECTED DISCHARGE LIMITS FOR TOTAL PHOSPHORUS

	Current Average	Design average
Flow (MGD)	5.5	6.31
Total Phosphorus (mg/l)	0.37	0.32

The CT DEEP has also indicated that these proposed limits have been reviewed and supported by the EPA and are not anticipating any changes to the proposed phosphorus limits in the near future. Although it is not anticipated that the EPA will challenge the 2011 revised phosphorus discharge limits established by the DEEP, these proposed limits are still considered interim and not guaranteed for the long-term. It should be noted that the current permit limits were mathematically developed using a model to establish and project phosphorus enrichment factors for each discharge source within the watershed. These limits can potential be challenged in the future, moving the State to evaluate and establish a water quality based phosphorus limit as additional data are collected on specific watersheds. In addition, the status of receiving waters to achieve their legislated water quality criteria could also influence an adjustment of the established phosphorus limits. Accordingly, it will be important for Torrington to implement a tertiary phosphorus removal system that has the flexibility to potentially achieve an even more stringent limit (as low as 0.1 mg/L) in the future.

The ultimate objective of this section is to determine a plan that will allow the City of Torrington WPCF to meet the future proposed compliance schedule for phosphorus removal within the means of the planned facilities upgrade.

7.2 EVALUATION APPROACH AND CONSIDERATIONS

The phosphorus treatment alternatives recommended for consideration at the Torrington WPCF were determined by the following:

- **Cost**: A 20-Year Net Present Worth Analysis was completed to select the lowest cost wastewater treatment alternative, which analyzed both capital and operational costs. However, various impacts and benefits to the operation and overall treatment process were also considered during the evaluation process, providing for a more holistic approach.
- **Land Availability for Additional Tankage / Retrofitting:** For various biological and chemical phosphorus removal processes, additional tankage would be required. The site constraints were reviewed to confirm land availability.
- **Hydraulic Profile**

  The hydraulic profile of the existing facility was verified for existing and future flow conditions. The additional headloss of the various phosphorus reduction processes being evaluated herein were also evaluated to determine the need of an intermediate or effluent pump station. The hydraulic profile for the installation of a tertiary treatment alternative will need to be further fine tuned during the preliminary design phase of the project; it is likely that a high-flow, low-lift style pumping station will be required. Therefore, for this facilities plan, a "Tertiary Pump Station" was included in the overall recommended plan.

7.3 PHOSPHORUS REMOVAL TECHNOLOGIES

The specific total phosphorus limit will have a significant effect on the treatment technology requirements; and, as presented previously, the City should also be cognizant of possibly more stringent limits in the future. Phosphorus removal can be accomplished through biological and/or chemical removal processes. 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 (PO_4) is the largest component of this group.

A brief description and the generally accepted levels of treatment for total phosphorus are summarized below; each increasing level correlates to a lower total phosphorus effluent threshold.

- **Level 1 Phosphorus Limit (1.0 mg/l)** - A Level 1 phosphorus limit can be achieved either through biological phosphorus removal, chemical/physical removal or both. Generally, chemical/physical removal at this level is chemical injection upstream of the secondary clarifiers.
  - **Chemical Treatment**
    - Advantages : reliable (not prone to process upsets) and simple
    - Disadvantages: long-term O&M costs (chemical purchasing) and increased sludge production
  - **Biological Treatment**
    - Advantages: improved MLSS settling
    - Disadvantages: performance dependent on influent wastewater characteristics and secondary clarifier performance

- **Level 2 Phosphorus Limit (0.5 mg/l)** - A Level 2 limit can also be achieved either through biological phosphorus removal, chemical/physical removal or both. In some cases a filtration step may be required, but usually only necessary for overloaded or poorly performing secondary clarifiers.
  - **Chemical Treatment**
    - Advantages: not prone to process upsets, simple
    - Disadvantages: long term O&M costs, increased sludge production
  - **Biological and / or Biological with Supplemental Chemical Treatment**
    - Advantages: lower chemical use, dual processes provide robust phosphorus removal
    - Disadvantages: increased O&M costs due to supplemental chemical usage

- **Level 3 Phosphorus Limit (0.2 mg/l)** - Treating to Level 3 involves adding a physical removal process downstream of the secondary clarifiers (i.e. filtration using either cloth disk or shallow bed sand filter systems) in order to provide enhanced solids removal (i.e. capture of the small chemically-developed phosphorus solids that escape the secondary clarifiers). The costs to achieve Level 3 limits are greater than Level 2 limits.
  - **Effluent Filtration**
    - Advantages: reliable, fairly simple and operator friendly technology
    - Disadvantages: significant capital investment to pass the peak hydraulic loading, effluent filtration can result in significant headloss issues, may need to pump the wastewater, may not be suited for future expansion if more stringent phosphorus removal is required

- **Level 4 Phosphorus Limit (0.1 mg/l)** - Treating to Level 4 involves a chemical/ physical removal process downstream of the secondary clarifiers and typically employs a "ballasted flocculation settling process" (i.e., *Actiflo™; Co-Mag™; or Densadeg™*) or a "buoyant flocculation flotation process" (i.e. *AquaDAF™*). These technologies achieve significantly better phosphorus removal levels, due to their ability to achieve lower effluent TSS levels, thereby capturing more particulate phosphorus. It should be noted that the chemical conditioning to remove dissolved phosphorus is similar to Level 3; however, a minor increase in chemical demand would be needed. The costs to achieve Level 4 limits are greater than Level 3 limits.

- **Ballasted Flocculation**
  - Advantages: robust phosphorus removal capabilities, well suited for future expansion if lower effluent phosphorus levels are required, lower headloss characteristics vs. an effluent filter
  - Disadvantages: high capital cost, additional operator requirements, long term O&M costs, increased sludge production

- **Level 5 Phosphorus Limit (0.05 mg/l)** - Treating to a - Level 5 limits represent the limits of conventional treatment technology and thus the costs are significantly greater than for both Level 3 and 4 limits. Achieving Level 5 limits would typically require the use of: a) tertiary membranes; b) a combination of Level 4 plus Post-Filtration; c) dual-stage filtration (i.e. *Dynasand, BluePro*). The costs to achieve Level 5 limits are significantly greater than Level 4 limits.
  - **Tertiary Membrane Treatment**
    - Advantages: exceptional solids removal
    - Disadvantages: significant capital costs, significant O&M cost (power and chemical)
  - **Ballasted Flocculation Followed by Filtration**
    - Advantages: phased approach installation, flexible
    - Disadvantages: significant headloss (filtration), limited full scale application

It is anticipated with the reissuance of the NPDES permit, the Torrington WPCF will be within the Level 3 technologies (0.4 mg/l - 0.3 mg/l). However, as stated previously, the CT DEEP has also indicated that the proposed interim limits are still not necessarily final. Designing and constructing for Level 4 phosphorus limit (0.1 mg/l) will ensure future flexibility.

7.4 BIOLOGICAL PHOSPHORUS TREATMENT ALTERNATIVES

It should be noted that the Level 1 through Level 5 Phosphorus levels of treatment summarized above are based on past experience and engineering design criteria/safety factors for secondary treatment. Facility operating conditions and the performance within the secondary clarifier can have a significant effect on the efficiency of the treatment process. A portion of the influent phosphorus is used by the bacteria present in the secondary treatment process for growth. The remaining phosphorus is removed by enhancing the biological uptake of the phosphorus or chemical removal. In all cases, the phosphorus is converted to a solid and wasted out of the system.

Recently the Torrington WPCF has added mixers to the first two zones of Aeration Tanks 3 and 4, and moved the internal recycle pump discharge to the second zone in order to create an anaerobic zone for enhanced biological phosphorus removal within the secondary treatment process. Sampling of the effluent flow has shown the new anaerobic zones, at current loadings, have reduced the effluent phosphorus concentrations below the Level 3 limit; achieving effluent total phosphorus concentrations as low as 0.29 mg/l (May 2012).

To achieve a total phosphorus (TP) limit of 0.1 mg/l, a chemical phosphorus removal step will be required because biological phosphorus removal alone is not capable of meeting a 0.1 mg/l limit. Furthermore, a very high level of solids removal is 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.

Although the Torrington WPCF is currently achieving exceptional biological removal results, only a limited amount of data have been collected to date during warmer weather temperatures and low flows. The facility has been shown to achieve effluent total phosphorus concentrations as low as 0.29 mg/l; however, it cannot be assumed these low TP levels will be maintained under all loading conditions, including future annual average and max month loading conditions. As discussed in the previous subsection, theoretically, biological phosphorus removal within the aeration tanks, and chemical precipitation within the primary and secondary clarifiers, may not consistently remove a sufficient amount of phosphorus to meet the anticipated seasonal average limit of 0.32 mg/l to be included in the next NPDES permit renewal for the Torrington Facility. In addition, a limit as low as 0.1 mg/l, or lower, could be issued in the future.

It should be noted, during the evaluation of BNR alternatives, as discussed in Section 6 of this report, the model showed that some phosphorus reduction was achieved in the secondary treatment process. However, an analysis of the future flows and loads showed that the proposed BNR system could only achieve an effluent phosphorus concentration of 1.0 mg/l, while maintaining an acceptable effluent total nitrogen concentration that would meet the effluent total nitrogen permit limit. Biological phosphorus removal and nitrogen removal processes both rely on carbon as the driving force behind their performance. Hence the two processes are in competition with each other over carbon. This competition can therefore reduce the performance of both processes.

The Process model also determined the entire available treatment volume within the existing aeration tanks was needed in order to achieve an acceptable annual average effluent total nitrogen concentration. Under annual average loading condition, the volume within the existing aeration tanks, approximately 2.58 million gallons, provided the necessary anoxic and oxic treatment volumes to achieve an effluent total nitrogen concentration of less than 3.5 mg/l; however, during the maximum month loading condition, the 4-Stage Bardenpho process was shown to be overloaded and subsequently at risk for ammonia violations. To address nitrification concerns, the proposed 4-Stage Bardenpho process would need to have the ability to aerate the second anoxic zone (effectively turning the 4-Stage Bardenpho process into an MLE process). While the total nitrogen level would be significantly higher during this month, compliance with the annual average limit would still be achievable. In order to enhance biological phosphorus treatment and achieve an effluent phosphorus concentration of less than 1.0 mg/l, additional anaerobic volume would be needed within the secondary treatment process. However, as stated previously, all of the existing aeration tanks volume is needed to achieve the required effluent total nitrogen concentration permit levels.

Although incorporating a combined biological nitrogen and phosphorus removal process upstream of the tertiary process could reduce tertiary chemical usage, it would also compete for aeration tank volume utilized for nitrogen removal. Given the limited tankage availability for additional treatment volume, and recognizing a tertiary phosphorus system would ultimately still be required to achieve a future 0.1 mg/l effluent phosphorus standard, priority is given to utilize the aeration tanks for nitrogen removal. Therefore, combined biological nutrient removal processes using the activated sludge system were not further considered.

Discussed below are a series of technologies and alternatives that would be able to meet the anticipated seasonal average phosphorus limit of 0.32 mg/l, and potential future seasonal average phosphorus limit 0.1 mg/l.

7.4.1 Tertiary Treatment Alternatives

Two tertiary treatment alternatives (Effluent Filtration and Ballasted Flocculation) were considered since it was anticipated that the City of Torrington will receive a seasonal phosphorus limit of 17.29 lb/d of 0.32 mg/l at future design average flows. Each of the tertiary alternatives could be added downstream of the existing secondary clarifiers. In addition to the addition of metal salts to the tertiary treatment process, it is recommended to design the chemical feed system to allow for the addition of metal salts upstream of the headworks and secondary clarifiers to allow for some chemical phosphorus removal prior to the tertiary system. Chemical phosphorus removal consists of adding a metal salt to the waste stream to convert the soluble phosphorus to particulate form. Then the particulate phosphorus is removed via 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 exponentially as the effluent TP limit is reduced).

7.4.2 Effluent Filtration

Effluent filtration is a proven technology to remove particulate material. To achieve phosphorus removal, a metal salt (i.e. ferric chloride, PAC or Alum) is added to the secondary clarifier effluent wastewater to precipitate the soluble phosphorus. The precipitated solids are then removed by the filter. An effluent total phosphorus concentration of 0.2 mg/l (Level 3) is readily achieved by effluent filtration (unless an unusually high level of recalcitrant or nonreacitve phosphorus is present in the wastewater). There are several different types of filtration systems including sand filters, cloth filters and plastic media filters.

7.4.2.1 Cloth Filters

This approach would consist of using cloth disk filters (woven or pile) to meet a total phosphorus limit of 0.2 mg/L. Although there are a number of manufacturers for this type of equipment, the basis of design for this evaluation was the Kruger Discfilter. With Discfilters, water to be treated flows from the center drum to the inside of the filter elements. The cloth media is mounted on both sides of submerged discs. The media separates solids from the water, and clean water flows through the filter elements. A backwash pump (one per filter) rinses accumulated solids from the inside of the discs. Backwash water is then sent to the head end of the plant for treatment. A layout of a typical cloth disk filter is shown in the Figure 7-1.

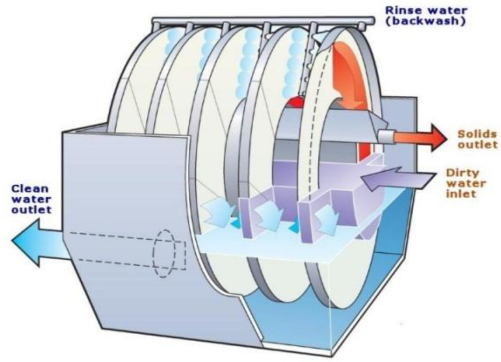


Figure 7-1: Typical Cloth Disk Filter

New internal baffle walls, influent and effluent channels, and piping would direct secondary effluent flow to one of the four filter units. Periodically, each filter would be backwashed to remove solids, which would be directed to either the primary clarifiers or solids handling facility.

An intermediate pump station would be needed to address hydraulic profile concerns. A properly designed and operated effluent cloth filter should allow Torrington to achieve an effluent total phosphorus limit of 0.2 mg/l.

A cloth filter tertiary alternative provides the following advantages:

- Effluent TP ~0.1-0.2 mg/l
- Simple operation
- Low O&M cost (associated with the filter unit)
- Low chemical use and no pH adjustment

A cloth filter tertiary alternative has the following disadvantages:

- Larger footprint required for multiple units

- May not meet a future 0.1 mg/l Effluent TP if required

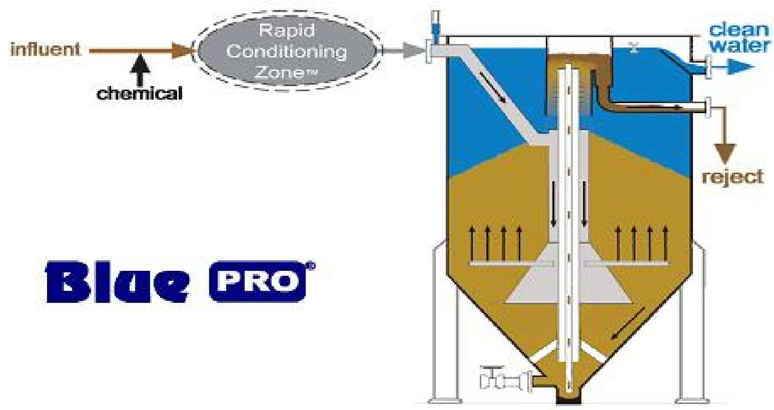
7.4.2.2 BluePRO

The BluePRO system relies on absorption and coagulation-filtration for phosphorus removal. This system is a deep bed, continuous backwashing sand filter that uses a hydrous ferric oxide (HFO) coating on the granular media surface which will adsorb the dissolved phosphorus in the secondary effluent. The adsorption within the BluePRO system helps with the capture of the phosphorus that can typically escape a conventional sand filter. The HFO coating regenerates continuously on the surface of sand grains in the media. This removal mechanism is more efficient than coagulation-filtration processes for removing phosphorus and can typically achieve phosphorus removal levels of 0.1 mg/l to 0.2 mg/l. A basic representation of the BluePRO process is shown in Figure 7-2.

A chemical coagulant is injected into the influent wastewater stream upstream of a rapid conditioning zone, prior to entering the filter. This mix zone allows for proper contact time for both co-precipitation and adsorption. The mixture enters the moving bed sand filter through arms at the bottom of the filters and flows upwards through the sand bed. Clean water exits at the top of the filters. The sand moves slowly from the top to the bottom and then returns to the top of the filter with the use of an airlift. This system does not require intermittent shut-down for backwashing or constant cycling and thus operates continuously on a flow through basis, nor does it require the use of polymer.

FIGURE 7-2

BLUEPRO PROCESS FLOW DIAGRAM



The BluePro Tertiary Alternative provides the following advantages:

- Combined P removal & denitrification
- Effluent TP ~0.1-0.2 mg/l
- Simple operation
- Low O&M cost (associated with the filter unit)
- Low chemical use and no pH adjustment
- Small footprint required for biological treatment
- Modular design with fiberglass packages or concrete tank installation
- Ability to recycle reject to head of plant

The BluePro Tertiary Alternative has the following disadvantages:

- Limited Full/pilot scale applications
- Proprietary media
- Large footprint to pass full peak hydraulic flow

7.4.3 Ballasted Flocculation Process

There are several competing ballasted flocculation processes available for use as a tertiary phosphorus removal process. In general, each employs 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 Siemens, CoMag processes were utilized for comparison purposes against effluent filtration. However, all of the ballasted flocculation processes in the marketplace should be considered in the future if ballasted flocculation is the preferred phosphorus removal technology.

7.4.3.1 Actiflo

ACTIFLO represents a ballasted flocculation process that consists of several steps within the overall treatment process; a typical configuration is provided in Figure 7-3. Each step is identified and generally described below.

- ***Chemical Injection*** - A coagulant (ferric chloride) is introduced upstream of the main process to ensure sufficient mixing at the point of chemical injection. An additional chemical (caustic) may be required at this injection point to maintain the proper pH through the process.

- ***Coagulation Tank*** - Following chemical injection, a coagulation tank is used to provide additional time to allow for complete chemical reaction.

- ***Injection Tank*** - Microsand (ballast) and polymer are injected into this tank. Mixing is provided to ensure complete bridging of the solid particles, polymer and the sand. The microsand acts as a weighted ballast to increase the settling ability of the solids particle.

- ***Maturation Tank*** - The maturation tank allows for the flocs to increase in size ultimately improving the settling performance of the particles.

- ***Settling Tank*** - The settling tank provides quiescent conditions to allow the flocculated particles to settle to the bottom; lamella tubes at the top of the clarifier improve the solids removal performance. Clarified water is collected in troughs above the lamella tubes and settled sludge is continuously pumped to the hydro-cyclones where the sand is separated from the sludge and re-injected back into the system.

These individual steps are shown in the process flow diagram presented in Figure 7-3.

The ACTIFLO Tertiary Alternative provides the following advantages:

- Can achieve Effluent TP ~ 0.05-0.15 mg/l (Level 4)
- No downtime for backwashing
- Multiple installations
- Small footprint
- Low hydraulic head requirements

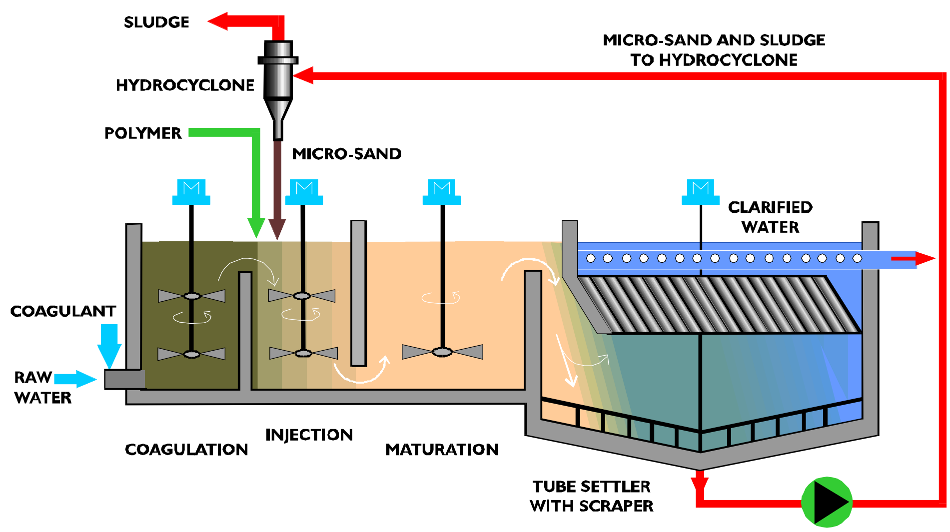
The ACTIFLO Tertiary Alternative has the following disadvantages:

- Power consumption
- Multiple pieces of equipment

This process was retained for further analysis because this ballasted flocculation process has multiple successful installations and can achieve excellent phosphorus removal within a relatively small footprint. This system also has minimal headloss associated with it as compared to the other tertiary systems being evaluated.

FIGURE 7-3

ACTIFLO PROCESS FLOW DIAGRAM



7.4.3.2 CoMag

The CoMag process, recently acquired by Siemens, 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 separated from the slurry composition by using a magnetic drum and returned to the process.

7.5 SELECTING TERTIARY TREATMENT RECCOMMENDATION

Each of the respective manufacturers of the tertiary treatment systems evaluated herein was initially approached for a system design that could meet the initial (0.37 mg/L) and eventual (0.32 mg / L) effluent requirements of the City of Torrington WPCF. The Tertiary system would be designed to treat the secondary clarifier effluent flow, with a phosphorus concentration of 1.0 mg/l. Sufficient treatment units would be provided so that "target treatment levels" at peak day flows can be met with one unit or train offline. For flow rates greater than the Peak Day flow rate, a wet weather operational strategy will be implemented (i.e. the additional flow will be bypassed around the process and blended back in downstream of the process). Design peak day flow was determined to be 15.93 mgd (98th percentile peak day flow).

A conceptual layout sketch of each system, along with preliminary operation and maintenance cost estimates, was developed for each evaluated technology. After review of the layouts for each alternative, the necessary tankage required for each system is available within the footprint of the existing Secondary Clarifier Tanks Nos. 1 and 2 (total dimensions of both tanks is 79' wide X 122' long X 10' deep). The existing tanks would need to be retrofitted, including additional concrete work, in order to fit the required equipment and layout dimensions of the selected alternative. The extent of the needed retrofit work/concrete work is dependent on the alternative selected.

7.5.1 Discfilter (Cloth Filter)

The following is a summary of the developed design parameters for the *Kruger Discfilter* system considered:

Number of Units	4 (3 duty, 1 standby)
Capacity per Unit, mgd	5.4 (at Peak Hour)
Total Capacity, mgd	16.2 (3 units online, 1 standby)

The *Kurger Discfilter* system would require approximately 316 gallons per day of Ferric Chloride to treat up to 6.3 MGD. This would equate to a daily (total) metal salt cost (at the future ADF of the facility - 6.3 MGD) of approximately just under $500 per day. The power required to operate the associated equipment for this alternative has been approximated at $10 per day (under the same operating conditions).

In order to facilitate the installation of the cloth filters, minor concrete work would be needed within the existing Secondary Clarifier Nos. 1 and 2, which would include alterations to the baffle walls and installation of an equipment room.

7.5.2 Blue PRO Granular Media Filtration and Adsorption

The following is a summary of developed design parameters for the Blue PRO system considered:

Number of Units	4 (3 duty, 1 standby)
Capacity per Unit, mgd	6.5 (at Peak Hour)
Total Capacity, mgd	19.5 (3 units online, 1 standby)
Foot Print, per Unit	5,600 ft^2 (80 ft X 70 ft)

Each unit would be made up of 16 'Quads" with a total of 64 "Quads". For phosphorus removal, the filter bed depth would be 60 inches (although required depth of tankage is 25-feet). In order to facilitate the installation of the Blue PRO system, major concrete work would be needed within the existing Secondary Clarifier Nos. 1 and 2, which would include alterations to the interior baffle walls, installation of 16 concrete tanks with 25-feet high walls, installation of two influent channels and installation of an equipment room.

The *Blue PRO* system would require approximately 316 gallons per day of Ferric Chloride to treat up to 6.3 MGD. This would equate to a daily (total) metal salt cost (at the future ADF of the facility - 6.3 MGD) of approximately just under $500 per day. The power required to operate the associated equipment for this alternative has been approximated at $78 per day (under the same operating conditions) regardless of effluent limit parameters and/or influent Phosphorus concentrations into the tertiary system.

7.5.3 ACTIFLO Ballasted Flocculation

The following is a summary of developed design parameters for the *Actiflo* system considered:

Number of Units	3 (2 duty, 1 standby)
Capacity per Unit, mgd	8 (at Peak Hour)
Total Capacity, mgd	16 (2 units online, 1 standby)
Foot Print	3,000 ft^2 (60 ft X 50 ft)

The system layout for the Torrington Facility to meet the future phosphorus limit of 0.32 mg/L would require three separate trains, each with a hydraulic capacity of 8 MGD. Each process train would have a side water depth of approximately 17 feet. In order to facilitate the installation of the Actiflo system, major concrete work would be needed within the existing Secondary Clarifier Nos. 1 and 2, which would include alterations to the interior baffle walls, installation of 9 concrete tanks with 20-feet high walls, installation of an influent channel and installation of an equipment room.

The *Actiflo* system would require approximately 316 gallons per day of Ferric Chloride to treat up to 6.3 MGD. This would equate to a daily (total) metal salt cost (at the future ADF of the facility - 6.3 MGD) of approximately just under $500 per day. The polymer and coagulant costs for this system under the ADF flow conditions are approximatly $29 per day. Each tertiary treatment train would require just over a total connected electrical load of 28.25 horsepower. The power required to operate the associated equipment for this alternative has been approximated at $76 per day (under the same operating conditions) regardless of effluent limit parameters and/or influent Phosphorus concentrations into the tertiary system.

7.5.4 Cost Evaluation of Tertiary System Alternatives

Using the information received from the tertiary system vendors a cost evaluation was completed to determine which alternative would be most cost effective. Table 7-2 is a summary of this evaluation.

TABLE 7-2

CITY OF TORRINGTON WPCF FACILITIES PLAN

TERTIARY SYSTEM COST EVALUATION

	Single Stage Cloth Filtration	**BluePRO Filtration**	**Actiflo Ballasted Floc**
Flow (MGD), Design Annual Average	6.3	6.3	6.3
Total Phosphorus Removed			
Influent TP, mg/l	1.0	1.0	1.0
Effluent TP, mg/l	0.32	0.32	0.32
Removed, lbs/day	42	42	42
Total Annual Cost[1]	$109,900	$124,700	$130,400
Present Worth of Annual Costs	$1,796,500	$2,039,000	$2,132,500
Total Construction Cost[2]	$7,046,000	$10,630,000	$7,364,000
Total Construction and Present Worth Cost	$8,842,500	$12,668,900	$9,496,500
Present Worth Cost/pound of TP removed	$210,000	$301,000	$226,000
Construction Cost/pound of TP removed	$168,000	$253,000	$175,000

1) Total Annual Cost includes estimated chemical costs and electrical costs associated with the tertiary system operation.
2) Construction Costs include the estimated equipment, structural, architectural, electrical, instrumentation and mechanical costs associated with the installation of the tertiary system.

Table 7-2 shows that the *Discfilter* tertiary treatment system is the most cost effective alternative of the technologies evaluated. However, the *Actiflo* system is shown to be only approximately 7% higher in cost.

Of the phosphorus removal tertiary systems evaluated, the *BluePRO* system was determined to be the most expensive. This is mainly due to the required foot print and additional concrete work needed to facilitate the installation of the equipment. The Actiflo system, for example, has three separate process trains, each occupying approximately 1,000 square feet, for a total space requirement of 3,000 square feet for the process tanks. In comparison, the Blue PRO system has 16 separate tanks that would require in total nearly 5,600 square feet.

7.6 TERTIARY TREATMENT TECHNOLOGY RECOMMENDATION

All three types of tertiary treatment systems evaluated herein are able to meet the phosphorus limits of 0.32 mg/l and 0.37 mg/l (at 6.31 MGD and 5.50 MGD average daily flows respectively). However, only two of these systems, *BluePRO* and *Actiflo*, can also reach a potential long-term future limit of 0.1 mg/l without additional equipment or structural changes. In order to achieve the lower TP limit, *BluePRO* and *Actiflo* systems would need additional chemical to increase the TP capture rate.

Although the D*iscfilter* system was shown to be the most cost effective, it does not provide the flexibility to meet a more stringent 0.1 mg/l effluent limit without major alteration including the installation of additional equipment downstream. The *Actiflo* system was shown to be within 7% of the cost of the *Discfilter* system, and would provide the flexibility of meeting a TP effluent concentration of 0.1 mg/l.

It should also be noted that the *Actiflo* ballasted flocculation system is essentially a high performance clarifier. As summarized in Chapter 6 of this report, the secondary treatment process will need the addition of a third secondary clarifier with the implementation of the recommended secondary treatment process, and a fourth secondary clarifier when future projected influent peak instantaneous design flows are achieved. Installation of a ballasted flocculation type system would provide additional settling capacity downstream of the third secondary clarifier during high flow events, essentially eliminating the need for a future fourth secondary clarifier. Based on the cost and flexibility of the ballasted flocculation type system, it is recommended a ballasted flocculation type tertiary treatment system be installed at the Torrington WPCF for phosphorus treatment.

