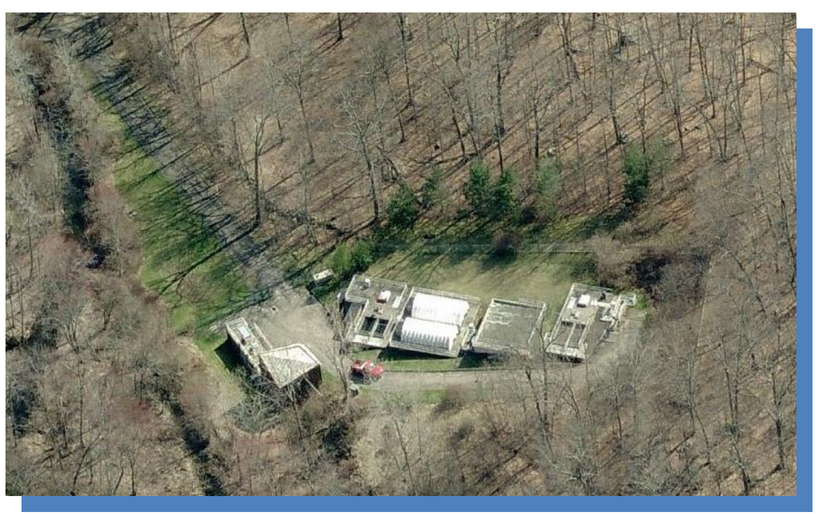
Draft Report to the Town of Ridgefield, Connecticut on the Phase 1 Wastewater Facilities Plan Volume 1

April, 2015



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April 28, 2015

Water Pollution Control Authority
66 Prospect Street
Ridgefield, CT 06877
Attention: Ms. Amy Siebert, P.E., Chairperson

Subject: Phase 1 Wastewater Facilities Plan
Draft Report

Dear Authority Members:

In accordance with our Agreement, we are pleased to submit this draft Phase 1 Wastewater Facilities Plan for your review. This report presents a review of the existing flows, loads, and capacity at the Town's two Wastewater Treatment Facilities (WWTFs), and projects the future flows and loads to the WWTFs in the year 2035. This report includes a summary of the seven previously submitted technical memorandums and reviews alternatives for accommodating future flows and loads and recommends the scope of work for the Phase 2 Facilities Plan.

We would be pleased to discuss the report and any comments you may have. If you have any questions concerning this report, please feel free to call me at (781) 224-6270.

Very truly yours,

Jon R. Pearson
Vice President
Metcalf & Eddy, Inc.

JRP/jrp

Encl.

**DRAFT REPORT TO THE
TOWN OF RIDGEFIELD, CONNECTICUT
ON THE
PHASE 1 WASTEWATER FACILITIES PLAN**

April, 2015

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CHAPTER ONE
INTRODUCTION

BACKGROUND

The Town of Ridgefield owns and operates two wastewater treatment facilities: the South Street Wastewater Treatment Facility (WWTF) which serves Sewer District 1 and the Route 7 WWTF which serves Sewer District 2.

The South Street WWTF is the larger of the two WWTFs with a design average flow of 1.0 million gallons per day (mgd). The South Street WWTF provides advanced treatment using the activated sludge process to treat wastewater collected from Sewer District 1 which includes downtown Ridgefield. The collection system consists of approximately 100,000 feet of sewer and 6 pump stations, with a portion of the sewer system over 100 years old. The South Street WWTF discharges its treated wastewater to the Great Swamp, and was last upgraded in 1992.

Sewer District 2 is located in the northeast portion of town in the area where Route 7 and Route 35 intersect. The District 2 collection system consists of approximately 6,300 feet of sewer and 1 pump station. Wastewater collected in this area is treated by the Route 7 WWTF. The Route 7 collection system and WWTF was constructed in the mid 1980's to serve the needs of Sewer District 2 which included flows from the Wells-Benrus facility. The Route 7 WWTF provides advanced wastewater treatment using rotating biological contactors, has an average daily design flow of 0.12 mgd, and discharges treated wastewater to the Norwalk River. Both town owned WWTFs and the Sewer District 1 and Sewer District 2 collection systems are operated by United Water through an operations contract with the Town.