SECTION 8

SECTION 8

EVALUATION OF SLUDGE DISPOSAL FACILITIES

8.1 INTRODUCTION

Wright-Pierce completed a sludge disposal study for the Torrington Water Pollution Control Authority (WPCA) in 2002. The objectives of the study were to evaluate the anticipated future cost of liquid versus dewatered sludge disposal, options for dewatering of the thickened sludge, and other potential improvements to the existing solids handling facilities. The evaluation of the solids handling process was updated in 2007 and again in this Facilities Plan Report for the City of Torrington WPCF.

Since the initial evaluation was performed, the following has changed at the WPCF:

- The Gravity Belt Thickener has aged further;
- CT DEEP phosphorus regulations are in the process of being implemented; planning is needed for the processing, hauling and disposal of tertiary sludge in the future;
- The Transfer Sludge Pumps have been replaced; and
- The WPCF staff has become more confident in dewatering technologies after piloting a Huber Screw Press unit in the Fall 2010 and Fall 2011.

As part of this Facilities Plan study, the following objectives were identified:

- Calculate future design year 2035 sludge production rates, including tertiary sludge;
- Investigate sludge hauling and disposal costs for liquid and dewatered sludge to determine how the market has changed since the last evaluation update;
- Evaluate cost effective alternatives for waste sludge thickening;
- Evaluate dewatering options and determine capital improvement costs; and
- Develop a layout/site plan for recommended solids handling improvements.

The existing solids handling process and equipment are summarized in Section 4 of this report. Currently the Torrington WPCF thickens primary and secondary sludge. The thickened liquid sludge is hauled to the merchant sludge incinerator operated by Veolia in Naugatuck, CT for disposal. The existing solids handling equipment includes a gravity thickening tank, a gravity belt thickener, unthickened and thickened sludge storage tanks, and sludge transfer pumps.

8.2 SLUDGE QUANTITY

8.2.1 Current Sludge Production

Currently, the solids produced at the Torrington WPCF are a combination of primary and secondary sludge. Generally, sludge production is related to influent wastewater flow, BOD_5 and TSS loadings, and temperatures. A summary of the historic flow, BOD_5 loading, and sludge production is provided in **Table 8-1**.

From an analysis of the existing operations data, the following average sludge yields were obtained:

- Dry Tons / MGal = 0.596
- Dry lbs / lb BOD_5 = 1.175

TABLE 8-1
SUMMARY OF SLUDGE PRODUCTION

	2007	2008	2009	2010	2011*	Average
Annual Average						
Sludge (dry lbs/day)	6450	6608	6587	6654	6534	6567
Influent BOD (lbs/day)	5365	5961	5038	5176	6424	5593
Influent TSS (lbs/day)	4950	5653	4695	4691	6253	5249
Maximum Month						
Sludge (dry lbs/day)	8133	7753	7321	7873	8451	7906
Influent BOD (lbs/day)	6207	7019	6201	5986	7635	6610
Influent TSS (lbs/day)	5838	6442	5989	5903	7772	6389

* 2011 does not include the full November month or entire month of December.

8.2.2 Design Year Sludge Production

The future sludge production at the Torrington WPCF is dependent on the proposed nitrogen removal modifications to the plant. These quantities were developed using the BioWin model based on a four-stage bardenpho process being implemented for nutrient removal and the use of ferric chloride in the secondary clarifiers to achieve an effluent phosphorus concentration of 1 mg/l. To achieve an effluent phosphorus limit of 0.32 mg/l, or future potential limit of 0.1 mg/l, it was assumed that a tertiary process would be required and tertiary sludge would be sent to the head of the WPCF and be co-settled in the primary clarifiers with the primary sludge. The waste activated sludge (WAS) would be handled separately.

The chemical sludge production from the tertiary treatment process was predicted based on meeting effluent phosphorus limits of both 0.32 and 0.1 mg/l. To evaluate the relative capital and O&M costs between the various solids handling alternatives considered, the design-year sludge quantities based on meeting an effluent phosphorus limit of 0.1 mg/l was used. (It should be noted that the DEEP has more recently indicated that an effluent total phosphorus limit of 0.1 mg/l could be imposed in the future; refer to Section 2 and Section 7 of this Report.) The design-year sludge quantities used as the basis of this evaluation are presented in Table 8-2. For the purpose of this evaluation, the sludge flow rates anticipated were based on an *Actiflo* ballasted floc system.

Table 8-2 also includes estimates of the sludge quantities that would be disposed of off-site based on hauling a thickened liquid sludge or hauling a dewatered sludge cake. The values presented in Table 8-2 are based on an assumption that a thickened liquid sludge concentration of 6% total solids or a dewatered sludge cake of 24% total solids could be achieved. Additional backup for the Design Year Sludge Production values, with a summary of all assumptions, can be found in **Appendix D**.

TABLE 8-2

BASIS OF DESIGN: 4-STAGE BARDENPHO PROCESS

DESIGN YEAR SLUDGE PRODUCTION

(YEAR 2035)

	No Chemical P-Removal		Total P=0.32 mg/L		Total P=0.10 mg/L	
	Annual Average	Maximum Month	Annual Average	Maximum Month	Annual Average	Maximum Month
Primary Clarifiers (cosettled with Tertiary Sludge as applicable)						
Primary Sludge, lbs/day	4,037	5,967	5,000	7,798	5,701	8,998
Primary Sludge, gal/day	96,927	130,241	120,081	170,217	136,888	196,409
%TS	0.50%	0.55%	0.50%	0.55%	0.50%	0.55%
Secondary Clarifier						
WAS, lbs/day	4,052	6,033	4,052	6,033	4,052	6,033
WAS, gal/day	69,491	103,464	69,491	103,464	69,491	103,464
%TS	0.7%	0.7%	0.7%	0.7%	0.7%	0.7%
Total Sludge Production						
Total lbs/day	8,089	11,999	9,054	13,831	9,754	15,031
Sludge Disposed- Thickened						
Total, wet lbs/day (92% capture rate)	6,957	10,324	7,724	11,782	8,283	12,741
Total, gal/day (6% solids)	19,465	28,884	21,610	32,962	23,175	35,645
Total, gal/year	5,060,900		5,618,683		6,025,548	
Truck Loads/yr (6,500 gal/yr)	779		864		927	
Sludge Disposed - Dewatered						
Total, dry lbs/day (93% capture rate)	45,239	67,128	50,284	76,699	53,925	82,941
Total, wet tons/day (24% solids)	94	104	105	160	112	173
Total, wet tons/yr	4,901		5,447		5,842	
# of 30 CY Roll Off Containers/yr	220		245		262	

8.3 EVALUATION OF SLUDGE DISPOSAL COSTS

8.3.1 Summary of Current Disposal Costs

The Torrington WPCF currently disposes of their sludge at the Naugatuck incineration facility operated by Veolia. The original contract, with U.S. Filter (now Veolia), was a 10-year agreement that commenced in 1989. Since the expiration of the contract in 1999, the City of Torrington and Veolia execute annual renewals. Torrington now is in a three year contract (March 2010 - December 2013) with Veolia to haul and dispose of the WPCF sludge at the Naugatuck facility.

The Torrington WPCF is currently paying $0.085 per gallon of hauled and disposed sludge. There is a demurrage charge of $60.00 per hour after the first thirty minutes for loading and the WPCF personnel are responsible for loading the trucks. At a solids content of 6%, the cost of transportation and disposal of the sludge (not including polymer consumption to condition the sludge) is approximately $350 per dry ton, exclusive of any demurrage charges.

8.3.2 Summary of Regional Disposal Costs

The current market price for contract disposal of sludge was evaluated through discussions with companies and municipal authorities providing disposal services within the area. The sludge disposal rates for other facilities in the region, as determined from this market survey, are summarized below in Table 8-3.

TABLE 8-3

REGIONAL SLUDGE DISPOSAL COSTS BY FACILITY

Facility	Liquid Sludge Disposal	Sludge Cake Disposal
	Disposal/Hauling Cost	Disposal/Hauling Cost
Synagro	$375 - $400/dt	$295 - $330/dt
MDC[1]	$390-$516/dt	$275 - $268/dt
Current Contract	$0.085 per gallon (approximately $350/dt)	$72.80 per Wet Ton (approximately $330/dt)

Notes: 1. MDC does not provide hauling. The municipality must contract separately for hauling. For this evaluation $180/load was estimated for transportation to the MDC.

8.3.3 Synagro

Synagro accepts both liquid and dewatered sludge, and has no problem with transitioning from accepting liquid to accepting dewatered sludge from a single client on a daily basis. Synagro facilities will accept sludge 24-hours per day and do not have a limitation on the amount of sludge that can be received over the course of any given day. There is a minimum limit of 65% volatile solids content in the received sludge.

Liquid Sludge

Synagro disposes of *liquid* sludge at its New Haven, Connecticut incineration operation. On average, disposal costs (not including cost of transportation) range from $250 to $315 per dry ton. Normally, liquid sludge is hauled in 6,500-gallon tankers. The New Haven facility typically accepts liquid sludge between 2% and 7% though it can accept lower sludge percentages as well as higher solids percentages (up to 9%). If sludge is lower than 2%, the customer must pay a per load rate not a dry ton disposal rate; if sludge is higher than 7%, the sludge must be flowable into and out of tankage. An additional demurrage charge can be accessed if loading takes longer than 60 minutes.

Synagro has provided a budgetary cost for liquid sludge disposal at the New Haven facility of $375 to $400 per dry ton. This cost includes transportation of the sludge from the Torrington WPCF. This range is based on contract duration; a long-term contract would correspond to $375 per dry ton, and a short-term contract would correspond to $400 per dry ton. For this evaluation the more conservative value of $400 per dry ton was assumed. For the future, a $50 increase was assumed per dry ton for the purpose of this sludge disposal and transportation evaluation.

Dewatered Sludge

Synagro disposes of *dewatered* sludge at its Waterbury, Connecticut incineration operation. The unit cost for the disposal (not including transportation) of dewatered sludge at this facility ranges from $240 to $315 per dry ton (based on contract duration as described above). Normally, dewatered sludge is removed in 30 cubic yard roll-off containers.

The Waterbury facility generally accepts dewatered sludge in the range of 20% to 28% solids, however, they can accept solids concentrations as low as 12%; a higher disposal price would be charged due to the excess fuel required to burn off the extra water. The facility is not set up to handle sludge with solids content greater than 30%. Because the disposal fee is determined on a dry weight basis, there is no disposal cost savings associated with improved dewatering performance. However, a savings in transportation costs can be achieved through improved dewatering performance by reducing the overall volume that is transported.

Synagro has provided a budgetary cost for dewatered sludge disposal at the Waterbury facility of $295 to $330 per dry ton. This cost includes transportation of the sludge from the Torrington WPCF. For this evaluation, the more conservative value of $330 per dry ton was assumed. For the future, a $50 increase was assumed per dry ton for the purpose of this sludge disposal and transportation evaluation.

8.3.4 The Metropolitan District (The MDC)

The MDC is located in Hartford, Connecticut and accepts both liquid and dewatered sludge at their facility. The MDC has indicated that there is no minimum or maximum solids concentration of the sludge accepted, as long as their pumps can move it through the system (It was determined a maximum of 35% solids content can be moved by these pumps; however, drier solids hauled to The MDC could be mixed with liquid sludge prior to being pumped into the system). Any sludge less than 17% solids is considered "thickened sludge", and any sludge greater than 17% solids is considered "dewatered, cake sludge". The facility is open 24 hours per day, and there are no limits on the amount of sludge that can be accepted per day.

Based on discussions with The MDC, a municipality could expect a price break (between $10-$20 off the average numbers presented above) if they sign a contract extending one year or more. A municipality can also receive a price break if they haul/dispose of sludge between 6 p.m. and 6 a.m. when there is less activity.

The MDC does not provide hauling and each municipality must contract for hauling separately. The hauler must secure the agreement with The MDC by submitting all pertinent information from the municipality such as sludge composition and frequency of loads. The MDC will then permit the hauler to dispose sludge at the facility. Before The MDC will accept sludge from a treatment facility, an analysis of that sludge for at least the past six months must be provided. Typically, 2-4 monthly or quarterly sludge reports are required for heavy metals analysis.

For purposes of this evaluation, The MDC estimated that for a private hauler to transport sludge from the Torrington WCPF to The MDC facility, it would cost approximately $180 per truck load for current transportation.

Liquid Sludge

The current average unit cost for disposal of liquid sludge at the Hartford facility is $295 per dry ton. The average unit cost for disposal and transportation of liquid sludge at the Hartford facility was estimated to range from $390 to $516 per dry ton based on duration of contract and percent solids content of the disposed of sludge.

Dewatered Sludge

The current average unit cost for disposal only of dewatered sludge at the Hartford facility is $230 per dry ton. The average unit cost for disposal and transportation of dewatered sludge at the Hartford facility ranges from $268 to $275 per dry ton based on duration of contract and percent solids content of the disposed of sludge.

8.3.5 Evaluation of Liquid versus Dewatered Sludge Disposal

As shown in Table 8-3, it was determined that the current liquid sludge disposal rate paid by the Torrington WPCF is below the regional market value. Based on the information obtained from the market survey of regional sludge disposal costs, and the current and projected sludge production, the projected sludge disposal and transportation cost for both liquid and dewatered sludge cake was determined.

For comparison purposes, a solids concentration of 6% for liquid sludge, and 24% for dewatered sludge were assumed in order to calculate the projected volume of sludge hauled from the Torrington WPCF. The budgetary unit disposal and hauling costs provided by the regional facilities, summarized in Table 8-3, were used in determining the projected annual sludge disposal costs for each sludge disposal alternative (Liquid vs. Cake). The total cost for current and future sludge disposal is based on current and future volumes, and 2012 pricing. The future sludge disposal volumes are based on the design conditions, assuming the recommended secondary and tertiary treatment alternatives summarized in Sections 6 and 7 are implemented. To be conservative, the future sludge value also assumed the tertiary system would treat the effluent wastewater to a total phosphorus concentration of 0.1 mg/l. The estimated projected sludge disposal costs for current and future production is presented in **Table 8-4.**

TABLE 8-4

REGIONAL SLUDGE (LIQUID AND DEWATERED) DISPOSAL COST

Disposal Parameter	Unit Cost	Current Production (No tertiary sludge) (est. 1220 dt/yr)	Future Production (with tertiary sludge) (est. 1400 dt/yr)
Current Contract	$350/dt	$427,000	$490,000
Liquid Disposal	$400/dt	$488,000	$560,000
Dewatered Disposal	$330/dt	$403,000	$462,000

Table 8-4 shows that in design year 2035, the Torrington WPCF would save approximately $98,000 per year to dispose of dewatered cake instead of liquid sludge. This differential cost over the design life of the facilities plan upgrade would need to be compared to other operation and maintenance costs, and capital cost for improvements required to implement a sludge dewatering process at the Torrington WPCF.

8.4 EVALUATION OF SLUDGE THICKENING ALTERNATIVES

The Torrington WPCF currently co-thickens primary and WAS in the gravity thickener during the colder part of the year, and thickens WAS separately using a gravity belt thickener in the summer, due to odor concerns. Under future conditions, tertiary sludge it is recommended to be co-settled in the primary clarifiers and sent to the gravity thickener. Because of the additional tertiary sludge to be sent to the gravity thickener, the potential of odors generation and the potential of increasing the nutrient loading to the gravity thickener overflow back to the head of the plant, it is recommended that the WAS be thickened and stored separately from the co-settled primary/tertiary sludge.

If the Torrington WPCF continued to thicken sludge, the facility could utilize the existing thickening process to produce a liquid sludge with a solids content of approximately 6% solids for disposal. Although no process changes to the thickening process would be needed, the existing equipment was evaluated in order to ensure it will continue to operate reliably to the design year.

The evaluation of the existing liquid sludge handling facilities and operation identified a number of issues and operational concerns. The following is a summary of the evaluation of the existing equipment.

8.4.1 Evaluation of Thickening Equipment Alternatives

The existing 1.5 meter gravity belt thickener (GBT) has a design capacity of 200 gpm/meter of belt width or approximately 300 gpm. The existing unit is currently observed to be in fair condition and is only utilized approximately 4 months per year, 3-4 days per week, 6 hours a day. Typically, waste activated sludge (WAS) is co-thickened with primary sludge in the gravity thickener. During the summer months, significant odors can occur in the gravity thickener. During these times, the co-thickening process is discontinued and the GBT is used to thicken the WAS separately while primary sludge continues to be thickened in the gravity thickener. The GBT is approaching the end of its useful life, and therefore, this evaluation includes an alternative analysis to replace the GBT either in-kind or with a different thickening technology. The GBT has been well maintained by the City WPCF staff; although, it was noted that they recently had to replace several parts of the unit in the last 2-3 years.

In this evaluation, a gravity belt thickener (GBT) and rotary drum thickener (RDT) were evaluated. It was assumed that this new equipment will thicken WAS from approximately 0.7% solids to 6% solids. Currently, the existing GBT only operates during working hours when a staff member is available to periodically monitor the thickening process. This is due to the need to inspect/adjust the sludge feed equipment, inspect/adjust the polymer feed equipment and due to the overall lack of reliable automatic controls on the current system. When operated, the GBT operates 6 hours per working day. For sizing purposes, it is assumed a new GBT would continue to be operated in the same manner as the existing, 5 days/week, 6 hours/day.

8.4.1.1 Gravity Belt Thickener

Based on the anticipated design max month WAS generation, it was determined that the existing GBT is undersized if limited to a 6 hour/day, 5 day/week operation period. The selected GBT should be sized to handle the design max month condition of 6,000 lbs/d, as summarized above. If the GBT is operated 6 hours/day, 5 days/week, the design max month condition would be 8,400 lbs/d:

- 6,000 lbs/d X 7 days/week = 42,000 lbs/week,
- 42,000 lb/week ÷ 5 days/week = 8,400 lbs/d

The existing 1.5 meter wide Komline-Sanderson GBT has a loading rate of 1,050 lbs/hr or 6,300 lbs/day (1,050 lbs/hr X 6 hrs/day). In order to meet the max month operating condition the existing GBT would have to operate 8 hours per day. This would exceed the maximum operating period per day desired by WPCF staff. Currently, a member of the WPCF staff operates the equipment during an 8 hour work period, including spending an hour starting up the equipment and an hour shutting it down and cleaning up. If the GBT was to operate 8 hours per day, it would require an operator to work 10 hours per day during the max month condition (8 hours of operation, plus two hours for startup/shutdown) yielding 2 hours of overtime per day (40 hours of overtime for the month).

In order to meet the design max month condition, a 2- meter wide GBT unit is needed. The 2-meter wide GBT has a design loading capacity of 1,400 lbs/hr or 8,400 lbs/day (1,400 lbs/hr X 6 hrs/day). Because of its size, only one unit would be able to fit within the existing space (approximately 18' x 30') available in the current thickening room, although this configuration is extremely tight. Preliminary dimensions suggest there would only be approximately 2-feet of clear space around the equipment which would make it difficult to complete routine maintenance/repairs on the equipment as needed.

The cost for one 2-meter wide GBT, to replace the existing unit, is approximately $165,000. This does not include the cost for a new polymer dilution and blending system. The GBT manufacturer proposed a polymer consumption rate of 6 lbs/DT of sludge; however, current polymer consumption is recorded to be on average 11.5 lbs/DT of sludge.

8.4.1.2 Rotary Drum Thickener

During review of available RDT equipment, and discussions with the WPCF staff, it was identified that the RDT is a less complex piece of equipment then the GBT and has reliable controls. The RDT was determined to need little to no attention by WPCF staff. Therefore, it was assumed the RDT would be operated 24 hours/day, 7 days/week. Using the projected WAS volumes summarized Table in 8-2, the following operating parameters were developed (based on 24 hours/day, 7 days/week operation):

- Current Annual Average : 126 lbs/hr @ 36 gpm (3,000 lbs/day)
- Design Annual Average: 169 lbs/hr @ 48 gpm (4,000 lbs/day)
- Design Max Month: 251 lbs/hr @ 72 gpm (6,000 lbs/day)

Because there is limited space on site, it was assumed that the thickening equipment would be located in the existing GBT room.

For the purpose of this evaluation, a Vulcan RDT was considered the basis of design. Based on the anticipated design max month WAS generation, Vulcan has proposed two (2) Model LFST-365 Rotary Drum Thickeners with a max loading rate of 100 gpm each. One of these units would be sized to run 24 hours per day, 7 days/week, at future max month conditions (72 gpm). The second RDT would be recommended to provide complete redundancy, should the main unit need to be taken out of service for maintenance. Preliminary dimensions show that the existing gravity belt thickener room would have adequate space for a two RDT system configuration.

The cost for two RDT units to replace the existing thickening equipment is $180,000. This does not include the cost for a new polymer dilution and blending system. The RDT manufacturer proposed a polymer consumption rate between 7 and 12 lbs/dry ton of sludge.

8.4.1.3 Cost Evaluation of Thickening Equipment

The key factors driving the thickening equipment recommendation include:

- Net Present Worth Cost of the Equipment (Capital and O&M Costs over the lifetime of the Equipment)
- Client preference
- Space/configuration requirements
- Ability for redundancy
- Ease of Operation and Maintenance

A net present worth analysis was performed on each thickening technology. As part of this analysis, the following operation and maintenance assumptions were made for the GBT:

- Net Present Worth analysis was completed for both the current polymer usage rate and the manufacturer estimated polymer usage rate: 6 lbs/dry ton of sludge and 11.5 lbs/dry ton of sludge.
- Polymer unit cost assumed to be $1.50/lb (2011 cost)
- Assumed 2.0 HP total for unit
- Electricity unit cost assumed to be $0.15/KW-hr
- Assumed 6 hours per day, 5 days per week operation;
- Assumed 2 hours per day of operator time to Operate (start-up, shutdown and clean up)

The following operation and maintenance assumptions were made for the RDT:

- Net Present Worth analysis was completed for both manufacturer estimated polymer usage rates: 7 lbs/dry ton of sludge and 12 lbs/dry ton of sludge.
- Polymer unit cost assumed to be $1.50/lb (2011 cost)
- Assumed combined 2.5 HP for drum drive motor and flocculator motor
- Electricity unit cost assumed to be $0.15/KW-hr
- Assumed 24 hours per day, 7 days per week operation; and only one unit in operation at a time
- Assumed 1 hour per day of operator time for routine checks of the equipment and periodic maintenance

A summary of the total present worth cost for the GBT and RDT is summarized below in **Table 8-5.** Budgetary capital costs for the equipment were provided by Vulcan and Komline-Sanderson; this cost does not include installation and other construction related costs. The Annual Cost is based on the assumptions listed above, the calculated future annual average WAS production rate of 612 dry tons/year and a 20 year life cycle at 3% inflation.

TABLE 8-5

ESTIMATED NET PRESENT WORTH COSTS FOR SLUDGE THICKENING OPTIONS

	GBT		**RDT**	
Polymer Usage Rate (lbs/Dry Tons)	6 lbs/DT	11.5 lbs/DT	7 lbs/DT	12 lbs/DT
Capital Costs	$165,000	$165,000	$180,000	$180,000
Present Worth of Annual Cost	$229,000	$304,000	$194,000	$262,000
Total Present Worth	$395,000	$469,000	$374,000	$442,000

Based on Table 8-5, the GBT and RDT thickening technology are close in cost although the RDT has a slightly lower present worth cost. As noted previously, the costs in the above table do not include installation cost. It was noted that the existing thickening room may not be big enough to house the new GBT and additional alterations may be required with this piece of equipment. If the GBT is selected, additional costs may be needed to expand the thickening room to house the new equipment. This additional cost could make the RDT the more feasible alternative.

The following is a list of "advantages" and "disadvantages" associated with the two evaluated thickening technology alternatives.

Compared to the RDT, the GBT has several advantages which include:

- Higher throughput unit capacity
- Lower polymer consumption
- Potential to produce a slightly higher solids thickened sludge

It also has some disadvantages such as:

- Large footprint (eliminating configuration space available for redundancy)
- Requires staff to periodically monitor equipment
- Cannot reliably operate after hours

Compared to the GBT, the RDT has several advantages which include:

- Lower capital cost with a smaller foot print
- Cost of two units, to provide 100% redundancy, equivalent to one GBT
- Easier to operate and maintain
- Can operate automatically with little staff attention
- More odor control capabilities (enclosure)

It also has some disadvantages such as:

- Lower throughput unit capacity
- Higher polymer consumption

Although the net present worth of the two thickening alternatives were determined to be essentially equivalent, the advantages of ease of operation, automation and 100% redundancy make the rotary drum thickener a more appropriate selection for this application. If thickening WAS is determined to be the best solids handling option, it would be recommended to replace the existing gravity belt thickener with two new rotary drum thickeners to be installed in the existing thickening room.

8.4.2 Evaluation of Sludge Pumps and Loading Area

8.4.2.1 Thickening Equipment Feed Pumps (TF-1, TF-2, TF-3)

Positive displacement, double disc type pumps are currently used to convey unthickened secondary sludge from the storage tanks to the gravity belt thickener. The original design capacity of the pumps, as reported in the 1995 Operations and Maintenance Manual, is 170 gpm at 32 feet of total dynamic head. In actual operation, it has been necessary to limit the pump speed to 50% of the maximum, due to excessive vibration. Consequently, all three pumps are needed to feed the gravity belt thickener. The combined output of the three pumps operating at a reduced speed to minimize vibrations is below the rated throughput of the gravity belt thickener. This results in additional sludge processing time, which increases operating costs.

An analysis of the pumping system was performed using a sludge factor of 2 to account for the solids content of the unthickened sludge. Based on this evaluation, at the design feed rate of 340 gpm, a total dynamic head (TDH) of approximately 32 feet would be required. This is consistent with the original design criteria of 170 gpm per pump at a TDH of 32 feet with two pumps in service. The assessment that the pumping capacity was hindered by excessive vibration caused by the high rotation speed of the pumps was confirmed as part of the Sludge Disposal Study.

The following recommended improvements are made to address the deficiencies with the gravity belt thickener feed pumps:

- Replace the existing pumps with two new 30 hp progressing cavity or rotary lobe gravity belt thickener feed pumps. Each pump would be equipped with variable frequency drives and sized to feed the gravity belt thickener at a maximum operating point of 400 gpm at 40-ft TDH. This would provide one pump under normal operation and one back-up pump.

- Install a magnetic flow meter on the thickening equipment feed pump discharge piping to better monitor sludge throughput to the thickening process.

8.4.2.2 Thickened Secondary Sludge Transfer Pumps (TA-1)

Recently the single positive displacement, double disc type pump used to convey thickened secondary sludge from the gravity belt thickener to the thickened sludge storage tanks was replaced with a Boerger rotary lobe pump. The pump is rated at 70 gpm at 100-ft TDH. This pump is noted to be operating well and is in good condition. Although the current pumping system is in good condition, the following additional recommendations have been identified for this system:

- Replace the existing aged magnetic flow meter on the thickened secondary sludge pump discharge piping to better monitor sludge throughput of the gravity belt thickener, and the performance of the pump.

- Provide provisions for bleeding trapped air from the discharge line.

8.4.2.3 Truck Loading Pumps (TL-1, TL-2)

Recently the two Vaughn Chopper pumps used to transfer thickened sludge from the storage tanks to tanker trucks for off-site disposal were replaced with two Boerger rotary lobe pumps. Each pump is rated for 400 gpm. These pumps are noted to be operating well and are in good condition. Another key consideration in the evaluation process is the need for speed control. In order to better operate the thickening process, variable frequency drives could be installed to allow the operator to control the loading rate. The following additional recommendation is noted for this system:

- Install VFD on pump to allow adjustment of pumping rate.

- Replace the existing aged magnetic flow meter on the truck loading pump discharge piping to better monitor sludge throughput of the truck loading pumps, and the performance of the pump

8.4.2.4 Truck Loading Area

The truck loading area is located adjacent to the Operations Building. An upgrade to the truck loading area is recommended to address issues with the current operation. The key components of the recommended upgrade include provisions to better contain and cleanup of any spills during the liquid sludge truck loading process. Currently, there is a pavement drain at the truck loading area. However, the area that slopes to the pavement drain does not encase the entire tuck. Additional grading is recommended in this area to better contain the entire truck loading area in order to collect any spillage from the sludge loading operation as well as any wash water used for cleanup.

Loading of thickened sludge into tanker trucks for off-site disposal is currently carried out by WPCF staff. The loading operation is limited to periods when the plant is adequately staffed. As part of the recommended upgrade, automatic site access and local control of the truck loading pumps should be provided. These improvements would allow haulers to access the site on weekends, without the need for treatment plant staff. Sludge disposal haulers could be provided with a key card and access code for site access at the gate.

The truck loading station could also have a simple control panel accessed by the hauler using the key card and access code. The control system would allow the hauler to turn on the truck loading pumps, pump approximately 6,500 gallons into the tanker, and shut the pumps off. The system would meter flow into the trucks and could include pacing control to prevent overflows, such as an automatic shutdown after 6,500 gallons have been pumped. The automated key card/code system for truck loading could be linked to the facility's SCADA system to record quantities of thickened sludge hauled off-site.

8.4.2.5 Summary of Recommendations for Thickened Sludge Pumps and Truck Loading

All of the existing sludge pumps are currently located in the basement of the operations building. There appears to be adequate space for the proposed replacement pump options for the gravity belt thickener feed pumps. A summary of the improvements to the thickening process pumping system and truck loading area are as follows:

- Replace the existing gravity belt thickener feed pumps with two new progressing cavity or rotary lobe pumps. One pump would be sized to serve as a back-up truck loading pump with additional discharge piping modifications.
- Install a VFD on the rotary lobe pump truck loading pumps.
- Provide magnetic flow meters downstream of the gravity belt thickener feed pumps, the thickened primary sludge pump, and the truck loading pump.
- Provide a new truck loading area with spill containment and provisions to allow hauler control of sludge pumping.

Table 8-6, is a summary of estimated costs for the required improvements needed should the sludge thickening process be continued at the Torrington WPCF.

TABLE 8-6
COST SUMMARY FOR SLUDGE PUMPING ALTERNATIVES

Demolition	$5,000
Process Equipment (New Pumps)	$118,000
Flow Meters	$59,000
New Truck Loading Area	$70,000
Automation of Truck Loading	$39,000
General Conditions	$48,000
Total Capital Cost	$339,000
Contingency (10%)	$34,000
Total Capital Cost	$373,000

8.5 EVALUATION OF SLUDGE DEWATERING

One of the goals of this study was to determine if a new dewatering facility would be cost-effective, and if so, what dewatering method would be most feasible. Although the current costs associated with the disposal of liquid sludge are favorable, the uncertainty of future disposal costs warrant the evaluation of alternative treatment and disposal options.

For the purposes of this assessment, the following technologies were evaluated:

- Belt Filter Press
- Centrifuge
- Screw Press

8.5.1 Belt Filter Press

Belt filter presses are perhaps the most widely used method of mechanical dewatering and have dominated the municipal sludge dewatering market for many years. Belt filter presses are continuous feed devices consisting of two filter belts and a series of progressively smaller rollers that apply pressure to the sludge forcing the water out through the belts while the sludge is retained between the belts. Sludge is dewatered through a combination of gravity drainage and compression. The dewatering process is made up of three operational stages: chemical conditioning, gravity drainage, and compaction in a pressure and shear zone. Belt presses can run unattended for extended periods as long as the sludge feed concentrations do not fluctuate.

Because fines and polymer are continually pushed into the pores of the filter belts a continuous spray of wash water is required for both belts. The wash water system typically requires a booster pump to get enough pressure, and requires a high flow. The wash water energy requirement can exceed the actual energy need to drive the press. Typically 3 to 5 hp variable speed motors are required to drive the belts, while a 10 to 20 hp motor is needed for the washwater booster pumps.

Some of the advantages of a Belt Filter Press include:

- Can be run unattended for extended periods of time as long as the sludge feed concentrations do not fluctuate (though this is a very difficult parameter to control)
- Technology has dominated the municipal sludge dewatering market for many years and is still a cost - effective means of dewatering

Some of the disadvantages of a Belt Filter Press include:

- Environment can be corrosive due to the high moisture content of air resulting from the spray wash water mist and the wash water and filtrate drains
- Requires a continuous spray of wash water on both belts because fines and polymer are continually pushed through the belts; the wash water system requires a booster pump and high flow

Laboratory bench scale testing of belt filter press dewatering was carried out by Ashbrook Inc. in April 2002. Based on the results of the laboratory testing, Ashbrook predicted a final dewatered cake of 25% to 26% solids using their high performance model. The full-scale performance of belt filter presses has typically been over-stated by manufacturers. Published data and past experience indicate a typical performance for a two-meter press in the range of 18% to 20% cake solids for conventional (low sludge age) secondary sludge at a loading rate of 400 lb/hr/meter belt width, and 20% to 24% for blends of primary and secondary sludge at a loading rate of 800 lb/hr/meter belt width. The higher performance indicated by Ashbrook may be possible with a reduction in the loading rate. During discussions with the manufacturer it was stated the performance of the equipment is highly dependent upon the percent solids of the influent sludge. For the purpose of the Ashbrook equipment selection, it was assumed the feed sludge would be thickened to 6% solids. If thickening is discontinued, a larger unit would be needed to dewater the feed sludge to the 20% to 24% solids range.

A belt filter press is typically constructed using open steel frames to allow full and ready access to all the roller bearings, tension and alignment controls. This is ideal for observing the dewatering process and for maintenance of the belt filter press, but results in the release of odors and moisture that can cause poor working conditions. Typically, the odors from dewatering secondary sludge only are relatively low. However, when primary or a blend of primary and secondary sludge are dewatered, the release of odors can be significant.

In addition to odor concerns, the working environment around the units will typically have high moisture content due to the spray wash water mist and the water falling from the belts. The high moisture content in the vicinity can contribute to corrosion and can create operator exposure concerns.

Because of concerns with odors, and a corrosive atmosphere, a ventilation and odor control strategy would be a key component of a belt filter press installation. An acceptable ventilation and odor control strategy would include:

- An exhaust air hood over the belt filter press.
- A clear strip curtain around the belt filter press area to create a flexible enclosure.
- A chemical feed system for addition of potassium permanganate or hydrogen peroxide.

- An odor control system to treat air exhausted from the enclousre.

For purposes of this dewatering assessment, information from Ashbrook was used for developing capital and O&M costs.

8.5.2 Centrifuge

Centrifuges also have a strong presence in the municipal sludge dewatering market. Centrifugal sludge dewatering uses the centrifugal force developed by the rotation of a cylindrical drum or bowl to separate the sludge solids from the liquid. Centrifuges have been favored whenever sludge disposal costs are significantly reduced by having a high solids content. The centrifuge market is very competitive, with several manufacturers offering units with significant ranges in price, size, capacity, and features.

The solid bowl centrifuge is horizontally mounted and tapered at one end. Thickened sludge is fed into the cylindrical bowl assembly, which rotates between 2,500 and 4,000 revolutions per minute. The high centrifugal force drives the solids against the bowl's interior walls. Difference in densities between the sludge solids and the liquid causes the formation of two distinct layers; sludge cake and liquid centrate. The dewatered sludge cake is discharged at the tapered end, while the centrate is discharged at the opposite end of the unit.

Some of the advantages of the Centrifuge include:

- Typically provides the highest dewatering capability
- Based on typical performance for primary/secondary mixtures, a high solids centrifuge can be expected to achieve a final dewatered cake of 24% to 30% solids. Dewatering performance would be expected to meet or exceed current requirements for regional contract disposal of dewatered sludge.
- Can be highly automated and run unattended for extended periods of time
- Small footprint
- Odor control system size is minimized because the process is totally enclosed

- A small exhaust air stream drawn from the centrifuge would provide containment of emissions. The exhaust would be directed to an odor control unit. Because the required ventilation rates are lower, the size of the odor control process would be smaller than for the belt filter press.

Some of the disadvantages of the Centrifuge include:

- High energy consumption
- High maintenance costs

For purposes of this dewatering assessment, the Centrysis centrifuge unit was used for developing capital and O&M costs. Although the Centrysis unit has automation to run unattended, due to the larger complexity of the equipment and as it operates at a high rpm rate, it is not recommended to run when the WPCF is not staffed. It is recommended that if a centrifuge is selected, it should be operated 6 hours/day, 5 days/week, during normal working hours.

8.5.3 Screw Press

Screw Presses have been used extensively in industrial applications and especially at pulp and paper wastewater treatment facilities for many years. Historically, screw presses have not been used in municipal sludge dewatering due to higher cost and lower throughputs, however, screw presses have proven cost effective on a life-cycle (LCA) costs basis due to the potential for higher cake solids.

There are two technologies that have proven successful; the horizontal rotary screw press (FKC) and the Inclined Rotary Screw Press (Huber).

The Inclined Rotary Screw Press consists of feeding flocculated sludge under pressure (or gravity, if equipped with a flocculation tank) at the head of an inclined screw (12 to 15 degrees) installed within a wedge wire screen. The screw rotates very slowly (typically between 1 to 3 rpm). Filtrate passes through the wedge wire to the drain port. The wedge wire spacing varies across the length (with the wider spacing at the head end and the thinner spacing at the discharge end). As the sludge becomes dryer, the flighting on the screw becomes tighter, increasing the pressure on the sludge to "squeeze" the sludge and remove additional water.

Huber's inclined rotary screw press has undergone several recent design improvements that include:

- Pressure feed versus gravity, flocculation tank feed - the technology can be designed with one of two types of flocculation tank feed systems
- Redesign of the wedge wire screen - the screen rotates while the spray wash bar / nozzles remain fixed
- Pneumatic pressure cone - located at the end of the screw discharge, automatically compensates for changing pressure versus a manually set discharge cone

Some of the advantages of the Inclined Screw Press include:

- Typically out performs belt filter presses with sludge of the same characteristics and performs very well with high concentrations of waste sludge
- Small Footprint
- Fully automated, designed to run unattended
- Slow rotation, small motor; lower energy cost

Some of the disadvantages of the Inclined Screw Press include:

- Limited number of standard sizes and models to choose from with various recommended solids loading and hydraulic loading rates (Note: Huber has stated that they are developing a larger unit model, however, as of the date of this report, it has yet to hit the market)
- The feed pressure requires a significant pressure drop at the polymer and sludge mixer; pressure loss requirements increase as the feed solids increase.

- Screw Press overloading can cause pressure build up in the inlet chamber shutting down the screw press; feed pumps need to be controlled automatically by the screw press system.

Additional Ancillary Items for the Screw Press include:

- Flocculation mixers
- Spray wash water system (continuous plant water flushing not required)

The Huber unit could process thickened sludge and unthickened sludge. In the fall of 2011, Huber piloted a screw press unit with feed sludge entering the inlet at on average 1.2% solids (blended thickened primary sludge and unthickened secondary sludge). The weekly results showed dewatered cake ranging from 23% and 30.7% with a polymer consumption range of 13.733 lbs/ton to 33 lbs/ton of sludge. On average, the dewatered sludge cake was 27.1%. Despite the fact that this sludge was not representative without tertiary sludge for future conditions, the pilot unit showed excellent potential for future use at the City of Torrington WPCF.

For purposes of this dewatering assessment, the Huber Screw Press was used for developing capital and O&M costs.

8.5.4 Evaluation of Dewatering Equipment Costs

An estimate of the probable capital improvement and O&M costs was developed for each of the three technologies considered.

The capital, O&M, and annualized costs take into account only the costs of the respective dewatering processes. The costs do not include the cost of hauling and disposal for the dewatered sludge. Because the cost of regional dewatered sludge disposal is calculated on a dry solids basis, the disposal cost for each alternative would be similar, regardless of the dewatering capabilities.

It should be noted that the sizing for both the belt filter press and the centrifuge was based on a 30 hour/week operational period. Although, both pieces of equipment have the capability to run automatically and unattended, because of the high speed/complexity of the centrifuge operation, and the need to periodically adjust the operation of the belt filter press to address fluctuations in the sludge feed concentration, greater attention may be required from the WPCF staff for these two technologies. Thus, at the request of the WPCF staff, these dewatering alternatives were evaluated assuming operation only during working hours.

8.5.4.1 Cost Evaluation of Dewatering Thickened WAS

Under this scenario it was assumed that the existing thickening process would continue. The thickening process would remove some of the water content from the feed sludge prior to entering the dewatering equipment. This would reduce the hydraulic loading and overall size of the dewatering equipment. The capital cost for the respective dewatering processes was based on sizing for the following operating condition (assuming a 6% solids concentration feed sludge).

- <u>Centrifuge</u> - 5 days/week for 6 hours/day. Design Loading:
  - Future Average: 2,000 lb/hr @ 64 gpm
  - Future Max Month: 3,000 @ 100 gpm
- <u>Belt Filter Press</u> - 5 days/week for 6 hours/day. Design Loading:
  - Future Average: 2,000 lb/hr @ 64 gpm
  - Future Max Month: 3,000 @ 100 gpm
- <u>Screw Press</u> - 7 days/week for 24 hours/day. Design Loading:
  - Future Average: 350 lb/hr @ 11 gpm
  - Future Max Month: 550 lb/hr @ 18 gpm

For the purpose of the dewatering cost evaluation, it is assumed only one unit would be purchased for the belt filter press, centrifuge and screw press units, as only one unit is required to process the max month flow condition. For each of the evaluated technologies, it was assumed an additional back-up unit would not be needed for redundancy. Should the dewatering equipment break down, the Torrington WPCF could still continue to thicken and dispose of liquid sludge.

Excluded from this particular cost analysis is the estimated capital cost for any recommended liquid sludge improvements that may be implemented. The purpose of the annualized cost is to provide a means of comparison between the three dewatering technologies. A summary of these costs are presented in **Table 8-7**. A detailed breakdown of the estimated capital costs is provided in **Appendix E.**

TABLE 8-7
PRELIMINARY OPINION OF COSTS FOR
SLUDGE DEWATERING OPTIONS

	Belt Filter Press	**Centrifuge**	**Screw Press**
Equipment Size	9 Hp	90 Hp	3 Hp
Capital Costs(1)	$1,210,000	$990,000	$865,000
Annual O&M Costs(2)	$89,000	$92,000	$110,000(4)
Annualized Cost(3)	$149,000	$140,000	$151,000

Notes (1) Capital Cost in 2012 base year dollars.
(2) "Normalized" O&M Cost based on assumed costs for mid-point of service life. Cost for hauling and disposal is not included. Cost is shown in "constant" dollars. Does not consider effects of inflation or deflation.
(3) Annualized costs based on a 20% grant and CWF loan at 2% for 20 years.
(4) Screw Press is assumed to operate 24 hours a day, 7 days a week.

If handling thickened feed sludge at a 6% solids concentration, Table 8-7 shows the dewatering equipment with the lowest annualized cost was determined to be the centrifuge. Although this machine operates at the highest horse power, at 90 hp, it requires less operator attention and odor control equipment then the belt filter press.

The screw press was determined to be the dewatering equipment with the highest annualized cost. Although the screw press would operate at a lower horse power, 3 hp, then the centrifuge and belt filter press, and requires less operator attention, it has the highest annual O&M costs. This is due to the assumption the screw press would operate 24 hours per day, 7 days a week, requiring a much higher annual power demand. If the screw press operated 6 hours a day, 5 days a week, it was determined two machines would be needed to handle the design loadings rates, increasing both the capital and annual O&M cost.

8.5.4.2 Cost Evaluation of Dewatering Unthickened WAS

Thickening of sludge prior to sludge dewatering is not required. All three evaluated dewatering technologies can process an unthickened sludge. Abandoning the WAS mechanical thickening process prior to dewatering could actually improve the evaluated equipment performance. However, primary and tertiary sludge would continue to be thickened in the gravity thickener. In order to dewater sludge, additional polymer is added to the dewatering equipment feed stream. It is more difficult to add/mix polymer into a thickened sludge. If the feed sludge is not thickened prior to the dewatering process, it may be possible to mix polymer with the sludge more efficiently, potentially reducing the overall polymer usage in the system. However, an unthickened sludge would have a higher water content, which would substantially increase the hydraulic loading to each equipment alternative. The increased hydraulic loading rate would increase the size of the equipment needed to process the material.

For this evaluation, it was assumed the primary and tertiary sludge would continue to be thickened to 6% in the gravity thickener. However, the WAS would not be thickened and would be stored in the unthickened sludge holding tanks. The unthickened WAS and thickened primary and tertiary sludge would be blended and fed into the dewatering equipment. Under this scenario the existing mechanical thickening process would be abandoned. The capital cost for the respective dewatering processes that would process a blended, unthickened feed sludge, was based on sizing for the following operating conditions (assuming a 1.5% solids concentration blended feed sludge).

- <u>Centrifuge</u> - 5 days/week for 6 hours/day. Design Loading:
    - Future Average: 2,000 lb/hr @ 306 gpm
    - Future Max Month: 3,100 lb/hr @ 458 gpm
- <u>Belt Filter Press</u> - 5 days/week for 6 hours/day. Design Loading:
    - Future Average: 2,000 lb/hr @ 306 gpm

    - Future Max Month: 3,800 lb/hr @ 458 gpm
- Screw Press - 7 days/week for 24 hours/day. Design Loading:
    - Future Average: 360 lb/hr @ 55 gpm
    - Future Max Month: 550 lb/hr @ 82 gpm

For the purpose of this cost evaluation, it is assumed two belt filter press and centrifuge units, and three screw press units, would be installed to provide for redundancy with one unit out of service. Because it is assumed in this scenario there would be no WAS thickening process, redundancy is required to ensure the continual operation of the dewatering process, even during maintenance periods.

The purpose of the annualized cost is to provide a means of comparison between the three dewatering technologies. A summary of these costs are presented in **Table 8-8**. A detailed breakdown of the estimated capital costs is provided in **Appendix E**.

TABLE 8-8
PRELIMINARY OPINION OF COSTS FOR
SLUDGE DEWATERING OPTIONS

Equipment Size	**Belt Filter Press** 9 Hp	**Centrifuge** 275 Hp	**Screw Press** 3 @ 3 Hp
Capital Costs(1)	$1,970,000	$2,800,000	$1,550,000
Annual O&M Costs(2)	$96,000	$126,000	$111,000(4)
Annualized Cost(3)	$192,000	$263,000	$186,000

Notes
(1) Capital Cost in 2012 base year dollars.
(2) "Normalized" O&M Cost based on assumed costs for mid-point of service life. Cost for hauling and disposal is not included. Cost is shown in "constant" dollars. Does not consider effects of inflation or deflation.
(3) Annualized costs based on a 20% grant and CWF loan at 2% for 20 years.
(4) Screw Press is assumed to operate 24 hours a day, 7 days a week.

If handling unthickened feed sludge at a 1.5% solids concentration, Table 8-8 shows the screw press and belt filter press have the lowest annualized cost. Although the cost of the two pieces of equipment are essentially equivalent, the screw press would be recommended based on its ability to operate unmanned and create a much less corrosive environment then the belt filter press. The screw press will also require less odor control measures then the belt filter press.

The centrifuge was determined to have the highest annualized cost. Due to the high volume of unthickened feed sludge a much larger centrifuge was required. It was determined a 275 hp centrifuge would be needed to handle the design loading rates, increasing the overall power consumption and capital cost of the equipment.

The cost developed in Table 8-7 and 8-8 were used to evaluate three solids handling options as presented below.

8.6 EVALUATION OF SOLIDS HANDLING OPTIONS

In order to compare each of the three options presented below, a life-cycle cost analysis (LCCA) was also completed for each of the processes for dewatered sludge disposal and for the liquid sludge disposal only option. The life-cycle cost analysis considers the estimated construction cost as well as the operation and maintenance costs over the life of the process. The three evaluated options are as follows:

1. Option 1 - Continue to Dispose of Liquid Sludge

The costs associated with this alternative would include:

- Purchase of two new rotary drum thickeners
- Replacement of the two thickening equipment feed pumps
- The installation of three flow meters
- Alterations to the truck loading area
- Installation of a key card system.
- Hauling and disposal costs of liquid sludge

2. Option 2 - Dewater Thickened Liquid Sludge
   - Purchase of two new rotary drum thickeners
   - Replacement of the two thickening equipment feed pumps
   - The installation of three flow meters
   - Alterations to the truck loading area

- Installation of a key card system.
- Purchase of Centrifuge equipment that would process 6% feed sludge.
- Improvements to the garage area for cake storage in a roll off container.
- Hauling and disposal costs of Cake sludge

3. Option 3 - Dewater Thickened Primary/Tertiary Sludge and Unthickened WAS

The costs associated with this alternative would include:

- Purchase of three screw presses that would process 1-2% feed sludge.
- Improvements to the garage area for cake storage in a roll off container.
- Hauling and disposal costs of Cake sludge

A summary of the LCCA analysis is presented in **Table 8-9**. Additional backup for these costs can be found in **Appendix E.**

TABLE 8-9

PRELIMINARY OPINION OF COSTS FOR DEWATERING OPTIONS

	Option 1	**Option 2**	**Option 3**
Construction Cost	\$650,000	\$1,640,000	\$1,550,000
Present Worth of Annual O&M Costs(1)	\$700,000	\$2,070,000	\$1,650,000
Present Worth of Annual Disposal Costs(1)	\$8,331,000	\$6,873,000	\$6,873,000
LCCA Value	\$9,700,000	\$10,600,000	\$10,100,000

Note: (1) O&M and Disposal Cost based on an average of the annual cost over the service life.

The following key parameters were used in the development of the life-cycle cost analysis:

- Unit cost for liquid sludge hauling and disposal is \$400/dt.
- Unit cost for dewatered sludge hauling and disposal is \$330/dt.

At this time, the life-cycle cost analysis indicates that improving the solids handling facilities by implementing a dewatering process would have essentially the same life cycle cost as upgrading the current thickening process. However, it was noted that implementing a dewatering process would reduce the annual disposal cost as well as the operator time needed to run the dewatering equipment. It was noted that the screw press could operate 24-hours per day, seven days per week, with little operator attention. Due to the reduction in the annual disposal cost and routine maintenance of the Screw Press, compared to the thickening equipment, it is recommended to implement a dewatered sludge disposal process versus liquid sludge disposal. Before any final decision regarding the addition of a dewatering process at the Torrington WPCF is made, a commitment to a long-term disposal contract should be considered. If a long-term disposal contract was desired by the Torrington WPCA, a competitive bid process would provide the necessary cost information to verify the economic viability of sludge dewatering.

SECTION 9

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SECTION 9
EVALUATION OF ANCILLARY ITEMS

9.1 INTRODUCTION

In addition to the evaluations of the treatment process and equipment, evaluations of ancillary processes and facilities were also completed as part of the facilities study. The items evaluated in this section includes:

- Odor Control
- Staffing
- Building Systems
- Pump Station Security
- Plant Security

The work in this section was conducted for the *Wastewater Facilities Planning Study for the Torrington, Connecticut Water Pollution Control Facility* completed in February 2007. Any updated information related to these items or new operational concerns have been reflected in the respective discussions.

9.2 ODOR CONTROL

The WPCF processes raw wastewater, septage, and grease trap pumpings, all of which can create odor emissions that can have an off-site impact. There are no existing odor control systems at the Torrington WPCF. As part of the 2007 facilities plan work, an odor evaluation was conducted. The evaluation methods included ORP measurements at various unit processes and a qualitative assessment of potential odor sources. The evaluation considered both potential off-site affects as well as working conditions within the WPCF.

These evaluations were conducted during the summer season when the air temperature was around 76ºF. Typically, odor emissions from wastewater treatment processes are greater when the air and wastewater temperatures are higher. The odor generating potential of sludge processes is less dependent on temperature, but is affected by increasing solids concentration. Odors from sludge processes can be problematic throughout the year. In addition, intermittent processes such as sludge pumping, septage and grease handling, and sludge truck loading, can cause intermittent periods of high odor emissions.

9.2.1 Field Measurements

Oxidation-Reduction Potential Monitoring

Oxidation-reduction potential (ORP) measurements are used as an indicator of the odor generating potential of various unit processes at the WPCF. General guidelines for interpretation of ORP results are shown in **Table 9-1**.

TABLE 9-1
CITY OF TORRINGTON WPCF FACILITIES PLAN
CLASSIFICATION OF WASTEWATER CONDITION BY ORP

ORP, milli-volts	Comments
+200 or Higher	Aerobic Environment (No odor Concerns)
+50	No activity by anaerobic bacteria (Minimal odors)
0	Poor anaerobic activity (Moderate odors)
-100 to -200	Maximum efficiency for anaerobic activity (Problem odors)
-50 to -300	Favored by sulfate-reducing bacteria for production of sulfides (Problem odors)

The ORP monitoring results indicate that the wastewater enters the WPCF with an ORP of less than -100mV. This is indicative of a potential for sulfide production. The septage receiving and grease handling facilities would be expected to contribute to low ORP in general; however during the site visit, the contrary was observed. The results of the ORP testing conducted during the site visit are shown in **Table 9-2**.

TABLE 9-2

CITY OF TORRINGTON WPCF FACILITIES PLAN

RESULTS OF OXIDATION-REDUCTION POTENTIAL TESTS

Location	ORP, mV
Siphon Structure	-118 (1)
Distribution Box #1 -During Grease Decanting	-92 (1)
Distribution Box #1 -Without Grease Decanting	-130
(Former) Grease Fractionation Tank Decant	+10
Primary Settling Tank Inlet - During Grease Decanting	-110
Primary Settling Tank Outlet - During Grease Decanting	-120
Primary Clarifier Outlet - Without Grease Decanting	-139
Aeration Tank Inlet	-107
Return Activated Sludge	-135
Gravity Thickener Outlet	-135
Sludge Holding Tanks (2) - Near Surface	-170
Sludge Holding Tanks (2) - Near Bottom	-190
Sludge Holding Tanks (3) - Near Bottom	-170
Aerobic Digestion Pilot Test	+225 (1)
Thickened Sludge Storage Tank	-183

(1) Average Value of Multiple Measurements

(2) Non-aerated with 1.5% Solids

(3) Aerated with 1.5% Solids

It should be noted that strong odors were observed during the transfer of grease into the temporary frac tank as well as during the removal of thickened grease; ORP measurements were taken during a decant event. A measurement of ORP +10 was taken during the decant event. This high ORP reading from the grease decant is likely the result of a recent delivery of "fresh" grease trap pumpings. The grease trap pumpings are agitated as they are discharged to the primary scum well as well as during decanting into Distribution Box No. 1. This agitation may help to aerate and increase the ORP of the decant.

While a positive ORP generally indicates a reduced potential for odor emissions, the agitation of the decant water during discharge to the treatment process resulted in odor emissions which could be detected around the frac tank and the primary settling tanks. The odors were characteristic of a volatile organic acid, with a sulfide smell. The sulfide odors tended to quickly dissipate following the short duration of decanting.

The recently constructed FOG Receiving Facility is an enclosed system from the truck to the frac holding tanks, thus odors from the transfer of grease are not as prevalent as the temporary frac tank set up. In addition, a carbon style canister was installed to aid in mitigating any odors associated with the FOG Receiving Facility. This odor system is meant to be a temporary system and has been working adequately; however, the FOG odor system would be tied into any future odor control system.

Septage was not received on the day of the evaluation; therefore, no ORP measurements were made. However, septage receiving facilities typically contribute to odor emissions at most wastewater treatment plants. Odors from septage can be strong, particularly during the unloading process and when the septage is transferred to the influent wastewater stream. Septage typically has a low ORP and can increase the septicity of the wastewater.

No ORP measurements were taken in the Screenings Building and no significant odor emissions were noted during the site visit. Typically, headworks facilities can be a source of odors as the influent wastewater is agitated. The odor concentration would be expected to vary as the characteristics of the influent wastewater vary. Strong odors were noted at the screenings container, but the odors were localized odors indicating that the overall emission rate was low.

Odor emissions at the primary settling tanks were not high during the evaluation except when the frac tank was decanted. The ORP at the primary settling tank inlet was measured as -110 mV, indicating the potential for sulfide generation. A relatively small decrease in ORP across the primary settling tanks was measured. Often the primary settling tank effluent weirs can be a source of odors as the wastewater is agitated. Again, the odor concentration at the effluent weirs would be expected to vary with the characteristics of the primary settling tank wastewater, such as during septage pumping or when decant from the FOG system is discharged.

The ORP of the aeration tank inlet was similar to the primary settling tank outlet ORP. Odors observed at the aeration basins and final settling tanks were typical of the activated sludge process and were not observed to be unusually strong even though the ORP of the return sludge was -135 mV. This low ORP could result in higher than normal odor emissions from the aeration basins and final settling tanks due to the potential for sulfide generation.

The low ORP of the return sludge is typically a result of maintaining a significant sludge blanket. Operators reported a seven foot sludge blanket depth in the final settling tanks on the day of the evaluation. Generally sludge blankets of this depth are not recommended because it can promote certain types of filamentous growth, result in denitrification and "popping" of the sludge blanket, and has the potential to generate odors. It is likely that the blanket was being maintained at this level to achieve the conditions needed for the aerobic digester trials that were being carried out at the time of the evaluation. Equipment changes that would accompany long term use of aerobic digestion would correct this potential odor causing condition.

The ORP measurement of both the return activated sludge and the effluent from the gravity thickener were -135 mV. These ORP results are indicative of high sulfide generation potential.

The ORP measurements in the four sludge holding tanks varied greatly as the tanks were being operated differently. Waste Sludge Holding Tanks No. 1 and No. 2, which were being used for secondary sludge thickening, had ORPs of approximately -190 mV and -170 mV respectively. These low ORPs suggest the thickening operation was producing odor generating conditions. The sludge holding tanks contained waste secondary sludge that had been decanted to approximately 1.5% solids. This is more than twice the typical concentration. Tank No. 1 was in the process of being decanted further and was not aerated and Tank No. 2 was being aerated. Generally, concentrating waste secondary sludge through decanting would be expected to promote odor generation.

Sludge Holding Tanks No. 3 and No. 4 were dedicated to the aerobic digestion trials and were not observed to produce odors. Both tanks had been filled with thickened, co-settled primary and secondary sludge two or more weeks prior to the field measurements. The ORP measurements on the day of the evaluation were greater than 200 mV indicating that the digestion process had proceeded to the point that the coarse-bubble aeration system was able to maintain aerobic conditions in the digesters. The WPCF staff has reported that strong odor emissions had occurred at times during the trials. This could indicate that during the initial stages of digestion odor generation could be a problem but would decrease as digestion proceeded.

Mixing of primary sludge with secondary sludge can produce a rapid increase in biological activity and oxygen demand. Depending on the relative concentrations of primary and secondary sludge, it may be difficult for the aeration system to initially satisfy the oxygen demand. This would likely result in significant odor production.