Discharges from both plants are regulated by the Connecticut Department of Energy and Environmental Protection (DEEP) through permits issued under the National Pollutant Discharge Elimination System (NPDES) program. The existing permits for both WWTFs expired in 2009 and the Town submitted permit renewal applications as required by the program rules. The DEEP deferred issuing new permits for both WWTFs until DEEP's Phosphorus Reduction Strategy for Inland Non-Tidal Waters could be developed and finalized. In the meantime, the expired NPDES permits were administratively continued and remained in effect.

As a result of the implementation of the Phosphorus Reduction Strategy, in October 2014 DEEP issued a new NPDES permit for the Route 7 WWTF that includes an effluent phosphorus limit that the existing treatment facility cannot meet without modifications. The permit also includes a compliance schedule that defers the implementation of the new limit until August 2019 to allow time for the Town to complete the ongoing facilities planning effort and implement modifications to the Route 7 WWTF to meet the new phosphorus limit. It is anticipated that the South Street WWTF permit, once issued, will also contain a more stringent limit on effluent phosphorus as well as a compliance schedule to meet the new limit. It is anticipated that the existing WWTFs will not be able to meet their future permit limits without some modifications.

In addition, a condition of the NPDES permit for both plants requires if the 180 day rolling average for the plant average daily influent flow exceeds 90 percent of the design flow rate, the Town shall develop and submit a plan to accommodate future increases in flow to the plant. Historically, the South Street WWTF has operated below the design capacity of 1.0 mgd, except for occasional storm induced high flows, but the 90 percent flow threshold has been exceeded on several occasions. The Route 7 WWTF has not experienced any exceedances of this flow threshold.

To respond to the NPDES permit requirement to initiate planning to address the increases in flow, to address the new and pending phosphorus limits, and to address the aging equipment and components at the two WWTFs, the Town has undertaken preparation of this facilities plan. The facilities planning effort is being completed in two phases. In Phase 1 the current and future needs of the collection system for Sewer District 1 and Sewer District 2 have been identified, projected future flows and loads were developed, the capacities of the two WWTFs have been evaluated and the feasibility of land applying of treated effluent at the South Street WWTF has been assessed. Phase 2 will include the assessment of the condition and the current and future needs of both the South Street and Route 7 WWTFs, investigation of future effluent limits that may be imposed on the facilities, further infiltration/inflow control efforts, review of the cost effectiveness of eliminating the Route 7 WWTF by pumping collected flow to the South Street WWTF, and development of a recommended plan to accommodate the future flows and loads and meet the future effluent limits.

PURPOSE AND SCOPE OF REPORT

The purpose of this study is to review the existing flows, loads, and capacities at the two WWTFs, project the future flows and loads to the WWTFs in the year 2035, and formulate an approach to handle the future flows and loads. The scope of the report is outlined in detail in the engineering agreement between the Town of Ridgefield and AECOM Technical Services, Inc. and is comprised of a series of seven major tasks summarized in Table 1-1 below:

TABLE 1-1. SCOPE OF WORK TASKS

Task	Description
1. Review and Analyze Existing Flows and Loads	Obtain and review existing information and data from the WWTFs related to current flows and loadings. Conduct in-plant sampling program at the WWTFs to characterize unit process performance.
2. Evaluate Collection System	Conduct smoke testing in Sewer District 1; Update pump station evaluations, project future flows and loads for each district for a 20 year planning period. Conduct evaluation of hydraulic restrictions in the Sewer District 1 collection system.
3. Review WWTF Capacity	Review capacity of each WWTF unit process against accepted standards.
4. Assess Land Application Feasibility	Assess the feasibility of land application of treated wastewater from the South Street WWTF to reduce effluent phosphorus load.
5. Formulate Approach to Accommodating Future Flows	Consider approaches to accommodate the projected future flows.
6. Develop Scope of Work for Phase 2 Facilities Plan	Develop a scope for the Phase 2 Facilities Plan based on the approach selected to accommodate the projected future flows.
7. Summarize Phase 1 Findings and Action Plan	Provide a summary of findings from tasks 1 through 6.