The strongest odors from the thickened sludge holding tank were observed when the hatches to the mechanically mixed thickened sludge storage tanks were opened. The ORP in the thickened sludge storage tank was -180 mV. The thickened sludge storage tanks are enclosed, but not ventilated.

Loading of a tanker truck with thickened sludge was not observed. However, on past visits, the truck loading operation releases strong odors through the tanker vent line that did not dissipate quickly.

<u>Community Odor Survey</u>

A community odor survey was conducted during the site visit. The survey was carried out by slowly driving the roads surrounding the WPCF and making note of off-site odors. Although no objectionable odors were noted at this time, there have been periodic odor complaints received at the WPCF in the past.

<u>On-Site Odor Survey</u>

Significant odor emissions were not observed during the site visit either at the siphon structure or in the Screening Building, even though the low ORP measurement of the influent wastewater indicates a potential for sulfide generation and odor emissions.

Strong odor emissions were noted during grease trap receiving. Odor levels of greater than 170 dilutions to threshold (D/T) were observed at Distribution Box No. 1 when grease trap pumpings were being received in the primary scum well, pumped to the frac tank, and decanted from the frac tank. As stated earlier, the recently installed FOG Receiving Facility included the installation of a carbon canister style odor control system to mitigate odors during the transfer of grease from the truck to the frac tank. However, the volatile organic acid odors that are characteristic of grease traps were also noticeable at the primary settling tanks following decanting. Sulfide odors from the primary settling tanks were noticeable, but quickly dissipated.

At the time of the evaluation, strong odors were emanating from the small grit dumpster in the primary sludge grit handling room. In addition, there is a larger dumpster for both screenings and grit located adjacent to the septage receiving area. Reportedly these dumpsters and the drain catch basins for the dumpsters can be a source of strong odors.

As noted during the ORP measurements, odors at the aeration basins and final settling tanks were not observed to be unusually strong even though the ORP of the return sludge was low.

Odors were noticeable at the gravity thickener and at the sludge holding tanks, but appeared to dissipate quickly. Odors were not particularly strong in the gravity belt thickener room during the odor survey. However, this may reflect the impact of an "essential oil" system that was operating in this room.

9.2.2 Development and Evaluation of Odor Control Alternatives

The on-site odor survey confirmed the existence of strong odors that, on occasion, have caused objectionable odor impacts beyond the property fence line. Significant odor sources should be grouped for treatment in a common odor control system. Based on observations, anecdotal information, and the data collected, the following sources may require odor containment and exhaust treatment to eliminate objectionable off-site odor impacts:

Preliminary Treatment Facilities

- Septage Receiving and Holding Tank
- Siphon Structure
- Screening Building

Primary Settling Tanks

- Influent Channels
- Primary Clarifiers
- Effluent Channels
- Grit Dewatering Area

Sludge Handling Facilities

- Gravity Thickener
- Waste Sludge Holding Tanks
- Thickener Room (GBT)
- Thickened Sludge Storage Tanks
- Truck Loading Facilities for Thickened Sludge
- FOG Receiving Facility

Preliminary Treatment Facilities

One grouping of sources to a common odor control system would be the preliminary treatment facilities, including the septage receiving facilities and the Screenings Building.

Septage receiving facilities are typically odor sources at most wastewater treatment plants. Potential septage odor emission locations at the Torrington WPCF include the septage receiving channel, the septage holding tank vent, and the septage discharge into the raw influent wastewater in the Screenings Building. Septage trucks are typically not an odor source as they draw in air as they discharge septage.

The septage receiving channel has hinged aluminum plate covers. These hinged aluminum plates constitute the primary means of controlling emissions. The covers are typically left open due to the difficulty in opening and closing them. The installation of small hatches, or adding hatch hardware to assist with the opening and closing of the existing hatches would likely improve the current situation.

Septage Receiving Facilities

The septage receiving tank currently has a passive vent. To provide odor control, the ventilation rate in the septage receiving and holding tank would need to be relatively high to contain emissions when a section of the cover is open for raking the manual bar rack. It may be preferable to construct a small structure over the channel that would provide containment over the manual bar rack. The ventilation rate for the headspace above the maximum water level of the septage holding tank should be 12 air changes per hour or as necessary to maintain negative pressure in the tank. The storage of septage in the unventilated tank could increase the risk of accelerated corrosion of the tank walls. In addition to providing active ventilation, coating of the concrete is recommended to reduce corrosion.

The lack of aeration in the septage holding tank is another odor consideration. A vertical shaft mixer is provided to mix the contents of the septage holding tank and resuspended any grit and solids prior to the operation of the septage transfer pump. An aeration system could replace the mechanical mixer. It is likely that aeration would produce greater emissions from the holding tank; however it should reduce emissions during discharge of septage to the

Screenings Building. This may be desirable, especially if odor control is not provided at the Screenings Building or Primary Settling Tanks. Also, the improved condition of the septage may benefit the secondary system both in terms of reduced odor emissions and reduced potential for septicity-related filamentous growth in the aeration basins.

As part of the process evaluation detailed in Section 5, replacement or modifications to the existing septage tank mixer was recommended. Should this recommendation be implemented, and aeration not provided, it may be appropriate to size any future odor treatment system to include air from aerating the septage holding tank.

Odor control should be provided for the septage handling facilities as part of a larger odor control system for multiple sources. If the exhaust from the septage tanks and channels is grouped with exhaust from the screening building or other odor sources (i.e. primary scum well or Distribution Box No.1), the viable odor control technology options would be biofiltration and wet scrubbing. These odor sources are expected to be too humid and high strength for activated carbon and an iron sponge may not be appropriate for the organic reduced sulfur compounds in the septage. The high strength of the exhaust would tend to favor biofiltration provided sufficient space is available. Biofiltration is generally considered the best odor control technology for these compounds, although wet scrubbing may also be feasible.

Based on field observations and measurements, odor control of the Screening Building might be considered a secondary priority. The channels within the screening room are covered with a combination of aluminum checkered plate and concrete. The air exhausted from the screenings building is not treated and the doors are often left open during non-freezing conditions. A concern with this operational procedure is the odors associated with the screenings. The screening material removed from the wastewater contains organic material that can be odorous. However, a grinder/wash/press for the screenings is being recommended as part of the process improvements. This wash press would reduce the organic content of the screenings and presumably the associated odors.

Odor control of this area could be provided by drawing exhaust air directly from the covered channels and wetwell, and potentially the grinder/wash/press units and canister area, and directing it to an appropriate odor control process. As previously noted, grouping the Screenings Building with other adjacent odor sources into a common odor control system would be an economically desirable alternative.

Odor control technologies would be the same as mentioned for the septage receiving facility, namely biofiltration or wet scrubbing. An alternative for localized odor control for the general exhaust of the screenings building would be an essential oil system. Essential oil systems are safe for inhalation and could be sprayed directly into the workspace to improve working conditions or into the exhaust duct to minimize off-site odors. Essential oil spraying in the area of the proposed screenings wash press may be advisable.

Primary Settling Tanks

Sulfide odors from the primary settling tanks were minor during the site visit. However, considering the ORP readings and relatively large surface area, the primary settling tanks should be considered a potential odor source. Current operation of the primary settling tanks limits the sludge blanket depth. This practice minimizes odor forming conditions. The future operation of the primary settling tanks should continue to balance the need to hold primary sludge in the clarifier to enhance thickening against the potential to generate odors as the sludge is held.

Distribution Box No. 1 is covered by grating and could easily be modified to contain odors using rubber mats over the grating. This low cost approach is recommended over changing to aluminum checkered plate, not only because it is more effective in sealing the structure but also provides easier access for inspection.

A flat cover system constructed of aluminum or FRP would be recommended to contain and control odors for the primary settling tanks. The cost for this type of containment however is relatively high. The primary sludge degritting is a strong odor source that could be addressed in combination with the odor control system for the primary settling tanks.

Odor control at the primary settling tanks could be considered as a second phase improvement and could be grouped with the other sources in the area to provide a cost effective solution. Recommended treatment technologies for the primary settling tanks would include biofiltration and wet scrubbing.

Activated Sludge Process

The aeration and final settling tanks were not considered to be major odor sources. Although it is less common, some treatment facilities have found it necessary to cover aeration basins for odor control. Some facilities have also covered final settling tanks, but typically for protection against freezing. For the Torrington WPCF, any odor reducing benefits from covering the aeration and final settling tanks would not appear justified at this time based on the associated costs and increased operational efforts.

Sludge Handling Facilities

The sludge handling facilities appear to be the largest sources of odor emissions at the plant. The WPCF staff has confirmed that these facilities occasionally cause off-site odors. Containment and treatment of these sources would appear to be the highest odor control priority.

The gravity thickener is an open tank and is considered a significant and consistent odor source throughout the year. Typically, plant water is added to the gravity thickener in an effort to maintain a water blanket and minimize odorous emissions. The continued use of the water blanket approach is recommended.

Containment and treatment of emissions from the gravity thickener, waste sludge holding tanks, thickened sludge holding tanks, and the truck loading area could be directed to a common odor control system, due to their proximity on the site. Flat cover systems should be evaluated for the gravity thickener and the waste sludge holding tanks. These covers should have a sufficient number of hinged hatches at appropriate locations to facilitate necessary maintenance activities. Dome type covers could also be used; however, domed covers would result in a higher exhaust air flow and a larger exhaust and treatment system. The ventilation rate under the covers of the gravity thickener and sludge storage tanks should be the greater of 12 air changes per hour, based on the volume above the maximum water surface or as necessary to maintain a negative pressure within the tanks. The emissions from sludge truck loading operations can be addressed by connecting an exhaust duct to the tanker vent prior to loading the trucks and venting this system through the odor control system.

Given the strength of the odors, appropriate control technologies to consider would include biofiltration, wet scrubbing, activated carbon, or iron sponge. Biofiltration would generally be favored for the types of strong odors generated by these sources. Furthermore, biofiltration may be more desirable than wet scrubbing from an operations perspective because of chemical handling issues.

There appears to be adequate space either to the north or west of the waste sludge holding tanks for either a conventional or proprietary biofilter. A conventional biofilter should utilize a minimum detention time of 60 seconds. Shorter detention times are possible for proprietary biofilters.

A wet scrubber would include a packed bed scrubber with a minimum packing depth of 10 feet. A two-stage configuration with the exhaust routed to above the roof line of the Operations Building would be the preferred design. A packed bed scrubber would use sodium hypochlorite as an oxidant and sodium hydroxide for pH adjustment. Activated carbon or iron sponge would be feasible control technologies, but are not expected to be cost-effective given the high strength and the provisions needed to address the high humidity of the exhaust air.

The gravity belt thickener is located in the operations building in a room that is ventilated via a two speed fan at either 900 or 1,800 cfm. This corresponds to greater than 6 and 12 air changes per hour respectively. This ventilation capacity is considered sufficient. In fact, a typical design would provide for lower air change rates due to the expense of heating in the winter.

Typically, fresh secondary sludge has a relatively mild odor and problem working conditions are uncommon. The Torrington WPCA has recently experienced high odors during thickening operations due to the storage and decanting operation as well as from the aerobic digestion trials. To control odors, an essential oil formulation called Evane Scent from Hinsilblon Laboratories is sprayed into the thickening room to improve working conditions. The existing essential oil system would be considered adequate in most cases for gravity belt thickener handling operations.

The FOG Receiving Facility was constructed in one of the former sludge holding tanks and generally located adjacent to the sludge thickening operations. The FOG Facility currently includes a small carbon style canister to aid in mitigating odors from this area. The recommended odor control system for the sludge handling facilities could also be designed to accommodate treating any air from the FOG Facility.

For thickening operations, both chemical addition and exhaust air treatment facilities are often installed to aid with odor control. Options for chemical addition could include peroxide, permanganate, or ferric chloride. It may be desirable to proceed with pilot testing of chemical addition alternatives as part of the decision-making process. Exhaust air treatment options include biofiltration, wet scrubbing, activated carbon or iron sponge. A biofilter would be suitable and could be located on the east side of the operations building adjacent to the thickener room. In this case, activated carbon or an iron sponge would likely be acceptable alternatives, because high sulfide levels would be expected to occur infrequently and the heated air stream will be less susceptible to humidity problems.

9.2.3 **Recommended Odor Control Improvements**

Odor control improvements should be implemented as part of overall WPCF improvements or in separate phases. If done in phases, the necessary odor containment, ventilation and treatment system should be evaluated during the preliminary design phase of each project. Because recent evaluations have indicated that the use of a proprietary biofiltration system can have similar or lower life-cycle costs to conventional wet scrubbers, and because biofiltration may be more appropriate for specific unit processes, the cost for providing proprietary biofiltration systems has been carried in the development of the recommended improvements. Specifically, it is anticipated that three biofiltration systems would ultimately be constructed for the following unit processes:

- Preliminary Treatment Area (including septage receiving, siphon structure, and Screenings Building).
- Primary Treatment Area (including primary clarifiers, influent and effluent distribution structures, and primary sludge degritting). Containment of odors at the primary clarifiers could include either covering influent and effluent weirs or covering the entire tanks. As part of the planning process, it is assumed that only the influent and effluent weirs would be covered. If additional containment is necessary in the future, the WPCF could cover the surface of the entire clarifiers and ventilate through the odor control system.
- Solids Handling Area (including gravity thickener, sludge storage tanks, grease-receiving, and sludge truck loading areas).

9.3 STAFFING

The Torrington WPCF represents a significant investment by ratepayers and proper operation is the direct responsibility of plant personnel. As regulatory requirements increase, the sophistication of wastewater treatment processes and equipment increase as well. It is important that sufficient qualified personnel be provided for the efficient operation and maintenance of the plant. Flexibility and some degree of overlapping of duties are necessary for efficient operation.

The Torrington WPCF is currently staffed by a **total of** 13 full-time employees with various levels of responsibility and expertise. There is also one part-time clerical worker. Future staffing requirements were developed using the methodology found in EPA published criteria. The EPA developed this methodology based on information obtained by visits to 35 sewage treatment plants across the country and by information supplied by regional offices of EPA, trade organizations, and water pollution control agencies. This EPA method of estimating staffing requirements covers treatment plants with capacities form 0.5 mgd to 25 mgd, using primary, secondary, and advanced treatment processes. The first step in estimating staffing requirements for Torrington is developed from a "Table of Adjustment for Local Conditions". This table is initially set up for an "average" plant, and then factors for adjusting staffing needs specific for the Torrington WPCF are applied. The adjustment factors established for Torrington are presented in **Table 9-3**.

TABLE 9-3

CITY OF TORRINGTON WPCF FACILITIES PLAN

ESTIMATED STAFFING REQUIREMENTS

ADJUSTMENT FOR LOCAL CONDITIONS

Local Condition	Comment	Operation	Maintenance	Supervisory	Clerical	Lab.	Yard
Plant Layout	Average	---	---	---	---	---	---
Unit Processes (1)	Standard, Diff.	---	10%	---	---	---	---
Level of Treatment	Advanced	10%	-15%	2%	2%	5%	---
Removal Requirement	% and Limit	5%	---	---	---	10%	---
Industrial Wastes	Some	5%	---	---	---	5%	---
Productivity	Average	---	---	---	---	---	---
Climate	Severe Winters	---	10%	---	---	---	5%
Training	Some Certified	-5%	---	-10%	---	---	---
Auto Monitoring	Monitor only	---	---	---	---	---	---
Auto Sampling	Inf., Pr. Eff, Eff	-5%	--	--	--	-5%	---
Off-Plant Laboratory	Some Analysis	---	---	---	---	-25%	---
Off-Plant Maintenance	Corrective Only	---	-25%	---	---	---	---
Age of Equipment	Old & New / Cared for	---	5%	---	---	---	---
Storm, Infiltration	Some	---	---	---	---	---	---
Present Flow	Less than Design	---	---	---	---	---	---
TOTAL		**10%**	**-15%**	**-8%**	**2%**	**-10%**	**5%**

Notes: (1) Unit Processes are standard with several variations in manufacturers.

The second step is to estimate annual staff-hour needs based on curves provided in the EPA manual. Annual projections for supervisory, clerical, laboratory and yard work are made on the basis of plant design capacity only. Operation and maintenance projections are on the basis of both plant design capacity and types of unit processes. The average treatment plant employee was estimated to work 1,313 "active" hours per year. This estimate was based on a 5-day work week, an average of 58 days for holidays, vacations and sick leave, and 6-1/2 hours per day of "active" work. Estimated annual hours are presented in **Table 9-4**. The total hours for each job classification are then adjusted based on the Local Conditions Adjustment Factors presented in **Table 9-3**.

TABLE 9-4

CITY OF TORRINGTON WPCF FACILITIES PLAN

ESTIMATED STAFFING REQUIREMENTS - ANNUAL ACTIVITY HOURS

Unit Processes	Operation	Maintenance	Supervisory	Clerical	Lab.	Yard
Collection System	---	3,750				
Off-Site Pumping	---	2,000				
Screenings	650	35				
Grit	580	52				
Primary Clarifier	800	440				
Gravity Thickener	310	310				
Aeration / BNR Process	8,250	940				
Secondary Clarifier	1,800	380				
Solids Handling/Disposal	1,040	300				
Disinfection	300	360				
Grease Handling	1,040	260				
Septage Management	260	260				
SUBTOTAL	15,030	9,087	2,800	570	2,800	2,800
ADJ FACTOR (%)	10%	-15%	-8%	2%	-10%	5%
TOTAL	**16,533**	**7,724**	**2,576**	**581**	**2,520**	**2,940**

The estimated hours presented in **Table 9-4** are generally based on curves in the EPA manual with the following exceptions:

- Collection System & Off-Site Pumping - The EPA manual does not cover collection system staffing. This was estimated as four full-time employees for collection system maintenance plus approximately ½-hour per day, 5 days per week for each of the fourteen pump stations.
- Primary Settling Tanks - The EPA estimating method includes hours for manual primary sludge pumping. Because the sludge pumping at the Torrington WPCF is performed automatically, the estimated hours from the EPA manual were decreased by one-half.
- Grease Handling - The grease handling process was estimated to require 4 hours per day of operations time and one hour per day of maintenance time, 5 days per week.
- Solids Handling/Disposal - Liquid sludge handling and disposal is not thoroughly covered in the EPA manual. Actual sludge disposal requirements were calculated as 4 hours per day of operation (5 days per week) and 2 hours per day of maintenance time, at 3 days per week.
- Septage Handling - The operator time required at the septage handling/receiving station was estimated to be 1 hour per day for operation and one hour per day for maintenance, 5 days per week on average.

The estimated staff-hours presented in **Table 9-4** are then developed into staff needs. The EPA methodology utilized provides for plants to be staffed 24 hours per day. The EPA data uses a typical staffing pattern of weekday nighttime staff equal to one third of the weekday daytime staff and weekend staff equal to one third of the weekday staff. Because the Torrington facility is not staffed on weeknights, and is only staffed part-time on weekends, appropriate deletions from the staffing projections were made.

A summary of the theoretical staffing requirements for the Torrington WPCF is shown in **Table 9-5**. The total "equivalent" weekday estimated staffing requirements for the Torrington WPCF is approximately 16 people, including four full-time collection system staff, one-part time clerical worker, and no weeknight staff. This assumes that the equivalent remaining weekend staffing needs would be provided during the week days. If no allowance is included for the weekend equivalent hours, then total weekday staffing needs are 14 people. These estimates are based on no additional laboratory analyses being conducted on-site.

TABLE 9-5

CITY OF TORRINGTON WPCF FACILITIES PLAN

ESTIMATED STAFFING REQUIREMENTS - ANNUAL STAFF HOURS

Staff Classification	Hours	Equivalent Staff	Weekend Staff	Weeknight Staff	Required Staff
Operation	16,533	12.6	3.1	2.1	7.4
Maintenance	7,724	5.9	1.5	1.1	3.3
Supervisory	2,576	2.0	0.5	0.4	1.1
Clerical	581	0.4	0.1	0.1	0.2
Laboratory	2,520	1.9	0.5	0.4	1.1
Yard	2,940	2.2	0.6	0.4	1.3
TOTAL	**32,874**	**25.0**	**6.3**	**4.7**	**14.0**
Total Theoretical Staff:		25.0			
Adjustment for Weekend Staff:		-4.3	(1)		
Adjustment for Weeknight Staff:		-4.7	(2)		
TOTAL WEEKDAY STAFF:		**16**			

Notes: (1) Facility is currently staffed with two part-time employees on the weekend
(2) No staff is anticipated at this time

The Torrington WPCF currently has a staff of 13, including one part-time clerical person. Based on the staffing analysis, the WPCA could consider hiring up to three additional operational staff, if necessary.

9.4 BUILDING SYSTEMS

The buildings and associated systems were evaluated to determine their general condition and improvements that should be implemented. Most of the structures at the WPCF were constructed during the 1970s addition of secondary treatment and sludge processing facilities. The Chemical Building and secondary pump galleries were constructed during the 1994 upgrade and the garage building was completed in 2000. Some of the structures originally constructed with the primary treatment plant in 1935, are still in use today.

The buildings and tanks located at the WPCF were evaluated for structural integrity and for general condition and compliance with electrical and ventilation codes. In general, all the facilities were found to be well maintained and in good condition. Although most tanks were in service and could not be drained for a detailed inspection.

9.4.1 General Building System Observations

The evaluation identified several general issues that should be addressed as described below.

Roof Drainage

The roof design, materials, and drainage systems vary from building to building. This is the result of buildings being constructed during different time periods and various roof repairs completed over time. Design deficiencies that were observed in several locations include roof drains that are piped to the building exterior and discharge at grade. Given the site limitations, this arrangement may be necessary, but the leaders are prone to ice blockage. Additionally, several gutters discharge water onto drives and walkways. In some locations, residential style gutters have been installed to redirect the flow of water.

Drainage from the roofs is further hampered by both the limited number of roof drains and clogged roof drains. Building code requires that all roof structures be designed to withstand the maximum possible depth of ponded water. When this code requirement is applied to buildings with roof parapet walls, the depth of ponding can amount to several feet should the roof drains become clogged. Therefore, buildings with roof parapets should have a secondary drainage system in case the roof drains become clogged. Simple overflow scuppers installed in the parapet walls would address this need.

Moisture in Exterior Walls

The exterior of many of the masonry walls were provided with weep holes to allow moisture to travel out of walls. It was observed that these weep holes were plugged by mud in several locations. The suspected origin of the mud is nesting insects. These weep holes should be cleaned. Future scheduled building maintenance should include inspections to ensure these holes remain cleared.

Efflorescence was observed on the masonry walls. This efflorescence indicates moisture moving through the walls. To protect the exterior walls as well as the internal structural components of the wall it is recommended that the exterior masonry walls be cleaned and damp proofed with a penetrating masonry sealer.

Electrical Systems

From the initial observations, the electrical distribution equipment appears to be in good condition. The condition of unseen components such as buswork is unknown. Some of the existing distribution and motor control equipment pre-dates the 1990s upgrade and is obsolete. Conduit and wiring for power, control, instrumentation signals, and the fire alarm system appear to be properly separated according to applicable codes. The conduit systems in the facility generally appear to be in good condition.

Ventilation and Plumbing

The ventilation and plumbing was generally found to be in good condition. A few areas were noted as having inoperable exhaust fans. Appropriate repair or replacement of these fans is necessary. The ventilation rate in some areas should be confirmed to be in compliance with relevant codes. Gas detection equipment is installed only in the screenings building. It may be beneficial to install gas detection equipment in other areas of the WPCF, particularly enclosed areas with limited egress such as the primary and secondary pump galleries.

The original exterior doors in many of the buildings were louvered to introduce fresh air into the ventilation systems. This likely resulted in significant heat loss and the louvers have since been covered. The ventilation rates as well as odor generation in these spaces may warrant new controlled intake louvers.

9.4.2 Detailed Building System Observations

Each building and treatment unit was inspected during a site visit. The conditions observed were recorded as well as comments on areas needing improvements. Comments from the WPCF staff on the need for additional space for the laboratory, shop/maintenance, and storage are also considered.

Screenings Building

The Screenings Building was constructed as part of the 1970 upgrade. An addition to house the screenings container room was completed as part of the 1994 improvements. It appears that at the time of the container room addition a new Irma type roof was installed on the entire building. Roof insulation beginning to separate from the roof deck was observed indicating loss of adhesion.

The roof of the original section of the screenings building is constructed of steel beams and light weight concrete roof planks. The steel roof beams appear to be in good shape. The concrete planks are beginning to show signs of deterioration, as indicated by spalling on the underside. This type of deterioration is likely to be accompanied by exposed wire reinforcing in the moist interior environment.

The electrical distribution equipment for this building includes the original motor control center with motor starters added as part of the 1994 upgrade. This equipment appears to be in good condition. The electrical room in this building is very small, and no additional space is available for future equipment. Any future process upgrades in this building may require an expansion of the electrical room to provide additional space for new electrical equipment.

The electrical equipment in the screening area appears to be properly rated (explosion proof) for a Class I, Division 1, Group D hazardous area. New fire alarm, intercom, and instrumentation signal conduits, installed in this building during the 1994 upgrade, enter through the electrical room. The building is equipped with a hazardous gas detection system. The gas detection system however is inoperable. Some of the lighting fixtures appear to be older fixtures, and some fixtures were not functioning during the site visit.

The roof exhaust fan in the screenings room appears to be inoperable. The capacity is not known. The electric cabinet unit heaters are in fair condition. The outer casings on the units are rusted and four of the units do not operate off the thermostat. The roof exhaust fan and the heaters in the screenings room should be upgraded as part of the recommend process improvements for the Screening Building.

The container room is ventilated by a two-speed exhaust fan rated at 500/1000 cfm, which equates to 12 and 24 air changes per hour, respectively. This fan is in good condition and operates in excess of the recommended number of air changes per hour. The electric heaters in this space are in good condition.

Other observations include steel framing that should be cleaned and repainted, and lack of emergency lights and illuminated exit signs. Additionally, the originally installed windows were not insulated, and the steel lintel is showing signs of corrosion.

<u>Grit Building / Primary Pump Gallery</u>

The grit building was originally constructed as part of the 1970 upgrade and received minor renovations during the 1994 improvements. The renovations included enclosing the egress stair, new doors, and mechanical improvements. A new fully adhered EPDM roof has been installed since the 1994 upgrade. A single roof drain is provided with no secondary provisions for drainage. Installation of a scupper to provide overflow drainage is recommended. The grit building is generally in good condition. However, the original construction did not include control joints and the expansion of the masonry has caused the concrete to crack at the corners of the building.

With the exception of the overhead door, the electrical equipment in the grit room does not appear to be rated for a Class I, Division 1, Group D hazardous area, as required by code. The overhead door is properly rated. The electrical equipment should be upgraded to an appropriate rating or relocated to a non-rated space.

The electrical equipment for the primary pump gallery is located below grade in the pump gallery. This equipment includes motor control centers (MCC), variable frequency drives (VFD), lighting panels and other miscellaneous items. Various items are rated for different conditions including NEMA 1, NEMA 4, and NEMA 4X. There does not appear to be adequate space to add any future sections to the existing MCCs or to add future variable frequency drives. The electrical equipment is in generally good condition, with much of it installed in the 1994 upgrade.

Light fixtures on both levels appear to have been installed at various times and several of the fixtures were not functioning at the time of the site visit. The light fixtures should be replaced with more energy efficient units. The fire alarm and intercom systems in the building date from the 1994 upgrade, and appear operable.

The ventilation system exhaust fan in the grit room was not operational. This system is rated at 1,000 cfm, equating to approximately 15 air changes per hour. The electric heater in this area was operational and in good condition.

The heating and ventilating systems serving the primary pump gallery were operational and in good condition. The exhaust fan associated with this ventilation system is rated for 1,000 cfm. This provides approximately 8 air changes per hour which is sufficient for this area. The primary pump gallery is a high humidity area. The installation of a dehumidifier in the pump gallery is recommended.

<u>Administration Building</u>

The Administration Building is the oldest building in the facility. Minor repairs and installation of a new fire alarm and intercom system were installed as part of the 1994 plant improvements. The building is in excellent condition for its age. While the building has been used for a variety of purposes including administration and housing the chlorine gas systems, it is currently used for the laboratory, storage and to house the hypochlorite system.

The mechanical systems in this building are operational and in good condition; however, it was noted the system is aged and beyond its useful life. The existing boiler was installed during the original building construction in 1935. The entire system should be evaluated during preliminary design phase and likely requires replacement. It was also noted that water leaks onto the floor from the pressure and temperature relief valve on the electric water heater located in the basement. The discoloration of the floor may indicate that this is an ongoing problem. In addition, the sump pump in the basement and the unit heater fan in the sodium hypochlorite room, are both operational but are old and should be replaced prior to failure.

Portions of the existing Administration Building are currently underutilized and could easily be rearranged to accommodate updated laboratory facilities, storage and administration space.

Operations Building

The Operations Building was constructed as part of the 1970 plant upgrade with significant renovations completed during the 1994 improvements. This masonry building was one of several around the plant that is in need of cleaning and sealing of the exterior walls. Weep holes were found to be clogged and efflorescence was observed along the base of the walls. Pressure washing of the walls followed by an application of a penetrating sealer is recommended. Additionally, control joints should be re-caulked.

Inspection of the roof revealed several conditions that, when considered together, may warrant the replacement of the roof. These conditions were: several large patches in the penthouse roof, at least one large wrinkle in the penthouse roof; loose insulation screws that can damage the EPDM membrane; and ponding of water on the roof because of high roof drains.