In order to provide an opportunity for the Town to review the findings of the study as it progressed, the various efforts conducted in several of the major tasks were detailed in a series of Technical Memorandums. This approach allowed the study findings to be discussed and subsequent study efforts redirected as necessary while the planning process continued. Technical Memorandum No. 1 presented the results of smoke testing of the Sewer District 1 collection system. Technical Memorandum No. 2 contained the Sewer District 1 collection system hydraulic restriction (or “bottleneck”) evaluation. Technical Memorandum No. 3 presented the results of manhole inspections conducted in one test subarea in Sewer District 1. Technical Memorandum No. 4 contained an updated evaluation of the two oldest pump stations in the collection systems. Technical Memorandum No. 5 summarized the results of the land application evaluation in Sewer District 1. Technical Memorandum No. 6 reviewed existing wastewater flows and loads at both WWTFs and assessed the capacity of each WWTF. Technical Memorandum No. 7 projected future flows and loads for both WWTFs. The Technical Memorandums are included in the appendices to this report with the findings of each summarized in the following chapters.

CHAPTER TWO
EXISTING FACILITIES AND REGULATORY REQUIREMENTS

The Town of Ridgefield has two wastewater treatment facilities (WWTFs) that serve two different sewer service areas: Sewer District 1 served by the South Street Wastewater Treatment Facility (WWTF), and Sewer District 2 served by the Route 7 Wastewater Treatment Facility. Discharges of treated effluent from each WWTF are regulated by the Environmental Protection Agency (EPA) and the Connecticut Department of Energy and Environmental Protection (DEEP). A description of the existing wastewater collection and treatment facilities is presented in this chapter together with a review of the regulatory requirements related to the WWTFs.

SEWER DISTRICT 1

Sewer District 1 is the largest of the town's sewer districts, serving downtown Ridgefield and the surrounding areas. Sewer District 1 includes a wastewater collection system that collects wastewater from approximately 1,230 acres which represents about 5.5 percent of the town's area. The gravity sewer system consists of approximately 100,000 feet of sewers ranging in size from 6 inches in diameter to 18 inches, with approximately 1,760 billed service accounts. Most of the collection system conveys wastewater by gravity, but there are 6 pump stations (PS) in Sewer District 1 that lift the wastewater to a higher elevation. Figure 2-1 shows the existing Sewer District 1 wastewater collection system. Table 2-1 lists the pump stations in Sewer District 1 and key characteristics of each.