In addition, the basement lacks illuminated exit signs, which is a code requirement. An additional potential egress hazard is water on the basement floor at the access to the circular stairs. A second potential building code violation that was noted is that the boiler is not enclosed in a fire rated space.

Paint pealing from the basement walls and floor was observed. This is typical of below-grade concrete surfaces. Because of the effort required to maintain the paint on these surfaces, cleaning without refinishing is recommended. An investigation to determine if the paint is lead based was not conducted.

The majority of electrical equipment in the Operations Building is located in the basement, both in the electrical room and in the process area. The original plant electrical service equipment was upgraded during the 1994 plant improvements. The main service equipment is G.E. Spectra Switchboard, rated at 2,000 amps at 480 volts, three phase. Also located in the electrical room are two older Motor Control Centers and one newer Motor Control Center. The mix of equipment pre-dating the most recent facility upgrade and equipment installed as part of the upgrade appears to be in good condition. The electrical room is filled with equipment leaving little if any space available to add equipment in the future. The electrical equipment located in the basement process area was found to be in good condition.

An emergency generator and automatic transfer switch are also located in the basement. Both the generator and transfer switch pre-date the most recent facility upgrade, but appear to be in good condition. The generator is rated 475 kW, 594 kVA, 480 volts, three phase. The capacity of the generator is greater than the total connected continuous load. The generator however must be able to handle the load to start the connected motors. This "in-rush" can be significantly greater than the continuous requirements to keep the motors running. Although operational provisions are in place, fluctuation in load requirements and dynamic load characteristics may result in the generator not being capable of handling the load. As part of any recommended improvements to the facility, consideration should be given to the requirements and capabilities of the generator due to the addition of connected loads.

Fuel for the generator is stored in a double-walled underground fuel storage tank. A full monitoring system of the interstitial space for both fuel and ground water leaks brings this tank into compliance with the Connecticut DEEP's current requirements for leak detection of underground fuel storage tanks. Compliance with the March, 2002 air emissions regulations requires documentation of the generator's run time and volume of fuel purchased to substantiate a run time of less than 500 hours per year and/or less than 21,000 gallons of fuel.

The heating system for this building appears to be in good condition; however it is nearly 26 years old and approaching its useful life. In addition, the WPCF staff reported that the air conditioning system does not adequately cool the area served. The system should be evaluated during the preliminary design phase, especially with regard to the potential reorganization of spaces within the building. It is likely the system will need to be replaced.

Aeration Tanks

As part of a structural assessment, Aeration Tanks No. 3 and Chlorine Contact Tank No. 1 were inspected in the Fall of 2002. The structural assessment was conducted due to Torrington’s concerns with the quality of the aeration tank concrete and the existing coating applied to the interior of Chlorine Contact Tank No. 1. The findings and recommendations of the assessment were detailed in a Technical Memorandum. A copy of this Technical Memorandum is presented in Appendix F.

In general, Aeration Tank No. 3 is in good condition. The interior walls appeared "sandy" indicating loss of some surface mortar. There was no significant pitting of the concrete surfaces, but exposed aggregate was noted below the water line. Several hairline cracks have formed extending up from the base of the wall ranging from 4 to 16 feet in length. Most of these cracks were dry at the time of the inspection indicating that they do not extend all the way through the wall or that they have sealed up.

Test core samples were taken to determine the in-place compressive strength of the concrete. Four core samples were taken in Aeration Tank No. 3. The compressive strength test results of the samples indicate an in-place strength ranging from 4,930 to 5,730 psi, exceeding the design compressive strength of 4,000 psi. Based on the test results, it appears unlikely that the deterioration is the result of substandard concrete.

The inspected tank requires minor repairs to seal the cracks and prevent leakage and corrosion of the wall reinforcing. It is assumed that the findings would be representative of all the concrete tanks. Therefore, during routine maintenance, the tanks should be drained, cleaned, and inspected for cracks and deterioration. The deficits noted are considered to be minor. The recommended repairs should be completed within the next three years. Additionally, an inspection program to monitor the deterioration of all tanks should be implemented and the tanks inspected at intervals of three to five years.

Secondary Pump Gallery No. 1

Part of Secondary Pump Gallery No. 1 existed before the 1994 upgrade. As part of the upgrade, the pump gallery was extended and an electrical room was built above the new portion of the gallery.

All of the electrical equipment for the pump gallery is located in the electrical room and was completely replaced as part of the upgrade. A fire alarm and intercom system were also installed as part of the upgrade.

Secondary Pump Gallery No. 1

A significant volume of water enters the lower level by way of pull boxes. This water is most likely coming into the pull boxes from underground conduits. This water has caused extensive corrosion of the pull boxes and adjacent areas. Other wall mounted equipment in other parts of the pump gallery is corroded due to water leaking down walls. The receptacles in the secondary pump gallery are not the weatherproof type. Repairs or other provisions should be made to seal the conduits, in order to stop this water source and eliminate any hazard and to minimize additional corrosion. Another approach would be to provide new junction/pull boxes above-grade. This approach would keep underground conduits from draining into the below-grade pull boxes.

The electrical room heating and ventilation systems are rated at 1,000 cfm, equating to approximately 6 air changes per hour. The systems functioned normally at the time of the inspection. NFPA 820, Fire Protection in Wastewater Treatment and Collection Systems, require below-grade pump rooms to be continuously vented at a rate of 6 air changes per hour to de-rate the space electrical classification from Class I, Division 2 to Unclassified. The existing system complies with this requirement only if run continuously. The thermostat controlling the electrical unit heater in this space is in poor condition and should be replaced with a NEMA 4X thermostat.

Secondary Pump Gallery No. 2

Secondary Pump Gallery No. 2 was constructed as part of the 1994 upgrade. The pump gallery is below-grade and the associated electrical room is located above the gallery. The conditions of the exterior masonry walls are typical of other structures at the WPCF. The walls have hairline cracks which show evidence of prior leakage. Observations made during various site visits noted that the cracks were not damp. Cleaning of weep holes and repair of cracks due to expansion is needed. Signs of leakage were also noted at the control joints and base of the wall between final settling tanks and the stairwell, as well as the pipe line seals to the tanks.

Roof drains that originally discharged onto the concrete deck over the pump gallery have been rerouted off the roof slab using residential gutter. This rain leader should be re-piped.

All of the electrical equipment, and heating and ventilating systems in the pump gallery and the electrical room were found to be in good condition.

Final Settling Tanks

Final Settling Tanks No. 1, No. 2, and No. 3 are not typically used but are in good condition. Final Settling Tanks No. 4 and No. 5 were constructed as part of the 1994 upgrade and the tanks appear structurally sound, and the associated electrical equipment appears to be in good condition. Final Settling Tanks No. 4 and No. 5 have an overflow structure on the interior of the wall that is connected to the tank groundwater drain system. To accommodate the future addition of an algae sweep system, this overflow structure will have to be removed. A preliminary assessment indicates that this can be done with no structural impact on the tanks.

Chemical Building

The Chemical Building was constructed as part of the upgrade in 1994. The building is generally in good condition. Originally, several chemical systems were contained in the building. Currently it is used to house the dechlorination system and a polymer system. The building has several rooms with much of the space either unused or used as storage.

The heating system in the dechlorination room is functional, and appears to be in good condition. The ventilation system exhaust fan is rated for 750 cfm, equating to 12 air changes per hour. The outside air intake damper is inoperable. The status of the intake damper may pose a potential code violation. The heating and ventilating systems associated with the polymer room are in good condition and are operating.

The electrical equipment, located in a dedicated room appears to be in good condition. The electrical room is small and is presently filled with equipment. A change in use of all or part of this building may require an expansion of the electrical room to provide space for new equipment. The ventilation system in this room is in good condition.

An electric water heater located in a storage room was observed to have a leaking drain valve. The electric unit heaters in the dechlorination room are in good condition and are operational.

Chlorine Contact Tank

Chlorine Contact Tank No. 1 was constructed as part of the 1970 plant upgrade. The tank was coated with a membrane waterproofing material called Decothane in 2000. This material began to blister and delaminate approximately one year after installation. Repairs were attempted with Flexideck 2000 which is a urethane coating. Neither of these products appears to be applicable to submerged applications. While the material has provided some degree of protection for the concrete, removal of the coating is recommended. Recoating of the tank interior is not considered to be necessary although some minor surface repairs similar in nature to those recommended for the aeration tanks should be completed within the next three years. Core samples of Chlorine Contact Tank No. 1 were tested and determined to be of 4,000 psi concrete.

Chlorine Contact Tank Coating Failure

Concrete repairs to the contact tank should include routing out of the cracks and injection with an epoxy or urethane based compound. The surface should be sealed with a trowel grade hydraulic cement material. Pitting and spalling should be repaired with polymer modified cement.

The inspection of the Chlorine Contact Tanks was completed at the same time as the inspection of Aeration Tank No. 3. The details of the inspection and recommended materials for remediation are presented in a Technical Memorandum included in Appendix F.

<u>Greenhouse</u>

The greenhouse is an aluminum framed structure with fiberglass siding that was originally used for sludge drying. The building does not have a floor and is unheated. The structure is currently used for equipment storage, household hazardous waste, and other cold storage.

<u>Maintenance Facilities</u>

The current plant maintenance facilities consist of a 4-½ bay garage on the south end of the Operations Building. Approximately half of the facility is used as a shop with two bays housing vehicles. The following items are needed to accommodate the staff's plans for future pump station maintenance at the WPCF:

- A higher ceiling to accommodate a hoist and large pieces of equipment, such as the effluent pumps which are approximately 12 feet tall.
- A trolley hoist to facilitate loading and working on large pieces of equipment.
- Additional space to provide a total of approximately 50 feet by 50 feet.
- A pump test bench.
- Assorted tools with appropriate storage.
- Space for several vehicles.

The garage is heated by horizontal hot water unit heaters. The layout and capacity of the units will need to be evaluated when a new garage layout is developed as part of the preliminary design process.

Site Water Systems

The potable water main comes onto the site towards the northeast corner of the WPCF through a metering vault. The plant personnel indicated the desire to have a pressure reducing valve installed in the vault to lower the water pressure throughout the facility. Additionally, a remote meter reading system is recommended so that entry into the vault is not required for every meter reading. The vault was flooded the day of the site visit. Reportedly, this is a common occurrence.

Plant effluent water is used for flushing, cleaning, preparation of chemical solutions, aeration tank and final settling tank spray systems, and for gravity belt thickener belt washing. Effluent water is withdrawn from the chlorine contact tanks by Effluent Water Pumps No. 1 and No. 2 (EW-1 and EW-2), located in Secondary Pump Gallery No. 1. As part of the 1994 plant upgrade, new effluent water pumps were provided to increase the system capacity.

Lifts and Hoists

The operations personnel indicated the importance of having a hoist at the effluent pumps to facilitate their removal for maintenance. This is also the case at several of the pump stations where heavy equipment must be removed and transported to the shop at the treatment plant. The WPCA may want to consider purchasing a boom truck to meet these needs.

9.5 PUMPING STATION SECURITY EVALUATIONS

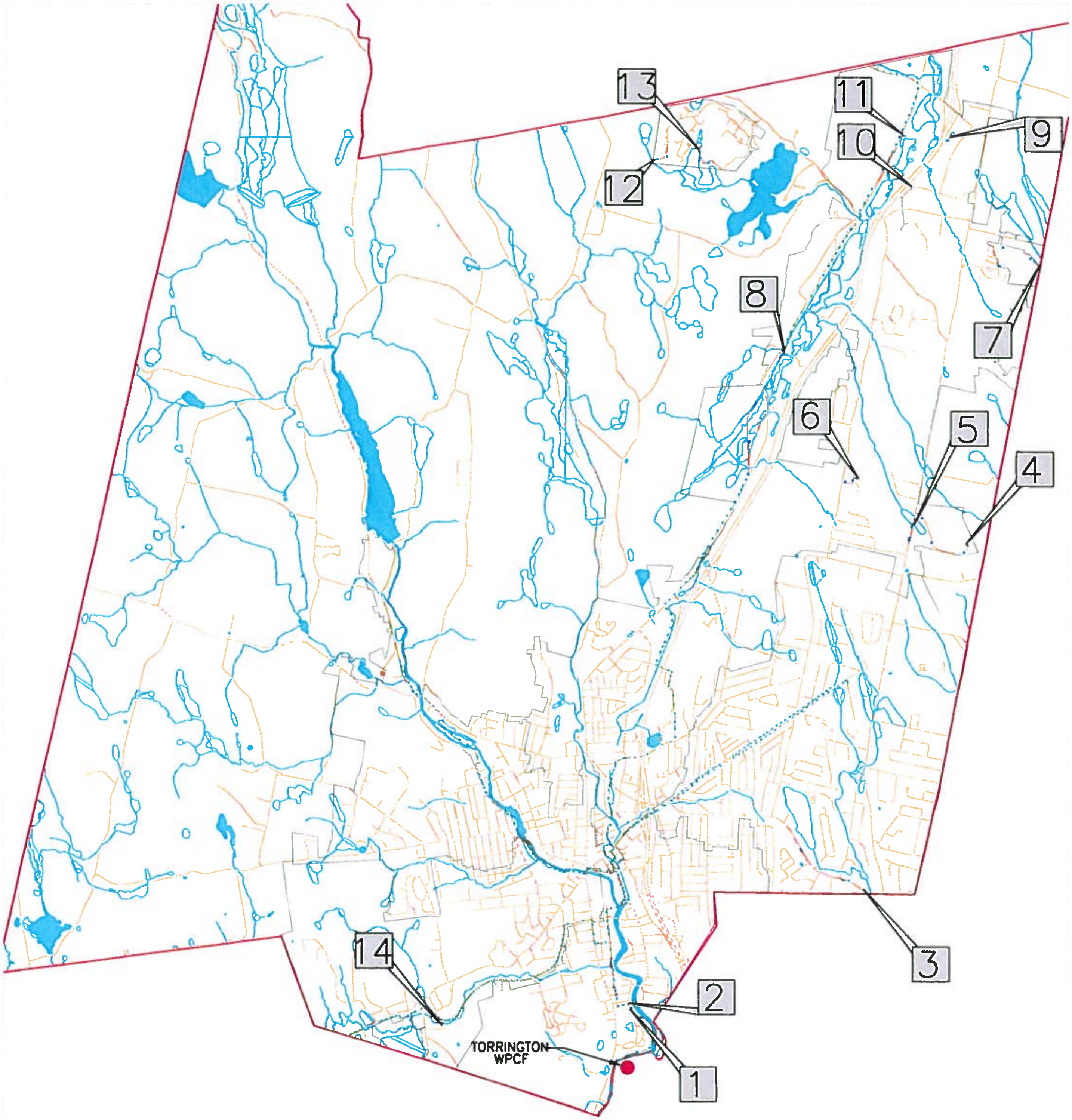
In addition to the operation and maintenance of the treatment plant, the Torrington WPCA is also responsible for approximately 230 miles of sanitary sewer and several wastewater pumping stations. Currently, the Torrington WPCA is responsible for the operation and maintenance of fourteen pumping stations. A map of the fourteen pumping stations is presented in **Figure 9-1**.

As part of the facilities study, field evaluations were conducted at each of the foureen pumping stations. The evaluations placed an emphasis on security issues relating to the adequacy of fencing, gating, lighting, security alarms, and overall aesthetics. An evaluation of the age, condition or capacity of the pump stations was not part of the scope of this study. Only the stations with security related recommendations are discussed herein and are summarized below.

Tara Drive Pumping Station

The Tara Drive Pumping Station is located on Torringford West Street. The pumping station serves approximately 15 homes on Tara Drive. Access to the station is down a paved driveway. There is not sufficient room to facilitate vehicles turning around. The below-grade wetwell is located at the end of the driveway in a swampy area and is equipped with a padlock on the access hatch.

The electrical and control panels are out in the open with no protection from the elements or vandals. Additionally, no site lighting is provided at this station. The only security alarm provided at this station is a tamper alarm installed on the access cover of the Remote Terminal Unit (RTU). Recently, a preliminary survey was performed to eliminate this station and run a gravity sewer line to Eastwood Road. This is being further evaluated by the City.



LEGEND	
1. WILLOWBROOK P.S.	8. HARRIS DRIVE P.S.
2. KING STREET P.S.	9. INDUSTRIAL LANE P.S.
3. NEW HARWINTON RD P.S.	10. ELLA T. GRASSO AVE. P.S.
4. MIDDLE SCHOOL P.S.	11. WINSTED ROAD P.S.
5. TORRINGFORD STREET P.S.	12. CLIFF SIDE DRIVE P.S.
6. CINNAMON RIDGE P.S.	13. EVERGREEN DRIVE P.S.
7. TORRINGFORD FARMS P.S.	14. FELICITY LANE P.S.

CITY OF TORRINGTON
TORRINGTON PUMP STATION
LOCATION

WRIGHT-PIERCE
Engineering a Better Environment

PROJ NO:	12411A
DATE:	JUNE 2012
SCALE:	NTS
FIGURE:	9-1

Cinnamon Ridge Pumping Station

The Cinnamon Ridge Pumping Station is located on Clove Court. The pumping station serves approximately 40 homes and an industrial facility (FuelCell Energy, Inc.). Vehicle access to the station is down an approximately 1,000 foot easement path through an access gate adjacent to the property at 26 Clove Street. The path consists of dirt and gravel. The access gate becomes very difficult to open in the wintertime when there is a heavy snowfall, impeding its operation.

Cinnamon Ridge Pumping Station

The below-grade wetwell is equipped with a padlock on the access hatch. There is no exterior lighting at the station. All electrical equipment is housed in a pedestal mounted electrical enclosure. The only security alarm provided at the station is a tamper alarm installed on the access cover of the RTU. The station is visible from adjacent backyards and is in need of aesthetic improvements.

Torringford Street Pumping Station

The Torringford Street Pumping Station is located on Torringford Street. The station serves approximately 10 homes. Access to the station is from a paved driveway off of Torringford Street. The paved driveway provides adequate room to turn a vehicle around. The pump station is visible from neighboring residences and provisions should be taken to make the station's appearance consistent with its surroundings.

A sodium hypochlorite chemical feed facility is located adjacent to the pump station in a temporary aluminum enclosure. There is no lighting at this station and the electrical and control panels are out in the open with no protection. The electrical enclosures and wetwell hatch to the pump station are equipped with padlocks and there is a tamper alarm installed on the access cover of the RTU.

Torringford Farms Pumping Station

The Torringford Farms Pumping Station is located deep in the woods. Access to this station is down a long dirt road accessible next to 300 Cedar Lane. There has never been any vandalism at, or concerns related to the aesthetics of, this station. This is generally attributable to its remote location far back in the woods. However, there is currently no lighting at the pump station site. The pumping station wetwell is located outside of an approximately 12 foot by 20 foot precast concrete building.

Torringford Farms Pumping Station

The interior of the building is well lit and houses an emergency generator, pump controls, and other associated electrical appurtenances. The building is equipped with an entry alarm and the wetwell is provided with a padlock.

Ella Grasso Drive Pumping Station

The Ella Grasso Drive Pumping Station is located on Ella Grasso Drive adjacent to the Burlington Building. Access to the pumping station is down a paved driveway with adequate room to turn around.

Ella Grasso Pumping Station

The station wetwell is located inside a fenced area with a locked gate, adjacent to a small precast concrete building. There is no locking mechanism on the wetwell hatch. The interior of the building is well lit and houses an emergency generator, pump controls, other associated electrical appurtenances. Exterior lighting at the station is operated by photocells that appear to operate reasonably well but reportedly have lost their photo-sensitivity over time. The building is equipped with an entry alarm.

Industrial Lane Pumping Station

The Industrial Lane Pumping Station is set back in the woods off of Industrial Lane next to McCoy Ltd. Access to the pumping station is through an aluminum gate and down a paved driveway. There is adequate room to turn a vehicle around.

Industrial Lane Pumping Station

The station wetwell is located inside a fenced area with a locked gate, adjacent to a small precast concrete building. There is no locking mechanism on the wetwell hatch. The exterior posts of the fence, as well as the fence itself, have lifted out of the ground as much as two feet due to frost heave. This allows easy access for anyone to enter the fenced area of the pumping station. The interior of the building is well lit and houses an emergency generator, pump controls, and other associated electrical appurtenances. Exterior lighting at the station is operated by photocells that operate reasonably well but reportedly have lost their photo-sensitivity over the years. The building is equipped with an entry alarm.

Winsted Road Pumping Station

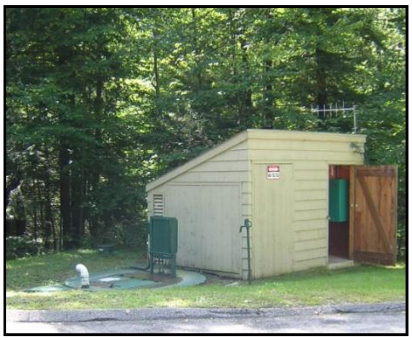
Winsted Road Pumping Station

The Winsted Road Pumping Station is located on Winsted Road across the street from Southwords Wayside Furniture. There is very little space available for parking at this station.

The Winsted Road Pump Station consists of a wetwell and an electrical pedestal enclosure with an alarm located inside of fenced area. The wetwell hatch is equipped with a padlock and the electrical cabinet is equipped with a drop light. There is no exterior lighting provided at this station. The access gate to this pump station is approximately 10 feet off the edge of the road. The rollers of the gate reportedly freeze up in the winter time during a snowstorm when the plow trucks push the snow from Winsted Road up against the moving parts of the gate.

Cliffside Drive and Evergreen Drive Pumping Stations

The Cliffside Drive and Evergreen Drive Pumping Stations serve over 425 individual units in the Lakeridge Condominium Complex. The City of Torrington is responsible for the operation and maintenance of the stations, including all associated costs. The LCA is responsible for the overall aesthetics of the two pumping stations. There are few security concerns related to these two stations. Both pump stations are on private property that is frequently patrolled by security guards employed by the LCA.



Cliffside Drive Pumping Station

Evergreen Drive Pumping Station

Both the Cliffside Drive and Evergreen Drive Pumping Stations consist of wooden sheds housing an emergency generator, pump controls, and all other associated electrical appurtenances. The wet wells at both stations are located outside of the buildings and are equipped with padlocks. There is no fencing provided at either one of the stations.

Both structures are equipped with adequate interior lighting as well as a building intrusion alarm; however, neither of the pumping stations is equipped with any exterior lighting. Yard hydrants are locate adjacent to the pumping station buildings. There are no padlocks provided on the yard hydrants. Additionally, the yard hydrant at Evergreen Drive is approximately 5 feet above grade and appears to be loosely mounted in the soil.

Access to the Cliffside Drive Pumping Station is from the side of the road. Access to the Evergreen Pumping Station is down a dirt driveway in a very wet area. Swampy and muddy conditions reportedly make it very difficult to get the vacuum truck down the driveway to clean the wetwell. Plowing snow in the winter time is difficult. Both stations are equipped with a propane tank that supplies fuel to the emergency generator. The propane tank at

Cliffside Drive is buried below grade. The propane tank at Evergreen Drive is above grade on the side of the building adjacent to the road.

King Street Pumping Station

The King Street Pumping Station is located on the corner of King and River Streets. Access to the station is down a paved driveway, through a gated fence. There is adequate room for vehicle turn around.

King Street Pumping Station

The building at this pumping station is constructed of block with a brick façade and houses all of the pump controls and associated electrical appurtenances. There is no emergency generator at this pump station, but the station is set up to run on the treatment plant's portable generator in the event of a power failure. The building is equipped with an intrusion alarm and there is adequate lighting inside of the structure. The exterior lighting of the pump station building is controlled by photo cells in addition to one switch operated light. The wetwell is located outside of the building and is equipped with a padlock.

Willowbrook Pumping Station

The Willowbrook Pumping Station serves approximately 30 buildings in the Willowbrook Condominium Complex. Access to the station is off of River Street through a gated entrance and down a long dirt road that is in poor condition.

Willowbrook Pumping Station

The station consists of a locked wetwell hatch at-grade, a small electrical pedestal equipped with a tamper alarm installed on the access panel to the RTU, and an emergency generator mounted outside. There is no building or fence at this pumping station. All of the associated equipment is set back in the woods behind the Willowbrook Condominiums. The access road and driveway provide little to no room to turn a vehicle around and there is no place to plow the snow to in the wintertime. There is no site lighting provided.

Conclusions of Security Evaluation

Access and security to the pumping stations was generally good; however, a few deficiencies were consistently noted.

- Lighting and/or lighting controls at several stations needed to be replaced.
- All station locks should be keyed to one master key.
- Locks and intrusion alarms were not present on several electrical enclosures or RTUs and should be provided.
- The appearances of several stations could be improved with the installation of wooden fences and/or shrubbery, also aiding in providing a secure site.
- Bollards or fencing to protect electrical equipment and pump control panels should be present at all locations.
- Access to all stations should be made as easy as possible

Of the fourteen pump stations in the Torrington WPCA service area, vandalism has not been reported to be a significant problem. Eight of the fourteen pump stations are enclosed in some type of building with an intrusion alarm. All of the stations protected by a chain link fence were equipped with a padlocked access gate. Eleven of the pump stations should be provided with new exterior lighting fixtures. The fixtures should be mounted on a pole or on the building, depending on the actual location and configuration of the station. The new lights should be operated off of a switch and/or by a dusk-till-dawn or motion detecting photo cell. All pump station buildings and RTU's equipped with an intrusion or tamper alarm are functional and are sent out through a pager system.

The majority of the pumping station locks and doors are all keyed to one master key. The pumping stations with locks that are not keyed to the master should have the lockset replaced with a compatible set. The evaluation led to aesthetic concerns only for those stations visible from roadways or residential areas. Several of the stations are located in the wooded areas, and there are no concerns about aesthetics at these stations. Those stations that are located in residential areas should be provided with fences and shrubbery to blend with the neighborhood.

In addition to lighting and security issues, the City should consider implementing a formal pumping station maintenance program. Such a program would require scheduled maintenance checks on all mechanical and electrical equipment as well as requirements to maintain the physical appearance of the pump stations, particularly those located in residential areas. Emergency generators should be run every few weeks with fuel reserves replenished as necessary. **Table 9-6** presents a summary of recommended improvements to each of the affected pump stations. The estimated cost to implement the recommended improvements is approximately $170,000. This cost estimate is based on a complete capital improvement contract. Because many of the recommendations are minor, substantial cost savings could be achieved if the improvements were made over time, as part of the normal pump station operations and maintenance program.

TABLE 9-6
CITY OF TORRINGTON WPCF FACILITIES PLAN
PUMP STATION SECURITY - RECOMMENDED IMPROVEMENTS

Pump Station	Recommended Rehabilitation
Tara Drive	Provide one new exterior lighting fixture and controls Provide concrete bollard in front of electrical pedestal Provide tamper alarm on electrical panels
Cinnamon Ridge	Provide one new exterior lighting fixture and controls Provide wooden fencing and shrubbery around station Provide tamper alarm on electrical panel
Torringford Street	Provide one new exterior lighting fixture and controls Provide wooden fencing and shrubbery around station Provide a permanent chemical feed facility Provide concrete bollard in front of electrical pedestal Provide tamper alarm on electrical panel

Torringford Farms	Provide one new exterior lighting fixture and controls
Ella Grasso Drive	Provide two new exterior lighting fixtures and controls Provide a locking mechanism on the wetwell access hatch
Industrial Lane	Provide two new exterior lighting fixtures and controls Provide a locking mechanism on the wetwell access hatch Reset fence posts to bring fence around station to grade
Winsted Road	Provide one new exterior lighting fixture and controls Provide one additional single leaf access gate to the station
Cliffside Drive	Provide one new exterior lighting fixture and controls Provide a locking mechanism on the yard hydrant
Evergreen Drive	Provide two new exterior lighting fixtures and controls Provide a locking mechanism on the wetwell yard hydrant Reset yard hydrant in the soil Move propane fuel tank to rear of building Provide compacted stone over the access drive
King Street	Provide one new exterior lighting fixture and controls
Willowbrook	Provide one new exterior lighting fixture and controls Provide access from Willowbrook Condominiums Provide tamper alarm on electrical panel

9.6 PLANT SECURITY EVALUATIONS

The Torrington WPCF is a significant public asset. In addition to the financial investment, the facility serves to protect public health and the environment. The protection of this asset therefore should be a daily priority.

Wastewater treatment plants have unique security concerns. The threats that can affect their operation include natural disasters, operational procedural errors, disruption of services, theft, and vandalism. To address these unique concerns, a specific vulnerability assessment methodology has been developed by the industry. The Vulnerability Self Assessment Tool (VSAT) is a program developed by the Association of Metropolitan Sewerage Agencies

(AMSA). VSAT provides a structured approach for the evaluation of vulnerabilities and identifying capital improvements, processes, and procedures to reduce security risks. A complete facility vulnerability assessment is beyond the scope of the Facilities Planning Study. As part of the facility study, however, an initial assessment of the treatment plant was conducted.

The initial assessment was limited primarily to plant security to protect against intrusions and other unauthorized entry or activities. The treatment plant includes several buildings and pump rooms. Generally these buildings and pump rooms are unlocked and unmonitored. Some of the areas, such as the chemical storage, are not easily visible from the operations building. Lighting is generally adequate for the buildings, but is more a function of safety rather than security.

The facility is enclosed by a perimeter fence. The fence is in good condition and generally free of visual obstructions on the interior. However, the gate is left open during normal operation hours. This plant “openness” is required to a certain extent to allow for access by sludge and septage haulers. The treatment plant also provides a household drop-site recycling facility for City of Torrington residents, thereby requiring accessibility throughout the day.

A comprehensive vulnerability assessment of the treatment plant is recommended. The vulnerability assessment should evaluate all aspects of facility threats including protection of assets, safety, retention of the plant knowledge base (record drawings, standard operating procedures, O&M manuals, and other business critical documents), and natural and intentional threats. More immediate improvements and procedural changes can be implemented to address some of the basic security issues. Recommended security improvements include the following:

- Provide sufficient security lighting throughout the plant, at all buildings, and possibly along the perimeter.

- Maintain locking mechanisms on all buildings. The locking mechanism could include a keypad or card access system so entrance by authorized personal is not hindered.
- Provide site access control. The access gate could be remotely controlled to permit access by visitors only by a plant employee. Access cards or PIN code controllers could also be provided to septage, FOG, and sludge haulers, permitting controlled access.
- Provide CCTV. Closed circuit television could be provided to permit monitoring of sensitive areas and areas not easily visible, such as the front gate and chemical storage room.

SECTION 10

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SECTION 10

ENERGY EVALUATION

10.1 INTRODUCTION

An energy evaluation of the Torrington WPCF and selected wastewater pump stations was conducted in order to assess the current energy use at the facility and identify opportunities for energy cost savings, efficiency and renewable energy applications. This section of the report will serve to summarize the results of energy efficiency and renewable energy evaluations and alternatives assessments performed for selected pump stations and the WPCF facility. The evaluation included an energy audit of the WPCF, which was performed through the following tasks:

- A review of the energy usage of the facilities through electrical bills.
- Site visits and on-site testing of flow, head and energy use of various equipment and systems to determine the quantity of energy being utilized in various parts of the facility.
- Development of an energy balance to justify current energy use and costs.
- Calculation of energy cost savings through various operational and equipment modifications.

10.2 CURRENT ENERGY USE

To determine the current energy use and relative cost of the existing WPCF and wastewater pump stations, a review of the electrical costs for 2010 was performed. The 2010 annual energy use and costs are presented in **Table 10-1**; the information is shown graphically in **Figure 10-1,** summarizing the relative electrical cost of each facility.

TABLE 10-1

2010 TORRINGTON WPCF AND PUMP STATION

ELECTRICAL USAGE AND COSTS

Location	Annual Usage (kWh)	Annual Cost	Average Cost / kWh*
Water Pollution Control Facility	2,164,320	$319,480	$0.15
New Harwinton Road	91,200	$17,589	$0.19
Harris Drive	66,624	$13,294	$0.20
Ella Grasso	21,565	$4,169	$0.19
T-Farms/Cedar Lane	19,360	$3,418	$0.18
Industrial Lane	9,206	$2,041	$0.22
Cinnamon Ridge**	8,125	$1,867	$0.23
Torringford Street **	8,019	$1,541	$0.19
King Street	5,879	$1,436	$0.24
Winsted Road **	3,209	$895	$0.28
Felicity Lane **	1,884	$786	$0.42
Willowbrook ***			
Cliffside ***			
Evergreen ***			
Total	2,399,391	$366,517	

* Cost/kWh is average combined cost for generation and delivery

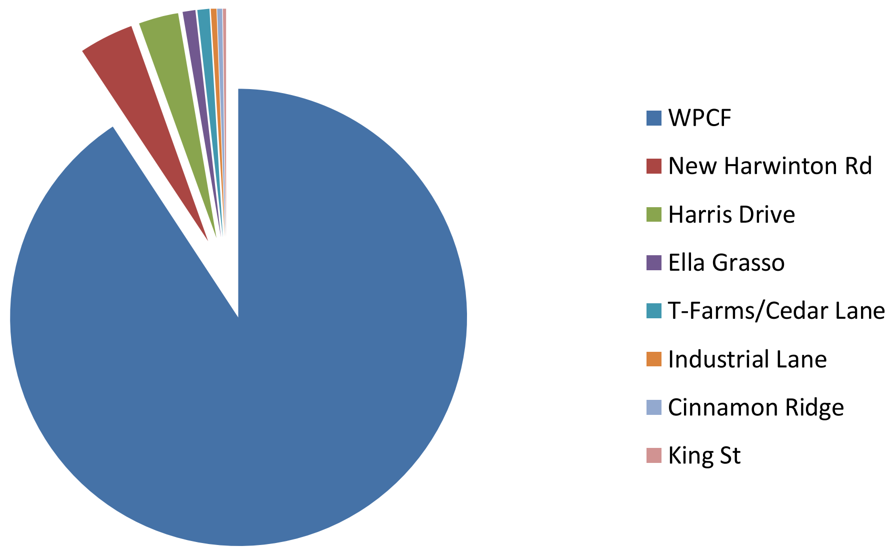
**Not visited on December 6, 2011

*** Utilities not paid by WPCA

FIGURE 10-1

TORRINGTON WPCA

2010 PERCENTAGE ELECTRICAL USAGE AND COSTS

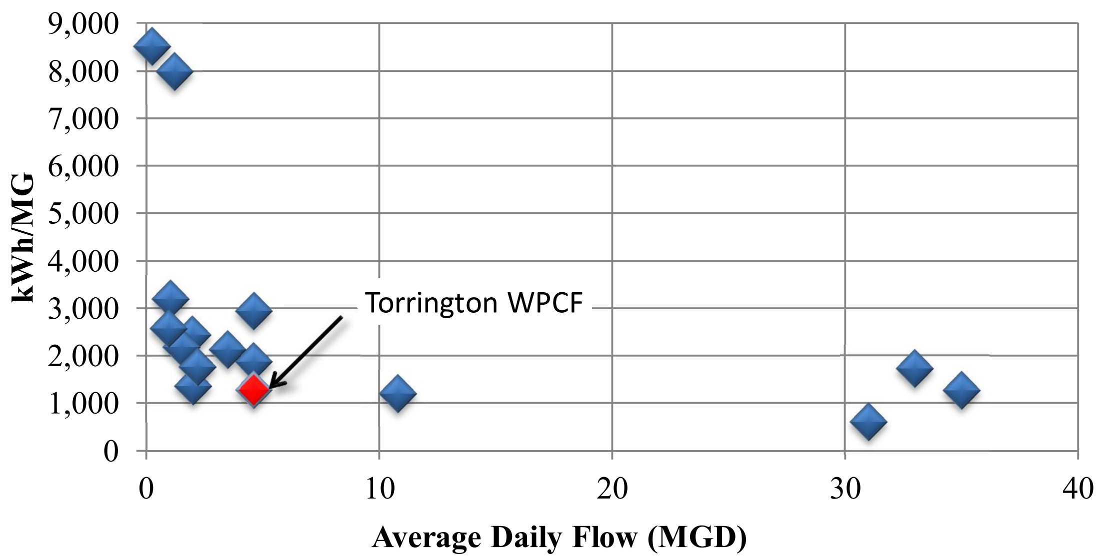


10.3 BENCHMARKING

The Torrington Water Pollution Control Facility can be compared to other wastewater treatment facilities in the southern New England area based on energy usage and gallons treated. Based on facility data, the WPCF treats, on average, approximately 4.61 million gallons per day, and treats a total of approximately 1,685 million gallons per year. Based on the electrical energy usage presented above, the plant consumes approximately 1,285 kWh per million gallons treated. Irrespective of the specific treatment processes utilized at the facilities, this energy usage is relatively low when compared to the typical range for regional wastewater treatment facilities of similar size, as shown in **Figure 10-2**. This comparison suggests that the Torrington WPCF is relatively energy efficient when compared to similar sized treatment facilities.

FIGURE 10-2

TORRINGTON WPCF BENCHMARK PERFORMANCE



Data is based on information provided by Torrington WPCF as well as previously evaluated facilities.

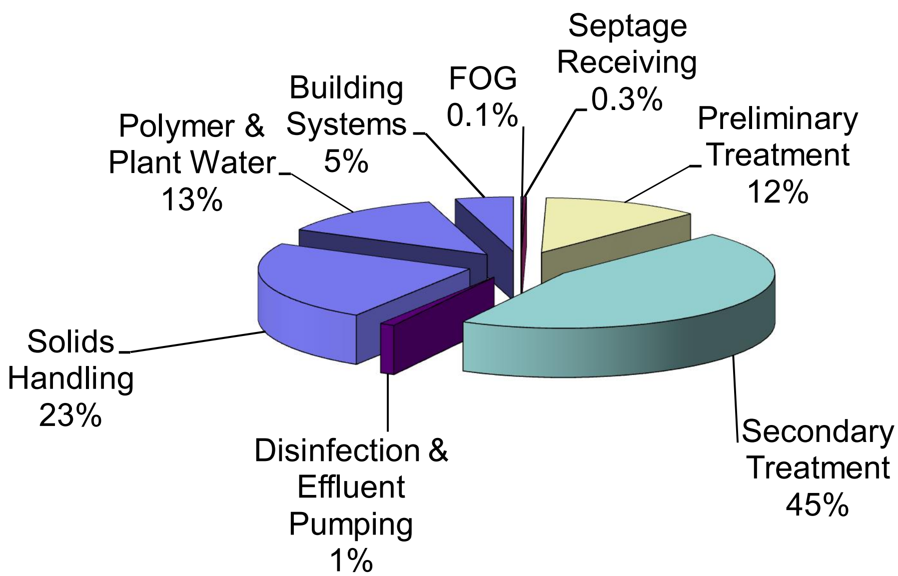
10.4 FACILITY ENERGY BALANCE

In order to identify where energy is being used at the WPCF and determine the most significant areas for efficiency opportunities an energy balance was prepared; **Figure 10-3** presents a graphical representation of this. A more detailed breakdown of the energy use by plant system is provided in the Energy Evaluation Report, included in Appendix H.

This energy balance assists in identifying the plant systems that have the greatest opportunity for improvement, and also indicates where energy conservation would have the most significant impact. Based on this analysis, Secondary Treatment accounts for almost half (46%) of the electrical energy utilized at the WPCF. Solids Handling accounts for the next largest demand, at 23%. Preliminary Treatment accounts for 12% while Polymer & Plant Water Systems account for approximately 13%. The remainder of the processes account for approximately 6%.

FIGURE 10-3

TORRINGTON WPCF ELECTRICAL ENERGY BREAKDOWN



10.5 OPERATION AND ENERGY CONSERVATION SUMMARY

An energy audit and investigation was completed for the existing equipment located at the Torrington WPCF and select pump stations. The recommended Operation and Energy Conservation Measures (OM's and ECM's) are summarized in **Table 10-2**; a detailed description of the OM's and ECM's is provided in the Energy Evaluation Report, included in Appendix H.

TABLE 10-2

2010 TORRINGTON WPCF AND PUMP STATION RECOMMENDED OPERATION AND ENERGY CONSERVATION MEASURES

	Cost Saving Measures	Annual Energy Savings (kWh)	First Year Annual Savings ($)	Initial Cost* ($)	Simple Payback (yrs)
	OPERATION MEASURES				
OM 1	Establish Pump Station Energy Monitoring Program	-	-	$0	Immediate
OM 2	Electrical Supply Rate	-	$55,000	$0	~ 1 year
OM 3	Demand Control	251	$1,592	$0	Immediate
OM 4	Modify Plant Water Pump Operations	57,093	$8,564	$50,600	5.9
	ENERGY CONSERVATION MEASURES				
ECM 1	Pump Station Heat Controls	-	-	$250/PS	Immediate
ECM 2	Harris Drive PS Modifications	18,835	$3,767	$25,300	6.7
ECM 3	New Harwinton Road PS Pump Modifications				
ECM 4	Rebuild RAS Pumps & Install New Motors	21,506	$3,226	$17,600	5.5
ECM 5	Modify Internal Recycle Pump Operations	4,108	$616	$7,425	12.1
ECM 6	Aeration Blower Replacement	259,359	$38,904	$392,150	10.1
	Potential Energy Program Cost and Savings	**357,044**	**$111,053**	**$486,650**	**-**

10.5.1 OM # 1 – Establish Pump Station Energy Monitoring Program

The critical parameters in determining the energy performance of the individual pump stations includes demand (kW), usage (kWh), operating hours, and gallons pumped. Several strategies can be implemented that would allow for on-going monitoring of the performance of each pump station and could to improve the energy performance of the overall system.

It is recommended that the monthly energy bills be evaluated, and the following parameters be recorded for each pump station location:

- Demand (kW)
- Total Usage (kWh)
- Supply Cost, Delivery Cost and Total Cost (all, $/ kWh)

Additionally it is recommended that the following data be collected from each pump station on a monthly basis:

- Run Time (Hours)
- Pressure Reading
- Flow Reading

Collecting and monitoring these parameters will help to identify overall trends in energy usage at each pump station, as well as potential billing anomalies or errors. Comparing monthly pump efficiencies, kWh, etc., will provide staff with a more long term view of the pump stations functionality than a one-time data collection visit can provide.

It is recommended that this information be combined in spreadsheet format to calculate:

- kWh per million gallon pumped
- Cost per million gallon pumped
- Estimate pump efficiencies

Recording and monitoring this data at each location provides the opportunity to continually assess the energy performance and pumping performance of each station. It will also aid in identifying the locations that are consistently the most expensive to operate, as well as giving a benchmark of the overall system performance on a monthly basis. Tabulating this data on a regular basis provides an opportunity to schedule pump maintenance and/or replacement when increased costs/gallons pumped or decreased pump efficiencies deem it feasible.

Therefore, it's recommended that an energy monitoring system be implemented for each pump station. This monitoring system would be useful in tracking overall pump efficiencies trends that can be helpful in determining when it is cost efficient to rebuild or replace a pump motor.

10.5.2 OM #2 – Electrical Supply Rate

Torrington's wastewater system currently utilizes two companies (Constellation and Dominion) to supply energy to the WPCF and various pump stations; distribution is provided to all facilities by CL&P. Constellation supplies energy to the WPCF while both Constellation and Dominion supply energy to the pump stations. Supply costs from the fiscal year 2010–2011 were analyzed. Based on this evaluation it is estimated the Torrington facility could achieve an approximate cost savings of $55,000 per year. Refer to Appendix H to review a detail breakdown of this evaluation. It is recommended that the Torrington WPCA consider switching electrical generation contracts to CL&P once any current contracts expire, or discussing opportunities to renegotiate rates with the current suppliers.

10.5.3 OM #3 – Demand Control

The Torrington WPCF is billed under CL&P's Rate Code 37 for electric delivery service. Included in this rate are two service rates: a Distribution Demand Charge and Production/Transmission Demand charge. The Distribution Demand Charge is based on the highest average 30-minute demand recorded during on-peak hours during the current billing month; the Production/Transmission Demand Charge is based on the highest average 30-minute demand recorded during the preceding 12 month period. CL&P defines on-peak hours as weekdays from noon to 8:00 p.m. (1:00 p.m. to 9:00 p.m. during Daylight Savings Time). The Production/Transmission Demand Charge is currently billed at $2.44/kW and is based on the highest average 30-minute demand recorded during the preceding 12 month period (during on or off peak hours).

The WPCF could achieve a demand reduction through onsite equipment management, such as re-scheduling unit processes to off-peak hours or managing when certain equipment is operating simultaneously.

10.5.4 OM #4 – Modify Plant Water Pump Operations

There are three 40 Hp constant speed Plant Water (PW) pumps. Typically one pump will operate continuously at a set pressure point of 105 psi. When the system pressure falls below 85 psi, a second pump will turn on; the third pump is a stand-by pump. Installation of a VFD and pressure control system could provide for energy and cost savings. These should be evaluated as an alternative during the Preliminary Design phase of the project.

7.1.1

10.5.5 ECM #1 – Pump Station Heat Controls

An energy usage profile indicates significant increases in pump station energy consumption during the winter months at some of the stations. This increase may be a result of a heater that was left running or another non-flow related electrical demand. Many of the collection system pump stations include electric heat, with manual thermostat controls. Programmable thermostats that would allow for minimum temperature settings and may provide an opportunity for energy and cost savings.

10.5.6 ECM #2 – Harris Drive Pump Station Pump and VFD Replacement

There are three 40 hP, 1175 rpm vertical shaft pumps, which operate on magnetic drives, within the Harris Drive pump station; staff report the pump speed is currently not adjustable The existing pumps were installed in the 1970's and appear to be running inefficiently. Rebuilding the existing pumps and installing new VFD's could to provide for energy and cost savings.

10.5.7 ECM #3 – New Harwinton Road Pump Station Pump Replacement and VFD Installation

There are three 40 hP, 1175 rpm constant speed pumps in the New Harwinton Road pump station. The existing pumps were installed in the 1970's and appear to not be running as efficiently as possible. A comprehensive upgrade was just completed in fall of 2012 and included the installation of VFD's.

10.5.8 ECM #4 – Rebuild/ Replace RAS Pumps

There are three 15 hP, 585 rpm pumps, with VFD's that are used to return activated sludge to the aeration tanks. The existing pumps do not appear to be running as efficiently as possible. Rebuilding the existing pumps or evaluation of replacing the pumps while maintaining the existing drives could provide for energy and cost savings.

10.5.9 ECM #5 – Replacement of Existing Centrifugal Blowers with Higher Efficiency Turbo Blowers

Air to the aeration trains is supplied by four, 200 horsepower blowers, one of which operates on a VFD. All the blowers are multi-stage centrifugal blowers and each has a throttling valve at its inlet. These valves are reportedly typically fully open, and are not automatically throttled to control airflow. The VFD blower is reportedly problematic and while use of it will reduce energy peaks, staff reports it has not been shown to provide any electrical cost savings. Since the treatment process at the facility is currently being evaluated as part of the Facilities Plan, it may be appropriate to replace one or more aeration blowers as part of a larger facility upgrade. Utilizing an automated throttling valve to match the required air flow or replacing the existing blowers with new high-efficiency turbo blowers appears to provide for energy and cost savings.

10.6 RENEWABLE ENERGY EVALUATION

Sustainability will be a central focus of the WPCF upgrade design, incorporating viable alternative energy opportunities and utilizing energy conscious and sustainable design and building practices. As part of the energy evaluation of Torrington's WPCF, a preliminary assessment of renewable energy alternatives was conducted in addition to the energy efficiency study detailed herein. The purpose of this conceptual evaluation was to identify viable renewable energy projects that provide environmental, economic and social benefits to the community.

Recent legislative changes have restructured the State's program for incentivizing renewable energy projects and are summarized below.

10.6.1 Geothermal Heating

Using geothermal energy to provide building and hot water heating is based on the concept of mining below-grade ground heat. It requires drilling to access the heat source and a recirculation system for the heat exchange medium. It is considered a viable technology as heat is present in the below-grade rock formations in all areas, although the below grade temperature and type of material can impact the efficiency of the system.

Shallow geothermal systems can be used for both heating and cooling that take advantage of the relatively constant temperature of the upper 10 ft of ground just below the surface (50-60°F). These systems consist of a geothermal heat pump, a system of buried pipes, a heat exchanger and heat exchange medium (water, antifreeze, or a mixture), and ductwork into the building. During the winter, heat from the warmer below-grade material is extracted for building heat. In the summer the heater exchanger removes heat from the building, transferring it to the cooler ground, or using it for hot water heating.

The cost of a geothermal system is several times that of a similarly sized traditional heating/cooling system, and requires significant subsurface work for installation of the piping. The ground loop is typically expected to last 50 years, with replacement of the above-grade equipment being required more frequently (20-30 years). Installation of this type of system also requires land area for installation of the in-ground pipe loop, which may be a limiting factor at the WPCF site. Assessment of geothermal heating applications requires an investigation of peak building heating and cooling demands, as well as an analysis of the soils and potential heat exchange efficiency of the subsurface material at the site. Horizontal or vertical systems can be installed, and heat exchange can be through either an open or closed loop system (groundwater versus a recirculating media). The payback period for a geothermal system can be expected to be at least 10 to 15 years, but is dependent on the type of system selected and as well as the site conditions and building requirements.

10.6.2 Hydropower

Small hydropower applications can be found at wastewater treatment facilities where significant head is available to drive a turbine and produce a relatively small amount of electrical power. The amount of power than can be produced is dependent on the vertical distance of the hydraulic elevation change and the volume of flow that can be captured through the turbine. There currently is not a location within the facility that appears to provide adequate head for this application. If possible, during the facility upgrade it may be prudent to consider if areas of sufficient head can be incorporated into the hydraulic profile. The following equation can be used for estimating potential power production:

$$\text{kW} = [\text{flow (gpm)} \times \text{head (ft)} \times \text{efficiency (typically 0.70)} \times 0.18 \text{ (constant)}] / 1{,}000$$

10.6.3 Solar

Solar photovoltaic (PV) systems can be a reliable, renewable energy source. There has been an increase in installation of PV technology in the northeast over the past several years as public and private agencies, and businesses look for opportunities to reduce their environmental impact and/or reduce electrical costs. A solar photovoltaic system at the WPCF, if feasible, would be connected to the grid so as to supplement and offset the incoming power required from the utility.

According to the U.S. Department of Energy's PV system calculator, the available solar energy in the Torrington area averages 4.2 kWh/day/m^2 of PV panel surface area under ideal conditions. Design considerations for solar installations include the site specific intensity and availability of the sun light exposure, available roof (or ground) space, and the structural capacity of the facility structures to support the weight of the panels. Space available for PV is limited by HVAC and other equipment that may be mounted on the roof of each building, and should be taken into account in determining the potential number of panels that could be installed on each building and the power production capacity.

It is typically more cost-effective to design new buildings to sustain the load of PV systems than to retrofit existing buildings; however, roof modifications are an option in some cases. Each of the existing buildings could be assessed to determine the strength of the roof and to identify any reinforcement that maybe required. Any new structures proposed as part of the planned facility upgrade should be considered for PV system installation.

Based on this preliminary analysis, installation of a solar PV system may be viable for the Torrington WPCF. However, prior to incorporating a solar PV installation at the facility, a more thorough analysis of the costs, electrical energy production potential, payback period, and financing options of PV installations on existing buildings as well as the proposed future buildings should be completed.

10.6.4 Wind Power

The number of wind turbine installations in the U.S. has been increasing, particularly over the past several years due to both improvements in the technology and the range of wind speeds that can be effectively utilized. There are several sizes of horizontal wind turbines currently available, ranging in height from 20 meters to 80 meters, and capacity from 7 kW to over 1,000 kW.

For an installation at the Torrington WPCF, an evaluation of turbines in the smaller range of these options is most appropriate. Location is critical for the viability of wind power as the wind speed determines the potential power production. Large scale wind turbines require wind speeds of 7 m/s or greater, while lower capacity units can produce power with as little as 5 m/s. To determine the true average wind speed and direction, meteorological monitoring is required and must be carried out at the specific location being considered. However, wind speeds can be approximated from available wind maps. These maps indicate low wind speeds for the Torrington area, averaging approximately 5.0 to 5.5 m/s, at 80 meters. Historically, these winds speeds would not produce enough power for a viable wind energy project. However newer technology may improve the viability of this project. Optiwind, a Torrington based company, states their technology can provide energy at wind speeds lower than traditionally necessary. Because local wind speeds can vary significantly, if Torrington is interested in investigating the opportunity for wind power generation, further site specific wind speeds can be determined through meteorological monitoring. Additionally, local zoning and siting regulations will need to be considered. Generally, this requires the temporary installation of a weather/atmospheric condition monitoring station or system. The data obtained from these systems can be used to perform site-specific analysis of wind energy generation potential.

10.7 INCENTIVES - ZREC

Recent legislative changes have restructured the State's program for incentivizing renewable energy projects. This section provides some background information on Zero-Emissions Renewable Energy Certificates (ZRECs) as well as interconnection requirements set by the electric utility company.

In the past, State incentives for renewable energy projects were based primarily on grants provided through the Connecticut Clean Energy Fund (CCEF). Under the establishment of the Department of Energy and Environmental Protection (DEEP) in 2011, the CCEF became the Clean Energy Finance and Investment Authority (CEFIA). Changes in agency structure have also resulted in a different approach to financing. Connecticut is currently in the process of transitioning from a grant based funding program to a Renewable Energy Certificate (REC) based incentive program model. Beginning this year, the Zero-Emission REC program will be the primary incentive funding source for Class 1 energy technologies, which includes solar PV, wind and hydro.

The ZREC program is managed by the electric distribution companies (EDCs) such as Connecticut Light and Power (CL&P) and United Illuminating (UI) and by the DEEP's Public Utility Regulatory Authority (PURA). Under the new regulations, the EDCs are required to allocate funding for the purchase of ZRECs from customers generating renewable energy. The customers, once deemed to be qualified bidders, will be able to sell their ZRECs to the EDCs for a fifteen (15) year contract period. The starting price will be capped at $350 per ZREC.

Power generating system categories include the following:

- Small (1 to 100 kW),

- Medium (>100 kW and < 250 kW), and
- Large (≥250 kW and ≤1,000 kW).

Generating systems are capped at 1,000 kW of ZRECs; a ZREC is earned for every 1,000 kWh of electricity the renewable energy system generates. Systems that qualify must be in the proposal phase or if installed, must be behind electric meters in operation after July 1, 2011. Projects which are in the proposal phase must be completed, permitted and in operation within one year after the contract award date.

CL&P and UI issued the first Request for Proposals (RFP) for the Low and Zero Emissions Renewable Energy Credit Program on May 1, 2012 and bids were due this past June. The RFP solicited proposals from generators of ZREC in the medium and large categories. Bidders were allowed to bid as high as $350 per ZREC and selected bids will enter into a 15 year contract with the EDC (CL&P and UI) at the fixed bid price.

The first RFP did not include solicitation for systems generating less than or equal to 100 kW. Small systems will be able to enroll in a tariff that should be opened by CL&P and UI shortly. This tariff will be available on a first-come, first served basis. The Small ZREC tariff will be based on the weighted average of the medium ZREC price (as determined by the bids) and 10% up to $350 per ZREC.

Additional RFP's for these ZRECs will be announced although no dates have yet been announced.

10.7.1 Financing Options

There are two options – municipal ownership and Power Purchase Agreement - by which Torrington could finance improvements through the use of ZRECs. For this discussion, it is assumed Torrington would want to finance a solar system.

Municipal Ownership

Under municipal ownership, the municipality would be responsible for acquiring the capital funding (i.e. issuance of bonds) necessary to purchase and install the proposed solar array. The municipality would own the solar system and the associated ZRECs acquired through the generation of kWhs. Under this scenario, the municipality would be responsible for maintaining and operating the system and for acquiring the necessary permits to build and operate it.

By owning the system, the municipality would see a direct offset in electrical consumption and an energy savings associated with the on-site generation of power. The system would be owned in perpetuity and would continue to provide power once the costs associated with the installation have been paid. However, a significant disadvantage involves the acquisition of funding for the project. In addition, the municipality would be responsible for operating and maintaining the system and for administering the sale of any acquired ZRECs.

Power Purchase Agreement / Third Party Ownership

A Power Purchase Agreement (PPA) is an alternative to municipal ownership in which the municipality becomes the host and the installer becomes the owner of the power system. In a PPA, the installing company owns the solar equipment and sells the electricity generated by the system to the municipality at a negotiated contract price. The installer is responsible for financing the project and for designing, installing, monitoring, operating and maintaining the system. The installer is also responsible for paying any property taxes associated with the system. Since installers are eligible to receive federal tax credits (30% for renewable energy projects), they can benefit from an additional incentive that is not accessible by municipalities. In addition, any associated ZRECs acquired through the operation of the system would be owned by the installer and not the host/municipality.

PPAs offer a number of advantages. First of all, the municipality would avoid acquiring any of the upfront costs necessary for the installation of the system. With the operation and maintenance of the system being the responsibility of the installer, the municipality would also avoid any of these costs. The municipality would see a savings based on the lower cost of electricity negotiated with the installer. At the end of the contract period, the host/municipality would have the option to buy the system at a negotiated price.

10.7.2 Interconnection Requirements

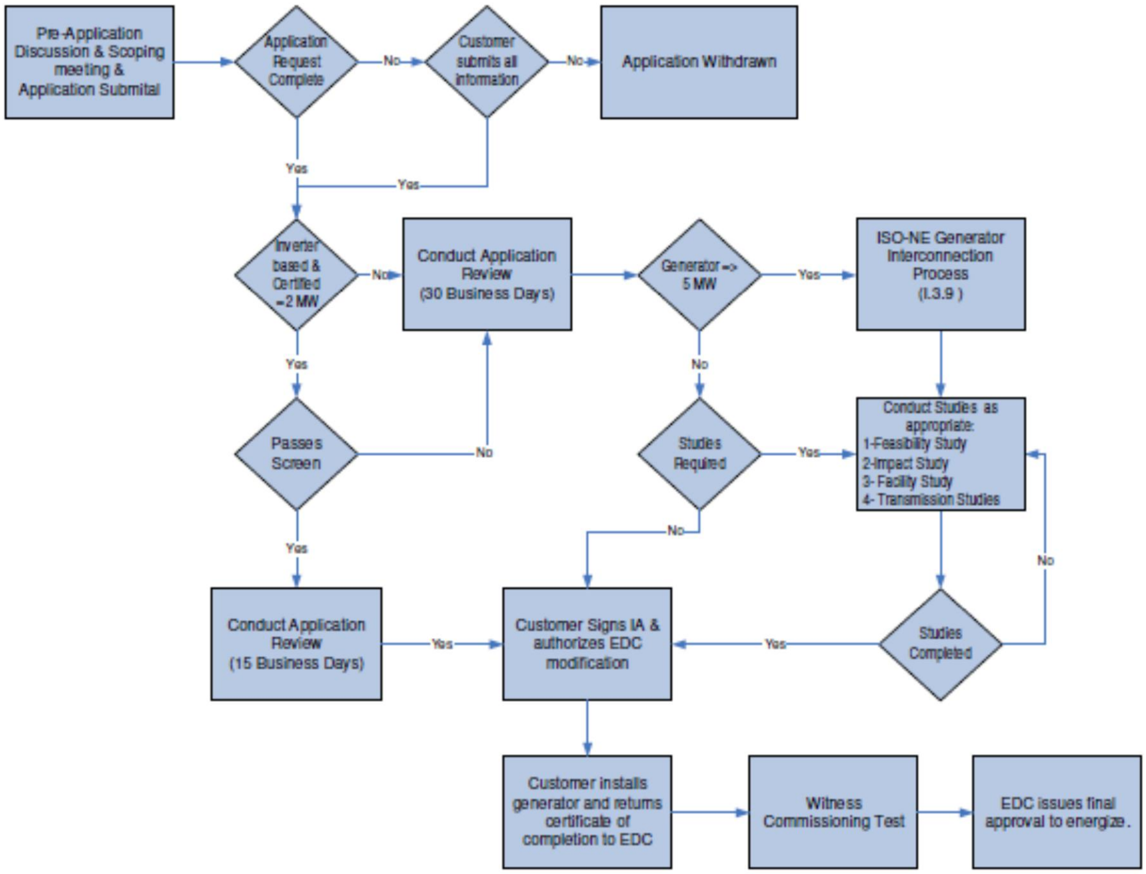
Connection to the existing power grid requires an Interconnection Agreement between the generator (owner of the power system) and the EDC. Certified inverter based generators larger than 10 kW qualify for the Fast Track Interconnection Process. Prior to submitting an Interconnection Request and prior to the purchase of any equipment associated with the generating system, it is recommended that the generator contact the EDC facilitator for an initial scoping meeting to discuss the proposed project and interconnection approach.

The application process involves a number of screening steps to determine the project's feasibility, safety, reliance and overall compliance with the EDC's interconnection design and legal requirements. An overview of the process is included as Figure 10-3 below. Key requirements of the Interconnection Application process include the following:

- The design of the proposed power generating system must comply with the EDC's technical requirements for the interconnection into the existing power system (per CL&P and UI Exhibit B Generator Interconnection Technical Requirements);
- The generator must provide proof of site control (i.e. ownership of site, leasehold interest in, or developing rights for the purpose of building a generating facility);
- Systems ranging in size between 100 kW and 1 MW must maintain general liability insurance of $1,000,000;
- The generator must pay for any necessary upgrades to the existing power grid resulting from the proposed interconnection;
- The EDC may require an Interconnection Study that assesses the feasibility of the project and the impact of the proposed power system to the existing power system;
- The generator must provide municipal approval of the proposed system;
- Commissioning of the system must be witnessed by the EDC;
- Upon the successful completion of the commissioning tests, the EDC will issue the final approval for the interconnection.

FIGURE 10-4

INTERCONNECTION AGREEMENT APPLICATION PROCESS



Source: Connecticut Light and Power Company and United Illuminating Company Guidelines for Generator Interconnection Fast Track and Study Processes, May 12, 2010.

10.8 GREEN DESIGN STANDARDS

In addition to the energy efficiency improvements and possible renewable energy technologies that can be incorporated into the treatment plant upgrade, the new and retrofitted facilities can also be designed using sustainable practices and incorporate applicable LEED design and construction standards. Some of the proposed green and LEED design principles that can be incorporated into this project include the following:

- Reusing existing buildings and structures can provide an economic benefit but also limits the environmental impact of the project. Upgrading the existing buildings wherever it is feasible will greatly reduce construction waste, as well as reduce expended energy and pollutants generated in the manufacturing and transportation of new materials. Existing building improvements should include improvements to the energy performance as well as water efficiency.

- Low emitting materials such as paints, coatings, wood and sealants can be used wherever possible.
- Stormwater management strategies that minimize run-off and water pollution can be implemented. More extensive methods such as a green roof and potential options for paved surfaces could also be assessed if Torrington desired to determine their applicability for this site.
- Minimize impervious areas where possible and feasible. This includes limiting pavement as well as minimizing building footprint and using building space in an efficient manner.
- Water efficient landscaping utilizing native plant species.
- Minimizing the use of potable water for any processes that do not require it, or replacing potable water with plant water supply when possible.
- New and renovated bathroom facilities, showers, break room, and lab can include high efficiency fixtures. This may include instantaneous hot water heaters if appropriate to meet the hot water demand.
- Maximize energy performance of new/retrofitted building envelope, HVAC systems, and lighting.
- Daylighting through use of skylights can be maintained and employed in new structures. Other options for daylighting can be investigated as part of the design effort to select appropriate alternatives for each building. New lighting controls can utilize occupancy sensors and HVAC systems can incorporate thermostats and adequate controls for providing efficient comfort.
- Minimize heating requirements and utilize heat recovery in ventilation systems.

These concepts can be included in the final structures and buildings, and can reduce the environmental impact of the facility over the long term.

The construction work itself can also be done in a sustainable manner, minimizing pollution and conserving resources. By including these standards in the construction documents the contractors will be required to employ these sustainable strategies as part of their work and in their purchase and procurement methods, creating benefit for both the local community and the environment. Some of the construction requirements that can be included in the final specifications include:

- Manage construction waste to maximize recycling, minimize landfill disposal, and improve opportunities to salvage materials.
- Allow for the use of salvaged or refurbished materials that are in acceptable condition, but do not require new resources.
- Use building materials with recycled content. Specific goals for the percentage of recycled content can be established.
- To the extent possible, incorporate materials and products that have been extracted, produced, or manufactured locally (within 500 miles of the site). Coordination of this requirement with the State's Clean Water Fund procurement requirements will be necessary.
- Incorporate materials that are considered rapidly renewable (i.e. specific types of wood). Require environmentally responsible wood products and consider species and harvesting technique.
- Manage indoor and outdoor air quality during construction by specifying low VOC materials (adhesives, paint, sealants, caulking), implementing dust control, controlling equipment exhaust, and avoiding contamination of porous material.

SECTION 11

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SECTION 11

RECOMMENDED PLAN

11.1 INTRODUCTION

An evaluation of alternatives for improvements to the Torrington WPCF is presented in the preceding sections. This section provides a discussion of the specific recommended improvements to the Torrington WPCF.

11.2 PROCESS OVERVIEW

The proposed site layout of the recommended facilities is presented in Figure 11-1. Generally, the recommended treatment process is similar to the current operation. Wastewater flow enters the plant through the existing siphon structure. The Screenings Building would be expanded and equipped with new mechanical bar screens and screenings grinder-washer-compactor units. The septage receiving facility storage tank discharge would be relocated to a point upstream of the screens. The septage facilities would be modified to improve and automate the operation.

From the Screenings Building, the wastewater would proceed to Distribution Box No. 1. The distribution box would be modified to provide better flow distribution to the primary settling tanks. A fourth primary settling tank would be constructed to improve operational performance and increase system redundancy. Primary effluent would flow to Aeration Tanks Nos. 3 and 4, and then from Aeration Tanks Nos. 3 and 4 to Aeration Tanks Nos. 1 and 2.

The activated sludge process would be modified to operate in a Four-Stage Bardenpho Process for improved nitrogen reduction. Anoxic zones would be created in the influent end of Aeration Tanks Nos. 3 and 4. The existing internal recycle pumps would be replaced with higher capacity units. A new pipeline would be constructed to transfer effluent flow from Aeration Tanks Nos. 3 and 4 to Aeration Tanks Nos. 1 and 2. The older aeration tanks, Aeration Tanks No. 1 & No. 2, would be modified to create the second half of the Four-Stage Bardenpho Process.

A Supplemental Carbon and Alkalinity adjustment Chemical feed system will also be needed as part of this system.

In addition to equipment upgrades within Aeration Tanks Nos. 1 and 2, hydraulic adjustments are needed within the two tanks. Due to the shallow depth and difficult operation associated with Final Settling Tanks No. 1, No. 2 and No. 3, it is recommended the three rectangular clarifiers be abandoned and the mixed liquor from Aeration Tanks No. 1 and No. 2 be directed to Distribution Box No. 5, which discharges flow to the 80-ft circular secondary clarifiers. There is currently three feet of freeboard available in Aeration Tanks No. 1 and No. 2 so that the effluent weir can be raised to provide additional head to allow the effluent from these tanks to flow to the Distribution Box No. 5.

A new circular final settling tank would be added to provide additional clarifier capacity to compensate for removing the rectangular final settling tanks from service. Modifications to the return activated sludge (RAS) piping system would be required to split RAS flow proportionally between each of the aeration tanks.

Effluent from the final settling tanks would continue to flow to a new tertiary treatment system which would provide phosphorus removal to meet the future seasonal permit requirements. It is recommended that a ballasted flocculation type tertiary process be installed. The existing Secondary Clarifier Nos. 1, 2 and 3 could be modified to install the required tankage and equipment needed for this process, including the required tanks for the tertiary treatment process, sludge pump room, chemical storage/feed equipment room and tertiary treatment influent pump station.

From the new tertiary treatment process flow would enter existing Chlorine Contact Tank No. 1. No process changes are recommended for the disinfection process. Effluent flow monitoring would be provided. During the preliminary design phase, consideration could be given to utilizing a Parshall Flume, an in-line magnetic flow meter, or modifying the effluent weir on the chlorine contact tank for effluent flow monitoring. The outfall pumps will be maintained for effluent discharge during high river elevations.

It is recommended the solids handling process be changed from a thickening process to a dewatering process. Primary and Tertiary sludge would be co-settled in the primary clarifiers and thickened in the existing gravity thickener. Secondary sludge would be stored in the existing sludge holding tanks adjacent to the FOG facility. The gravity belt thickener would be abandoned and replaced with screw press dewatering equipment, to be located within the exiting garage adjacent to the operations building. A portion of the auxiliary sludge holding tank would be converted into a blend tank, which would be used to blend stored secondary sludge and thickened primary/tertiary sludge from the gravity thickener. Flow from the blend tank would be transferred to the screw press equipment for dewatering. Extensive replacement and modifications to the sludge pumping/piping systems would be completed to improve the operation of the process. Odor control for this process would also be provided.

11.3 SUMMARY OF RECOMMENDED IMPROVEMENTS

A brief summary of the recommended improvements is presented below. A more detailed description of the recommended plan, including costs, is presented in subsequent sub-sections.

11.3.1 Screenings Building

- Provide a concrete spill containment wall for the Vactor truck dumping area upstream of the mechanical screens.
- Replace the existing mechanical bar screens with finer screening equipment that is more efficient and less susceptible to maintenance concerns.
- Install screenings handling (grinding, washing, dewatering, compacting, and disposal) equipment.
- Remove concrete flooring above the screen channels and install aluminum diamond plating with integral access panels.

- Expand the building to provide sufficient access to all equipment for operation and maintenance activities.

- Provide odor control for the headworks area and incorporate the Screenings Building and Septage Receiving Facility. Include covering and exhausting air from the Screenings Building, Septage Receiving Facility, Siphon Chamber, and Primary Sludge Degritting Facility and treating odorous air. At this time, a containerized biofilter system is recommended. However, during preliminary design, available alternatives should be considered and re-evaluated.

11.3.2 Septage Receiving Facility

- Install a stand-alone septage receiving pretreatment unit that would screen incoming spetage, and automatically meter and record the volume of septage from each hauler.

- Replace or modify the septage tank mixer to provide more efficient mixing.

- Relocate the septage pump piping discharge to a location upstream of the mechanical screens.

- Provide a new submersible chopper-type septage pump in the septage holding tank. Either leave the existing septage pump in place as a standby unit or purchase a second submersible chopper pump as a shelf spare.

11.3.3 Grit Facility

- No recommendations. The Huber grit removal system is relatively new and in good condition.

11.3.4 Primary Settling Tank

- Construct a fourth primary settling tank.

    - The configuration and dimensions of the new tank would replicate the existing tanks.
    - Effluent from the fourth primary settling tank would be directed to Distribution Box No. 2.

- Modify the existing or construct a new primary influent distribution box. Modify influent distribution within each tank
- Modify primary effluent distribution. Relocate the RAS discharge to the primary effluent distribution box.
- Modify the influent baffles within each existing tank to dissipate inlet velocity and reduce hydraulic short-circuiting.
- Reconfigure the drive location for the primary sludge cross collectors. The cross collector drives should be placed on the wall opposite the primary sludge pump suction so that sludge and grit are pushed away from the drive end of the screw.
- Provide automated scum removal.
- Provide five new primary sludge pumps with VFDs.
- Provide dehumidification in the Primary Pump Gallery.
- Provide odor control for the primary clarifiers. Including covering and exhausting air from the influent and effluent distribution boxes as well as the primary effluent launders and treating odorous air. At this time, a containerized biofilter system is recommended. However, during preliminary design, available alternatives should be considered and re-evaluated.

11.3.5 Biological Wastewater Treatment

- Modify each of the four aeration tanks to operate in the Four-Stage Bardenpho Process configuration for nitrogen reduction.

- Subdivide the anoxic zone so that, if necessary, a portion of the anoxic zone could be operated anaerobically for biological phosphorus removal. Maintain diffusers in anoxic zones so that they can be operated aerobically, if necessary, during cold weather periods to maintain nitrification.

- Replace the existing nitrate recycle pumps in each aeration tank with higher capacity units. Provide multiple discharge locations to allow for operating an anaerobic zone ahead of the anoxic zone, if necessary. Provide VFDs for the nitrate recycle pumps.

- Evaluate the existing hoists for nitrate recycle pumps in Aeration Tanks No. 3 and No. 4 to determine if they can be reused with the larger recommended recycle pumps. If possible, relocated and reuse hoists with new system.

- Complete cost evaluation to determine if it is beneficial to replace the existing blowers with new more efficient blowers at future air demands for the recommended Four-Stage Bardenpho Process. Also evaluate the reliability and remaining useful life of existing blowers.

- Pipe effluent from Aeration Tanks No. 3 and No. 4 to Aeration Tanks No. 1 and No. 2 influent channel. Pipe effluent from Aeration Tanks No. 1 and No. 2 to Distribution Box No. 5. Raise the effluent weir in Aeration Tanks No. 1 and No. 2 by approximately 12 to 18 inches to provide additional hydraulic head.

- Replace existing isolation valves on aeration system drop pipes.

- Install Nitrate, ORP and pH meters in the aeration tanks.

- Install a Supplemental Carbon and Alkalinity adjustment chemical feed system.

- Construct one new 80-foot diameter circular final settling tank with a 16-foot side water depth (SWD) (Final Settling Tank No. 6). Consider installation to the south of Final Settling Tank No. 5.

- Modify the existing Distribution Box No. 5 or construct a new distribution box for Final Settling Tanks No. 4, No. 5, and No. 6.

- Remove concrete overflow structure to storm drain in Final Settling Tanks No. 4 and No. 5.

- Provide algae sweeps and full-radius scum removal on all circular final settling tanks.

- Provide turbidity meters at effluent end of all final settling tanks.

- Provide new RAS/WAS pumps in the existing pump room for new Final Settling Tank No. 6.

- Replace VFDs on existing RAS pumps.

- Clean and repair concrete cracks and wall penetrations that contribute to water leakage in the pump room.

- Seal below grade electrical conduit to prevent groundwater leakage, or provide above-grade junction boxes for the pump room.

- Provide pipe and valve modifications in Secondary Pump Gallery No. 2 to facilitate tank draining and isolation.

- Provide dehumidification in Secondary Pump Gallery No. 2.

11.3.6 Disinfection and Effluent Discharge

- Provide new outfall pumps to replace aging system.

- Provide a catwalk around the outfall pumps to facilitate access.

- Provide hoist for outfall pump removal.

- Provide in-line chlorine analyzers.

- Provide effluent flow monitoring.

11.3.7 Tertiary Treatment

- Install new Ballasted Flocculation Tertiary Treatment Process

- Install Tertiary Treatment equipment room and Tertiary Treatment influent pump station.

- Install Chemical storage and feed equipment room. This room shall also include chemical storage and feed equipment for alkalinity adjustment for the secondary treatment process.

11.3.8 Sludge Disposal

- Provide coating on interior of thickened sludge storage tanks to reduce potential for corrosion.

- Abandon the existing gravity belt thickener and install three screw press type dewatering equipment.

- Replace the existing gravity belt thickener feed pumps with two progressing cavity or rotary lobe pumps, which will be used to feed the screw press type dewatering equipment.

- Provide magnetic flow meters downstream of the dewatering feed pumps and the thickened primary sludge pump.

- Improve mixing capabilities in sludge holding tanks.

- Provide overflow drains on the sludge storage tanks

- Cover primary sludge thickener and sludge holding tanks.

- Provide an odor control system for the covered tanks, the truck loading area, and the secondary sludge thickening area. At this time, a containerized biofilter system is recommended. However, during preliminary design, available alternatives should be considered and re-evaluated.

11.3.9 Plant Support Systems and Facilities

- Construct new Maintenance Building/Garage.
- Provide a slide rail system in all liquid process tanks for a portable submersible dewatering pump.
- Provide air compressors and compressed air distribution systems in each building or pump gallery.
- Upgrade HVAC system in the Administration Building and Operations Building.
- Provide an overhead crane and testing bench in New Maintenance Building Garage.
- Replace emergency generator.
- Relocate fence to match property line, and install an automatic front entry gate with card/PIN access.
- Expand the laboratory in the Administration Building.
- Provide pressure regulator on incoming potable water line.
- Upgrade roof drainage system on each building.
- Demolish the abandoned Burrville Wastewater Treatment Plant.

11.4 DESCRIPTION OF RECOMMENDED PLAN

11.4.1 Screenings Building

The recommended improvements to the Screenings Building are extensive. Although the existing facilities are both operable and meet the capacity requirements, the age and condition of the building and equipment, as well as regulatory requirements for screenings disposal, warrants improvements.

The mechanically cleaned bar screens would be replaced with equipment that is more efficient and would require less operation and maintenance attention. The recommended screens would be climber or step type. The new screens may need to be staggered approximately eight feet apart to provide additional space for equipment maintenance. A new screenings grinder-washer-compactor would be provided for each screen. The actual equipment configuration would be confirmed during the preliminary design process.

The screenings grinder-washer-compactor would meet the requirements to facilitate disposal of the wastewater screenings as municipal solid waste (MSW). Additional benefits include a reduction in the volume of screenings to be disposed and lower odor potential from the screenings.

To improve maintenance operations, it is recommended that the concrete above the influent channels be removed. The influent channels would be covered with removable aluminum diamond plating. Hinged access panels would be provided. A separate ventilation system would be provided to exhaust the air from beneath the channel covers to the Preliminary Treatment Odor Control System.

The recommended mechanical improvements will require extensive modification to the Screenings Building. Considering the height requirements of the new mechanical screens and the current condition of the existing roof, the recommended improvements would include the complete replacement of the roof. The new roof elevation would be higher than the current elevation if a new climber screen were provided. In addition to the new roof, the recommend improvements would include an expansion of the building.

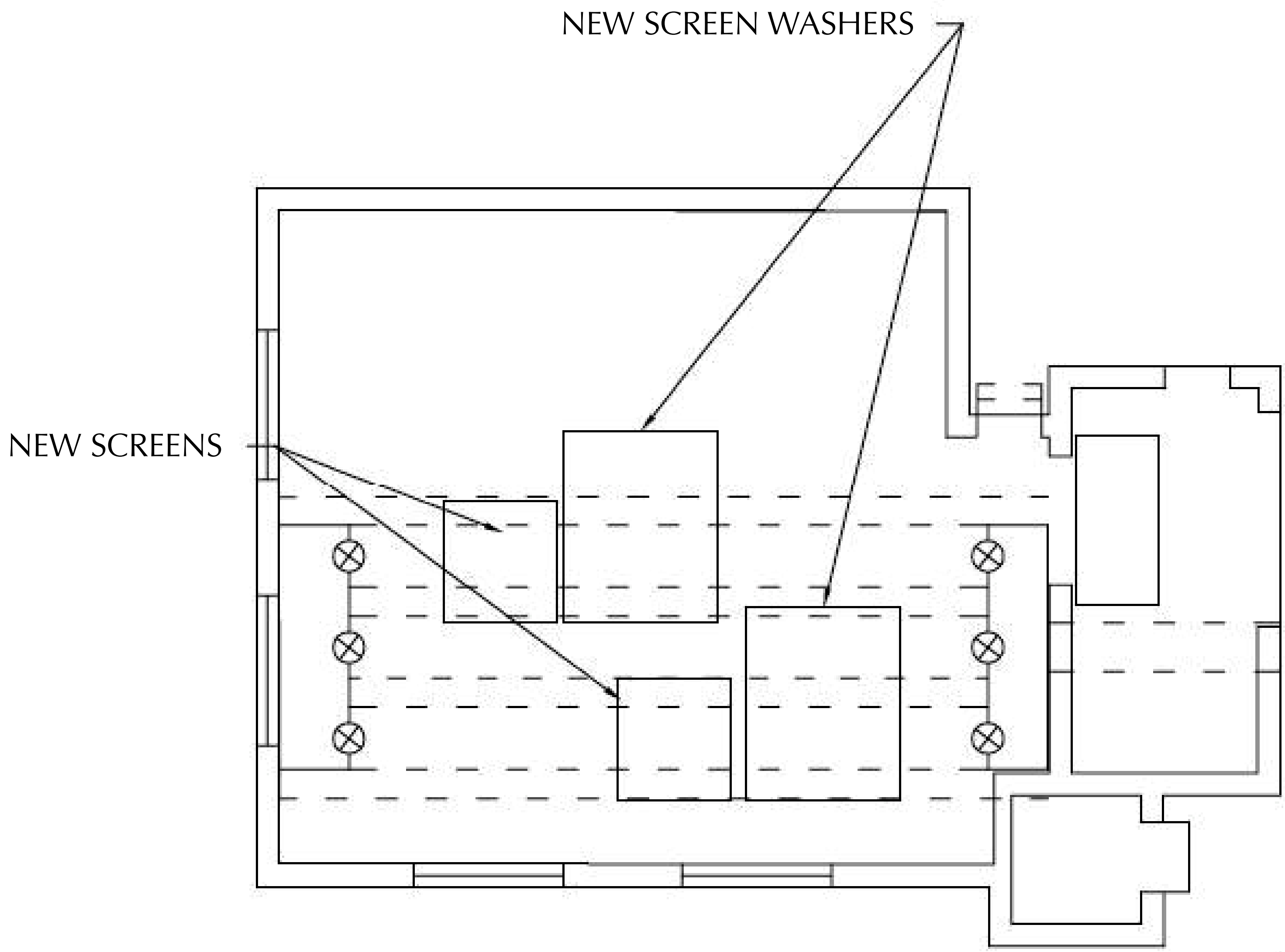
The building footprint would be expanded approximately ten feet to the north and ten feet to the east. The electrical room will also require expansion. The roof exhaust fan and the heaters in the screenings portion of the building would be upgraded. The operating space would be ventilated when the building is occupied. The ventilation system in the current screenings container room is adequate and requires no modifications.

Other improvements at the Screenings Building would be implemented for disposal of waste from Vactor trucks and recreational vehicles. Currently these wastes are disposed of just downstream of the outlet siphon structure. As part of the 1994 plant improvements, a simple pad and open drain system was installed. The pad has an 8-inch high curb on two sides. One side is bounded by the outlet siphon structure and the fourth side is open. The current disposal practice is messy and introduces significant quantities of grit and sand into the influent channel. Improvements would be made to better contain the waste discharge and separate the grit and sand from the influent wastewater.

To construct the Screenings Building improvements, the influent flow would need to be temporarily bypassed around the facilities. At that time, the influent channels would be drained, cleaned and inspected and any necessary concrete repairs would be completed. A conceptual layout of the recommended improvements is provided in Figure 11-2.

11.4.2 Septage Receiving Facility

Although the septage receiving facility is relatively new, several improvements are recommended. A stand-alone septage receiving unit is recommended to pretreat (screen and grit removal) incoming septage prior to discharging to the existing septage receiving storage tank. This would eliminate many of the current maintenance issues observed with the existing septage receiving and storage system.



not to scale

TORRINGTON WPCF
RECCOMENDED IMPROVEMENTS
– SCREENING BUILDING

PROJ NO: 12411A	DATE: DEC 2004	FIGURE: 11 - 2

WRIGHT-PIERCE
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In addition, the recommended unit would monitor and record volumes of incoming septage loads from individual haulers. The original system relied on a self-reporting system of volume discharged with the potential for abuse and lost revenue. Recently plant staff has fabricated a portable flow metering system that is manually operated. However, it is recommended that a septage receiving control system be provided with the package unit to automatically meter and record the volume of septage received. The recommended system would consist of a magnetic flow meter, a motorized ball valve, a new 4-inch quick disconnect, and an access control panel. The access control panel would be operated by a PIN code or key card. The operation of the motorized ball valve and flow meter would be dependent on authorized access. The volume of septage discharged would be measured and recorded for each authorized hauler and would be used for billing.

The mixer for the septage holding tank would be replaced. The new mixer would have a longer shaft, and a larger impeller to provide better mixing efficiency. The mixer could be operated using a timer or in conjunction with the discharge pump to reduce electrical power consumption.

The existing septage discharge pump is located remotely in the primary pump gallery. The long suction and discharge piping creates operational concerns. Therefore, the recommended improvements include a new submersible chopper-type septage pump. The new pump would be located in the septage holding tank and would include new level controls and timers, and an access hatch.

The septage pump discharge piping would be rerouted to discharge upstream of the mechanical screens. Piping and valve modifications can be made to allow the existing septage pump to remain in place as a redundant unit.

11.4.3 Grit Facility

The Torrington WPCF does not currently have a separate grit removal process. Grit settles out with the sludge in the primary settling tanks. The grit is removed from the primary sludge using a Huber grit removal system. Because there is little in-tank mechanical equipment associated with this type of grit removal unit, a complete redundant unit is not required. The Huber grit removal process is relatively new, in good condition and has not been noted to have any operation issues. Therefore, it was determined that no additional equipment or process improvements recommendations were developed for this unit process.

11.4.4 Primary Settling Tank

To meet current design criteria at projected future flows and to provide redundancy so that a tank can be taken off line for maintenance, a fourth primary settling tank (PST) is recommended. The fourth PST would be located adjacent to existing PST No. 3. The dimension and configuration of the new tank would replicate the existing tanks. The existing primary influent Distribution Box No. 1 would be modified or replaced with a new structure to improve the distribution to the primary settling tanks. Also, modifications would be required for primary effluent Distribution Box No. 2. As discussed below, to provide for modifications to the secondary treatment process, and to allow all four aeration tanks to operate as a single system, it may be necessary to discharge return sludge to Distribution Box No. 2. Modifications to allow complete mixing of RAS with primary effluent prior to flow splitting would also be necessary.

The existing primary sludge cross collectors would be modified so the drive is on the wall opposite the primary sludge pump suction. This will reduce the amount of sludge building up at the drive-end components and would address the issue of the mechanical components pulling out of the wall due to the force of trying to pull the significant quantities of sludge and grit currently received. Mechanical scum removal would also be provided.

Primary sludge is pumped through the Huber Grit Washing Plant system to the primary sludge thickener using the existing grit pumps. Because of the age of the pumps, the existing grit pumps would be replaced with new recessed-impeller centrifugal pumps with variable frequency drives. To accommodate the new primary settling tank and maintain the same level of redundancy, five new grit pumps are recommended. Modifications to the primary pump gallery, piping, and valve configuration would be required. Additionally, provisions to facilitate complete draining of the primary settling tanks are recommended. To address the moisture concerns associated with the primary pump gallery, a new dehumidification system would be provided.

11.4.5 Biological Wastewater Treatment

The recommended improvements to the biological wastewater treatment system would improve long-term equipment reliability, process reliability, and minimize the cost to purchasing nitrogen credits while minimizing capital costs.

The existing secondary treatment system could meet the WPCF's current NPDES permit limits under projected future flow and load conditions. In addition, the available aeration tank volume is sufficient to provide the level of nitrogen removal required by the *Nitrogen General Permit* at design year annual average conditions if all four tanks are converted to operate in the Four-Stage Bardenpho Process. During periods of extreme high flows and cold-weather maximum month flows, nitrification may be reduced or lost and the use of a swing zone is recommended to provide additional aerobic zone volume during cold-weather periods.

Conversion to the Four-Stage Bardenpho Process configuration in all four tanks would require the following modifications and improvements:

- The first portion of Aeration Tanks Nos. 4 and 5 would be modified to function as an anoxic zone. Submersible mixers would be installed in the anoxic zones to mix the contents without the introduction of oxygen. The existing aeration system grid may remain in place for operational flexibility, but provisions would be made to provide a positive air shut-off. The anoxic zones would be further divided to allow a portion of the zone to operate as an anaerobic zone in order to provide the flexibility for biological phosphorus removal.

- The remainder of Aeration Tanks Nos. 4 and 5 would operate under aerobic conditions. The existing fine-bubble aeration system is sufficient in each tank. To better control aeration, and provide operational flexibility, the aeration system drop pipes would be provided with isolation valves capable of providing complete shut-off.

- A deoxygenation zone would be provided at the effluent end of Aeration Tanks Nos. 4 and 5 for installation of the internal nitrate recycle pumps.

- A new pipeline would be installed to transfer flow from Aeration Tanks Nos. 4 and 5 to Aeration Tanks Nos. 1 and 2.

- The first portion of Aeration Tanks Nos. 1 and 2 would be modified to function as aerobic zones, with the installation of fine bubble diffusers. Aeration system drop pipes would be provided to each zone with isolation valves capable of providing complete shut-off.

- The remainder of Aeration Tanks Nos. 1 and 2 would operate as anoxic zones. Submersible mixers would be installed in the anoxic zones to mix the contents without the introduction of oxygen. An aeration system grid would also be installed for operational flexibility, but provisions would be made to provide a positive air shut-off.

- A re-aeration zone would be provided at the effluent end of Aeration Tanks Nos. 1 and 2.

- The effluent weirs on Aeration Tank Nos. 1 and No. 2 would be raised approximately 12 to 18 inches. Effluent from these two aeration tanks would be re-piped to discharge to Distribution Box No. 5.

- An internal nitrate recycle system would be provided at the end of the aeration zones within Aeration Tanks Nos. 4 and 5. The system would recycle mixed liquor back to the anoxic zone to provide a source of carbon necessary for denitrification. Each nitrate recycle pump would be equipped with a variable speed drive for operation at between 100% and 400% of the influent flow rate. These pumps would operate proportional to a signal from the new plant effluent flow meter.

- A supplemental carbon and alkalinity adjustment chemical feed system will be installed to maximize the efficiency of the secondary treatment process.

- Because the system will operate as a single process, it will be necessary to relocate the RAS discharge. It could be mixed with the primary effluent prior to splitting the primary effluent flow to the aeration tanks such as at Distribution Box No. 2.

The nitrification process consumes alkalinity. Because the primary effluent has a relatively low alkalinity, an alkalinity storage and feed system may need to be provided to improve the nitrogen reduction process. Sampling would be conducted during the preliminary design process to determine the required alkalinity storage volume and feed system sizing. To provide improved operational control, Nitrate analyzers, ORP and pH monitors would be provided in the aeration tanks.

The existing rectangular secondary clarifiers (Secondary Clarifiers No. 1, No. 2 and No. 3) are extremely shallow by current design standards and are difficult to operate. It is recommended that these tanks be eliminated from the secondary treatment process. In order to provide additional secondary clarifier capacity, the recommended improvements include the addition of a new circular final settling tank. The new final settling tank could potentially be located adjacent to Final Settling Tank No. 5. The new final settling tank would be 80-feet in diameter with a side water depth of 16 feet in accordance with current TR-16 Standards. The settling tank would be equipped with a sludge removal mechanism, full radius scum removal mechanism, density current baffles, energy dissipating inlet baffles, and an algae sweep system. The existing circular final settling tanks would also be retrofitted with an algae sweep system and full radius scum removal mechanisms. The algae sweep system would address the concerns associated with algae accumulation on the weirs and effluent launder. The weirs and effluent launder are located significantly below ground level. This makes it difficult and time consuming to thoroughly remove the accumulated algae. An algae removal system would require the removal of the concrete overflow structure to the storm drain.

New RAS and WAS pumps would be provided in Secondary Pump Gallery No. 2 for the new final settling tank. As part of the plant improvements completed in 1994, space was provided for a fourth RAS and WAS pump for a future final settling tank. The new RAS pump would be equipped with variable speed controls. The existing RAS and WAS pumps are in good working condition. However, the existing VFDs on the units are obsolete. Therefore, as part of the recommended improvements, the VFDs on all the existing RAS pumps (RS-1, RS-2, RS-3, RS-4, RS-5, RS-6, and RS-7) would be replaced. The discharge from the RAS pumps would need to be relocated to primary effluent Distribution Box No. 2 so that the RAS is thoroughly mixed with the primary effluent before it is distributed to each of the aeration tanks.

In addition to the new pumps and VFDs, other improvements are recommended for the Secondary Pump Gallery. Hairline cracks in the walls, pipe penetrations, and electrical pull boxes show evidence of previous leakage. The leak locations would be cleaned and repaired by pressure injecting a sealant compound. The electrical and HVAC systems in Secondary Pump Gallery would be modified to improve existing operation and code concerns. The pipe and valve manifold systems in the Secondary Pump Gallery would be modified. The recommended modifications would be implemented to allow complete and isolated drainage of individual aeration and final settling tanks while other tanks remain in service.

The existing blowers provide sufficient air supply for the current and proposed activated sludge process as well as for the post-aeration system. Each blower has an adjustable air flow rate of between 1,800 and 3,700 scfm. When the air requirements are low, one blower would be operated and the airflow adjusted by throttling the butterfly valve in the intake line. Under some scenarios, the air requirement could be below 1,800 scfm. The blowers are generally not operated below this value because surge would be induced resulting in vibration and overheating. Excess air can be diverted to the post-aeration tank in this case. If the air requirement is greater than 3,300 scfm, a second blower is put on line and the intake butterfly valves throttled to achieve the required air flow rate. During preliminary design, the cost-benefit of replacing the existing blowers with new premium efficiency blowers and VFDs should be evaluated.

11.4.6 Tertiary Treatment Process

The recommended addition of a tertiary treatment process would improve the long-term process reliability of meeting the future anticipated effluent total phosphorus limit. It is recommended to install a Ballasted Flocculation Tertiary Treatment Process. The Ballasted Flocculation process would provide the flexibility to meet the anticipated 0.32 mg/l total phosphorus limit as well as a future potential 0.1 mg/l total phosphorus limit, with no additional equipment or process changes.

As part of the tertiary treatment process, additional tank volume is needed for each step of the Ballasted Flocculation Process.

In addition to the installation of necessary treatment volume, a tertiary treatment influent pump station, an equipment room and a chemical storage and feed equipment room would also be needed. Preliminary investigations have determined there would be adequate space for the tertiary treatment process, as well as the necessary pump station, equipment and chemical storage rooms within the Secondary Clarifier Nos. 1, 2 and 3 tank structures. The existing rectangular clarifier tanks would need to be modified to facility the installation of the tertiary process tanks and equipment rooms. Additional structural, process piping and hydraulic requirements would be evaluated during the development of the preliminary design. The tertiary chemical storage and feed equipment room would also include space for chemical storage and feed equipment for alkalinity adjustment within the secondary treatment process.

11.4.7 Disinfection and Effluent Discharge

The existing chlorination and dechlorination facilities are in good condition. The existing chlorine contact tanks provide sufficient capacity for future peak hour flow rates. Therefore, conversion to a UV disinfection system is not cost-effective at this time.

As part of the recommended improvements, the coating system on the chlorine contact tanks would be removed, and the concrete cleaned and repaired. The recommended concrete rehabilitation is detailed in the Technical Memorandum provided in Appendix D.

The outfall pumps are used infrequently, but have still require regular inspection and extensive maintenance. The pumps are 18 years old and are approaching the end of their useful life. In order to ensure reliable operation, replacement of these pumps is recommended. In addition, to improve access to the pumps, a platform and hoist system would be installed adjacent to the pumps. The access platform would extend the full width of the outfall pump wetwell and be equipped with stairs and handrails as required.

Currently, the plant flow is measured only at the influent flow meter. It is anticipated that the DEEP will require provisions for effluent flow monitoring in the future. Therefore, an effluent flow metering system is recommended. Alternatives for flow metering include the installation of an ultrasonic meter above the existing effluent weir with modifications to the weir to improve flow metering accuracy, a Parshall flume and ultrasonic meter retrofitted in the chlorine contact or post aeration tanks, or an in-line magnetic flow meter on the effluent piping.

Sufficient flow metering throughout both the liquid and solid processes is a critical component for successful plant operation and performance optimization. As part of the preliminary design phase associated with the implementation of the recommended improvements, a comprehensive flow metering system would be established, to maximize data gathering, while minimizing costs.

11.4.8 Sludge Disposal

Based on the evaluation of current disposal costs and the capital costs associated with dewatering facilities, it is recommended that the City upgrade the solids handling facility to dispose of dewatered cake sludge.

Several improvements are recommended to implement a dewatering operation at the Torrington facility. The existing gravity belt thickener would be abandoned and three screw press type dewatering units would be installed in the garage bay area adjacent to the operations building. A conveyor system as well as a roll off container would also be installed in the garage bay area to handle and store dewatered cake until hauled off-site for disposal.

Several alterations to the current sludge pumping system would also be completed in order to facilitate the installation of the dewatering process. As part of the recommended improvements, the existing gravity belt thickener feed pumps would be replaced with rotary lobe pumps which would act as screw press feed pumps. Each pump would be sized to feed the screw press at a range of 40 gpm to 90 gpm. This would provide one pump for normal operation, and one redundant, back-up pump. A magnetic flow meter would be installed on the screw press feed pump discharge piping. This flow meter would provide improved monitoring of the sludge throughput on the screw press and allow for operation optimization.

Prior to the screw press, a blending tank is needed to blend the thickened primary and tertiary sludge and the unthickened secondary sludge, which would be fed to the screw press. A section of the existing Auxiliary Sludge Holding Tank could be converted into a blend tank. Additional pumps and piping would be needed to meter flow from the thickened sludge storage tanks and the unthickened sludge holding tank, to the proposed blend tank.

All of the pumps recommended to improve the sludge pumping process would be equipped with variable frequency drives to allow better operator control of the sludge discharge rate.

The sludge storage tanks are generally in good condition. Limited improvements are recommended for these structures. To provide better mixing of the thickened sludge holding tank contents, the mixer blades would be replaced and the shaft lengthened. Covering of the sludge storage tanks and gravity thickener are recommended for odor containment. Odor control facilities would be provided for the exhaust air from these tanks as well as from dewatering equipment area. Currently, based on life-cycle cost evaluations conducted at other facilities, it is anticipated that a containerized biofilter system would be the most cost-effective technology. However, the specific technology to treat odors should be re-evaluated during the preliminary design phase. Because the sludge storage tanks and gravity thickener would be covered, the tanks would also be coated to reduce the potential for concrete corrosion.

11.4.9 Plant Support Systems and Facilities

Several improvements are recommended for non-process systems and facilities.

Garage, Maintenance and Storage Area - There is a need for increased maintenance and storage space. Maintenance activities for the treatment plant are primarily carried out in the facilities on the south end of the Operations Building. Currently, approximately half of the available space is used for maintenance activities while the remaining space is used for vehicle storage. However, with the installation of the dewatering process within the garage space, new garage and maintenance space is needed. There are a few smaller areas within the treatment plant where maintenance activities are completed. Collection system related maintenance is generally performed off-site, or in the Sewer Maintenance Garage. This maintenance garage is a three-bay pre-engineered building located near the Operations Building. Various areas across the facility are used for storage including a room in the old Administration Building, two rooms in the Chemical Building, the garage of the Operations Building, and the Greenhouse.

As part of the recommended improvements, the maintenance and storage activities would be consolidated in a common area. The existing building used for sewer maintenance is in good condition, and would be retained. A new multi-bay, pre-engineered metal building would be located adjacent to the sewer maintenance garage. The recommended building would be approximately 50 feet wide and 160 feet long. If additional area was desired, a fourth bay could be added to the existing sewer maintenance garage.

The new building would allow the primary maintenance operations performed in the garage area of the Operations Building to be relocated. This would provide two additional vehicle bays in the existing Operations Building. The two pre-engineered buildings (1 existing, 1 proposed) would provide sufficient space for a consolidated maintenance and storage facility. The detailed requirements, such as configuration, functionality, heating requirements, storage systems, vehicle washing facility, etc, would be developed as part of the preliminary design process.

<u>HVAC in Administration Building and Operations Building</u> – Although the existing HVAC systems in the the administration building and operations buildings were found to be in good condition, it was noted that these system are old and approaching their useful lives. It would be recommended to upgrade these systems with more up-to-date/efficient equipment. It is also recommended that the existing electrical heating systems be evaluated during the preliminary design phase to determine whether to replace these systems in-kind or if it would be cost effective to utilize renewable energy technology, such as geothermal heating or using a heat pump with the plant water effluent as a heat source.

<u>Emergency Generator</u> - The existing emergency generator was installed as part of the 1970 plant upgrade. Although the current total connected continuous load is less than the generator capacity, not all necessary loads are connected. Critical equipment, such as the blowers and outfall pumps are not connected. Additionally, under certain conditions, the generator may not be capable of handling the required load. Because of the age of the existing emergency generator, and loading concerns, a new generator is recommended.

<u>Laboratory</u> - The existing laboratory facility in the Administrative Building does not provide adequate space to efficiently and effectively perform all of the required tests at the WPCF. The existing laboratory consists of an approximately 300 sq-ft area equipped with a ventilation hood, sinks, and counter space as well as an 85 sq-ft office room with a desk and two refrigerators. The laboratory is equipped with most of the major glassware and equipment needed to perform the required tests. However, most of the testing equipment is old and outdated and the WPCF is in the process of replacing some of the major pieces of equipment with newer technologies.

The ease and efficiency of conducting the required tests is constrained by the limited space. Additional workspace is recommended. To provide additional workspace in the laboratory, the wall along the east side of the existing laboratory would be removed, providing access to the existing storage room on the other side of the wall. The storage room area would provide an additional 120 sq-ft of laboratory work space. In addition to removing the wall between the laboratory and storage room, the storage room would be brought up to code and provided with a new heating and ventilation system. The heating and ventilation system would be tied into the existing laboratory system. Modifications would also be required to raise the floor in the storage room several feet to match the floor elevation of the existing laboratory.

Burrville Wastewater Treatment Plant - The Burrville Wastewater Treatment Plant, located at the corner of Winsted Road and Greenwoods Road was constructed in the late 1960's. Several years ago, the plant was decommissioned. As part of the recommended improvements, the plant would be demolished and any maintenance activities that currently take place there would be relocated to the new maintenance facilities. Appendix F includes a Technical Memorandum on the evaluation completed for the Burrville facility.

11.5 ESTIMATED CAPITAL COSTS

Estimates of the probable construction costs for the recommended improvements were developed. A summary of the estimated capital costs is presented in Table 11-1. These costs include estimates of pertinent allowances and contingencies. A detailed breakdown of the costs, and a summary of the economic parameters used in their development is included in Appendix G. The costs presented in Table 11-1 and Appendix G are based on mid-year 2014 costs.

TABLE 11-1
CITY OF TORRINGTON WPCF FACILITY PLAN
SUMMARY OF RECOMMENDED IMPROVEMENTS CAPITAL COST
(AUGUST 2012)

DESCRIPTION			**COST**
SITE WORK			$3,035,000
PROCESS PIPING			$1,451,000
SCREENINGS BUILDING MODIFICATIONS			$1,106,000
PRIMARY CLARIFIER #4 / INFLUENT & EFFLUENT BOXES			$1,362,000
SEPTAGE RECEIVING FACILITY			$465,000
SECONDARY TREATMENT MODIFICATIONS			$2,893,000
NEW SECONDARY CLARIFIER #6			$1,240,000
TERTIARY TREATMENT FACILITY			$4,329,000
EFFLUENT / DISINFECTION FACILITIES			$728,000
ODOR CONTROL SYSTEMS			$624,000
NEW SLUDGE HANDLING FACILITIES			$1,386,000
MAINTENANCE / EQUIPMENT STORAGE / GARAGE			$1,750,000
LABORATORY FACILITIES EXPANSION			$275,000
ROOF DRAINAGE SYSTEM, EACH BUILDING			$100,000
MISCELANEOUS BUILDING REPAIRS			$327,000
		SUBTOTAL	$21,071,000
SPECIALS			$505,000
HVAC/PLUMBING			$1,780,000
INSTRUMENTATION			$750,000
ELECTRICAL			$2,725,000
		SUBTOTAL	$5,760,000
SUBTOTAL, GENERAL CONSTRUCTION			$21,071,000
GENERAL CONTRACTOR OH&P AND GENERAL CONDITIONS	20.0%		$4,214,000
SUBTOTAL, SUBCONTRACTORS			$5,760,000
GENERAL CONTRACTOR MARKUP	7.5%		$432,000
BONDS & INSURANCES	2.0%		$630,000
UNIT PRICE ITEMS (Ledge Excav., Additional Materials, etc.)	2.0%		$421,000
SUBTOTAL, CONSTRUCTION COSTS			$32,528,000
PROJECT MULTIPLIER, DESIGN CONTINGENCY, 20%	1.20		$39,033,600
TOTAL 2014 CONSTRUCTION COST (2 Yrs @ 4% INFLATION).	1.08		$42,200,000

It should be noted that in Section 6 of this report, it was determined that additional aeration tank volume would be needed should the secondary treatment process be required to meet the 2014 total nitrogen effluent limit on a monthly basis without the ability to purchase nitrogen credits, as opposed to an annual average limit as is required as part of the current Nitrogen General Permit. Additional aeration tankage is not included in this cost estimate because the existing tankage was determined to provide enough treatment volume for the proposed Four-Stage Bardenpho Process to achieve an effluent total nitrogen concentration that would meet the annual average 2014 total nitrogen limit.

In addition, with the installation of a ballasted flocculation tertiary treatment system, which is essentially a high performance clarifier, the need to install a forth secondary clarifier was not needed to provide the necessary settling capacity for future peak wet weather flow. In Section 6 it was determined three secondary clarifiers, two existing and one new, would have the settling capacity to handle up to the 96th percentile design peak hour flow rate. Although typical design criteria for a secondary clarifier are based on the 98th percentile design flow condition, the construction of a fourth new clarifier, in order to plan for only the upper 2 percentile of the future design peak hour flow rates, is not considered to be cost effective. The installation of the ballasted flocculation system, which is needed for phosphorus removal, will also give additional protection downstream of the secondary clarifiers during peak flow. Therefore, it was not recommended to construct a fourth secondary clarifier as part of this upgrade, and the cost of a future fourth clarifier was not included in this cost estimate.

Cost estimates for capital improvements vary depending on the degree of project definition that exists at the time of the estimate. The Association for the Advancement of Cost Engineering (AACE) identifies three major project phases as exploration (planning and conceptual design), evaluation (basic/preliminary design), and execution (detailed engineering design). The level of accuracy in a cost estimate will become greater as the project stage proceeds from exploration through evaluation to execution. The levels of accuracy for each project phase are presented in Table 11-2. The Torrington WPCF Facilities Study is considered to be in the *Exploration* phase.

Capital costs used in the development of project costs estimates include material and installation costs for structures, site work, process equipment, and auxiliary equipment associated with the project.

TABLE 11-2
CITY OF TORRINGTON WPCF FACILITY PLAN
CAPITAL COST IMPROVEMENTS ACCURACY LEVEL

Phase	Type of Estimate	Expected Accuracy
Exploration	Order of Magnitude	+50% to -30%
Evaluation	Budget	+30% to -15%
Execution	Detailed	+15% to - 5%

11.6 IMPLEMENTATION PLAN

As presented in Table 11-1, the total anticipated construction cost of the recommended improvements is approximately $42.2 million, based on 2014 dollars. One implementation approach would be to construct all of the recommended improvements as a single project. However, the facility currently does not have any major compliance problems, is well maintained and operated, and appears to be able to currently achieve its phosphorus removal requirements. Therefore, another approach would be to prioritize the improvements for implementation in a phased approach to match available funding. Based on discussions with the City of Torrington, an implementation plan was developed for the recommended improvements to be completed in a phased approach as part of a long-term capital improvement plan. The breakdown of the proposed implementation phases are summarized as follows:

- Phase 1:
    - Preliminary Treatment Improvements
    - Septage Receiving Improvements
    - Preliminary Treatment Odor Control System
    - Construction of fourth Primary Clarifier and Primary Clarifier Odor Control System.

- Secondary Treatment Improvements including conversion to Four-Stage Bardenpho Process, modifications to existing Final Settling Tanks and construction of Final Settling Tank No. 6 and associated pumping systems.
    - Solids Handling Improvements and Solids Handling Odor Control System.
    - Implantation of the improvements to the existing buildings
    - Demolition of the Burrville facility
    - WPCF and Pump Station security improvements
- Phase 2:
    - Installation of the tertiary treatment system
    - Installation of tertiary treatment influent pumps station

A summary of the cost items included in each phase are presented in Tables 11-3 and 11-4 at the end of this heading. Note that the cost of the Phase 2 was estimated in 2018 dollars, assuming 4% inflation over 6 years.

TABLE 11-3

CITY OF TORRINGTON WPCF FACILITY PLAN

SUMMARY OF RECOMMENDED IMPROVEMENTS CAPITAL COST - PHASE 1

(AUGUST 2012)

DESCRIPTION		**COST**
SITE WORK		$2,565,000
PROCESS PIPING		$730,000
SCREENINGS BUILDING MODIFICATIONS		$1,106,000
PRIMARY CLARIFIER #4 / INFLUENT & EFFLUENT BOXES		$1,362,000
SEPTAGE RECEIVING FACILITY		$465,000
SECONDARY TREATMENT MODIFICATIONS		$2,875,000
NEW SECONDARY CLARIFIER #6		$1,240,000
CHEMICAL FEED STORAGE AND EQUIPEMENT		$805,000
EFFLUENT / DISINFECTION FACILITIES		$728,000
ODOR CONTROL SYSTEMS		$624,000
NEW SLUDGE HANDLING FACILITIES		$1,386,000
MAINTENANCE / EQUIPMENT STORAGE / GARAGE		$1,750,000
LABORATORY FACILITIES EXPANSION		$275,000
ROOF DRAINAGE SYSTEM, EACH BUILDING		$100,000
MISC. BUILDING REHAB		$261,000
	SUBTOTAL	$16,275,000
SPEICAILS		$415,000
HVAC/PLUMBING		$1,630,000
INSTRUMENTATION		$525,000
ELECTRICAL		$2,000,000
	SUBTOTAL	$4,570,000
SUBTOTAL, GENERAL CONSTRUCTION		$16,275,000
GENERAL CONTRACTOR OH&P AND GENERAL CONDITIONS	20.0%	$3,255,000
SUBTOTAL, SUBCONTRACTORS		$4,570,000
GENERAL CONTRACTOR MARKUP	7.5%	$343,000
BONDS & INSURANCES	2.0%	$489,000
UNIT PRICE ITEMS (Ledge Excav., Additional Materials, etc.)	2.0%	$326,000
SUBTOTAL, CONSTRUCTION COSTS		**$25,258,000**
PROJECT MULTIPLIER, DESIGN CONTINGENCY	1.20	$30,309,600
TOTAL 2014 CONSTRUCTION COST (2 Yrs @ 4% INFLATION).	1.08	**$32,800,000**

TABLE 11-4

CITY OF TORRINGTON WPCF FACILITY PLAN

SUMMARY OF RECOMMENDED IMPROVEMENTS CAPITAL COST - PHASE 2

(AUGUST 2012)

DESCRIPTION			**COST**
TERTIARY TREATMENT FACILITY			$4,794,000
SPEICAILS			$90,000
HVAC/PLUMBING			$150,000
INSTRUMENTATION			$225,000
ELECTRICAL			$725,000
		SUBTOTAL	$1,190,000
SUBTOTAL, GENERAL CONSTRUCTION			$4,794,000
GENERAL CONTRACTOR OH&P AND GENERAL CONDITIONS	20.0%		$959,000
SUBTOTAL, SUBCONTRACTORS			$1,190,000
GENERAL CONTRACTOR MARKUP	7.5%		$89,000
BONDS & INSURANCES	2.0%		$141,000
UNIT PRICE ITEMS (Ledge Excav., Additional Materials, etc.)	2.0%		$96,000
SUBTOTAL, CONSTRUCTION COSTS			**$7,272,000**
PROJECT MULTIPLIER, DESIGN CONTINGENCY	1.20		$8,724,000
TOTAL 2014 CONSTRUCTION COST (6 Yrs @ 4% INFLATION).	1.27		**$11,000,000**

Tables 11-3 and 11-4 show the total construction cost would be $43,800,000. If the City decided to implement the Torrington WPCF upgrade project in two separate phases, the construction cost would be approximately $1.6 million more than implementing the entire project in one phase. Table 11-5 is a summary of the total probable project costs for both project implementation options.

TABLE 11-5

CITY OF TORRINGTON WPCF FACILITY PLANNING STUDY

TOTAL ESTIMATED PROJECT COST

(AUGUST 2012)

PROJECT COMPONENT		Option 1 Complete Upgrade	Option 2 Phase 1	Option 2 Phase 2
CONSTRUCTION ESTIMATE		$42,200,000	$32,800,000	$11,000,000
CONSTRUCTION CONTINGENCY	5.0%	$2,110,000	$1,640,000	$550,000
TECHNICAL SERVICES	15%	$6,330,000	$4,920,000	$1,650,000
VALUE ENGINEERING		$150,000	$125,000	$25,000
LEGAL/ ADMINISTRATIVE/FINANCING	1.0%	$420,000	$330,000	$110,000
ENGINEER'S ESTIMATE OF PROBABLE PROJECT COST		$51,300,000	$39,900,000	$13,400,000
			Sum of Phase I - II **$53,300,000**	

11.7 FUNDING

There are a variety of potential funding sources that the City of Torrington could pursue. At this time one of the most viable sources of funding is the State of Connecticut DEEP Clean Water Fund (CWF). The majority of the proposed improvements should qualify for a 20% grant and 80% loan at the 2% interest rate over 20 years. Improvements related to nitrogen removal qualify for an additional 10% grant funding. In addition, recent legislation has passed to allow for an additional 10% grant for phosphorus removal as well. Similar to nitrogen, anything phosphorus related will receive a 30% grant. This equates to an approximate 23% to 24% grant for the eligible items for the entire project. The City of Torrington will need to consider the status of the CWF when deciding on the actual implementation schedule for the recommended improvements. The actual implementation schedule can also be better defined as existing wastewater facility bond debt is retired.

SECTION 12

SECTION 12

ENVIRONMENTAL IMPACT ASSESSMENT

12.1 INTRODUCTION

As indicated in the DEEP's Clean Water Fund Checklist, direct impacts of the recommended plan to air and water quality, floodplains, coastal zones, wetlands, farmlands, aquifer protection zones, historical and archaeological areas, and endangered species must be assessed. The recommended plan includes improvements to the existing WPCF but does not anticipate any significant growth within the sewer service area or any expansion of the service area. Therefore, the direct environmental impacts would be limited to activities during construction. The direct and indirect environmental impact of the recommended plan was assessed along with potential mitigation of adverse impacts. These impacts and potential mitigation are discussed below.

12.2 WATER QUALITY IMPACTS

The upgrade of the treatment facilities would not have a negative impact on water quality. Continued operation of the existing facilities during construction is anticipated and the upgraded facilities will enhance nitrogen removal while providing more reliable equipment. During construction, some impact on water quality may occur due to sedimentation and erosion. However, mitigation procedures for soil and erosion control will be implemented along with proper handling of discharges from dewatering systems.

12.3 AIR QUALITY IMPACTS

Temporary air quality impacts will occur during construction due to dust and emissions from construction equipment. The construction contractor will be required to implement dust mitigation measures during construction. Other air quality impacts related to the project would include the implementation of odor control systems which would provide a long-term improvement in local air quality.

12.4 FLOOD PLAIN IMPACTS

The Federal Emergency Management Agency (FEMA) has prepared flood insurance studies for communities throughout the country. These studies present data and related hazards denoting flood zones. The FEMA maps for the Harwinton area shows that the treatment plant is outside the 100-year flood boundary. The 100-year flood elevation for the planning area is approximately 520 feet, based on USGS datum.

A flood dike, with an elevation greater than 520 feet, surrounds the north, east, and west boundaries of the WPCF. The typical ground elevation within the treatment plant is below the 100-year flood elevation, ranging from approximately 515 feet to approximately 517 feet. Process structures, electrical, and mechanical equipment must be protected from the physical damage from flooding by the expected 100-year flood. The existing dike provides this protection of the existing facilities. All recommended improvements will also be sited and designed to remain protected from flooding. The existing outfall pumps would remain in place to ensure the plant can remain fully operation during a 100-year flood event.

12.5 WETLANDS IMPACTS

Wetlands boundaries for the site are not currently available and mapping would be delineated by a soil scientist during the preliminary design phase to properly locate any wetlands within the WPCF boundaries. Impacts to any wetlands would be temporary due to construction activities. As described above, the contractor will be required to implement and maintain proper erosion and sediment control procedures during construction.

12.6 OTHER DIRECT IMPACTS

The recommended plan will take place within the existing boundaries of the WPCF. Other direct impacts from this project would be temporary due to construction activities including noise and traffic impacts. These issues would be mitigated to the extent possible by requiring construction activities to occur during a normal weekday schedule.

12.7 INDIRECT IMPACTS

Indirect impacts from wastewater facilities projects can include items such as induced growth. Construction of new sewer lines to serve an existing area with failing septic systems can induce more dense residential development in areas because of the availability of a public sewer. This growth can place a burden on other City services such as the school system and public water supply system. As discussed above, the project does not include any planned expansion of the sewer service area and anticipates very little growth over the planning period. Therefore, no indirect impacts from induced growth or increased demand on the water supply system are anticipated.

12.8 PERMITS AND APPROVALS

A preliminary review of the permits and approvals that would likely be required for this project to proceed was completed. A listing of the anticipated permits and approvals is presented below.

- Planning & Zoning Commission Approval
- Inland Wetlands Commission Approval
- Local Building Permits
- Fire Marshall Approval
- DEEP Stream Channel Encroachment Approval and Flood Management Certification
- General Permit for the Discharge of Stormwater Associated with Construction Activities

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