TABLE 2-1. SEWER DISTRICT 1 PUMP STATIONS

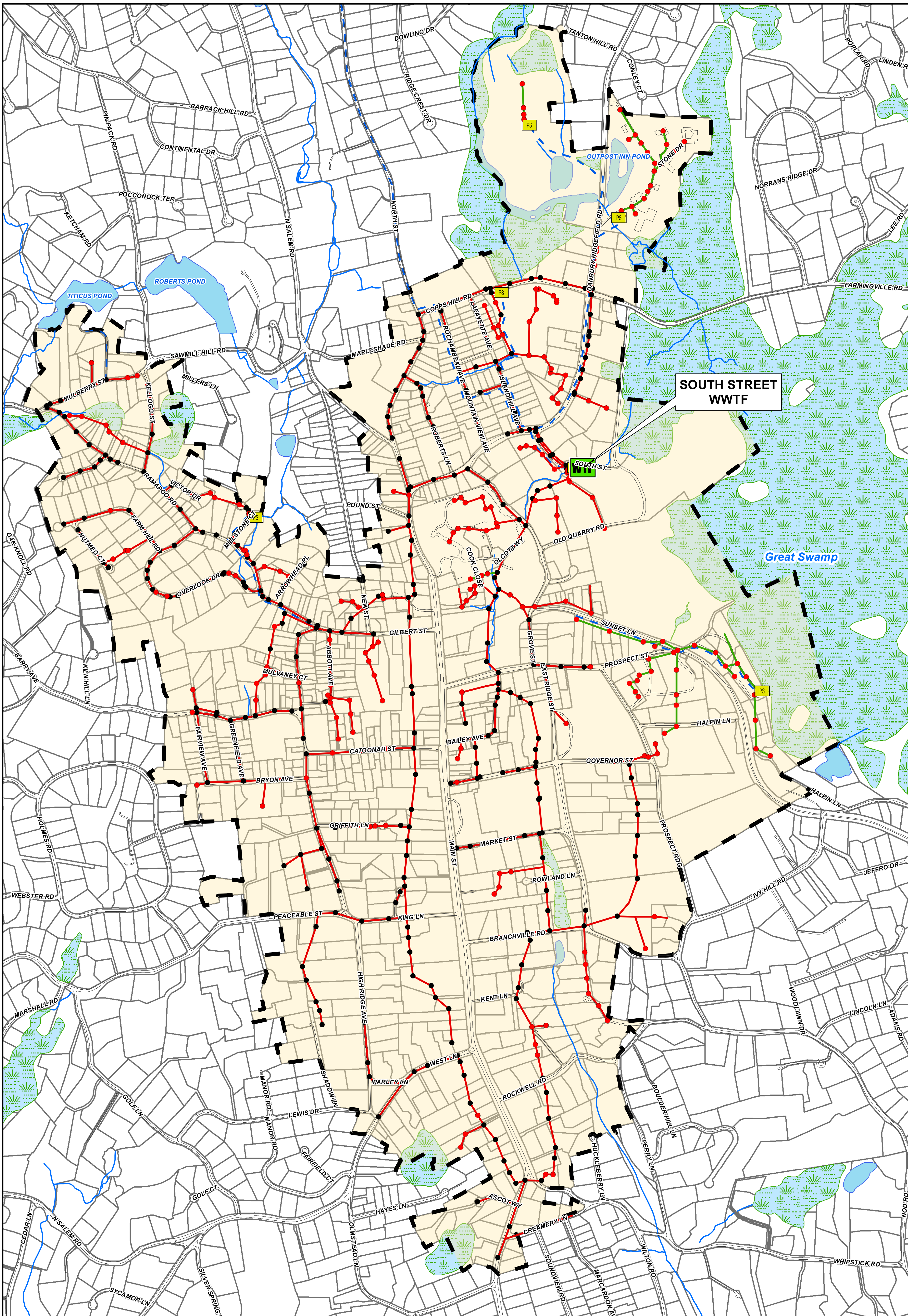
Pump Station Name	Pump Station Type	Pump Capacity (Gallons per Minute)	Year of Construction or Last Upgrade
South Street WWTF Influent PS	Duplex Submersible	680	2007
Copps Hill PS	Duplex Submersible	650	2007
Middle School PS	Duplex, Two-Stage Submersible	280	2003
Quail Ridge PS	Duplex Prefabricated Dry Pit	100	1985
Fox Hill PS	Duplex Submersible	300	2005
Ramapoo Road (Millstone Court) PS	Duplex Submersible	220	1998

History

Much of the Sewer District 1 collection system dates from 1902 when a gravity sewer system consisting of vitrified clay pipes was constructed to service the "village" or central section of Town. As the system was designed to flow by gravity, the sewers often followed watercourses, and cross under various brooks as the system conveys flow to the South Street WWTF. The original village collection system consisted of about 7.5 miles of sewer, and is reported to have an estimated 7.1 miles of service laterals.

The collection system has been expanded in stages over time since the construction of the original sewer system serving the village. In 1974, the Fox Hill Condominiums were constructed along with a privately owned gravity sewer system and connected into the village collection system through a privately owned pump station, and a 6 inch force main along Route 35, discharging into a 12 inch sewer at the intersection of Danbury Road and South Street. This pump station was reconstructed in 2005 with a new duplex submersible station, and while still owned by the Fox Hill Condominiums, it is operated by the WPCA.

In 1979, the Copps Hill and Peatt Park areas were sewered through construction of a system of gravity sewers discharging to the Copps Hill pump station located behind the Copps Hill Plaza. Collected wastewater is pumped through an 8 inch force main to a gravity sewer on Island Hill Avenue, which



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Drawn: BC 4/23/2015

Approved: AA 4/23/2015

Map Location

Legend

- WTP WWTF
- PS Pump Station
- Manhole
- Manhole - Not Field Located
- Municipal Sewer
- Private Sewer
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 500 1,000 2,000 Feet

FIGURE 2-1

WASTEWATER COLLECTION SYSTEM
SEWER DISTRICT 1

PHASE I WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 2-1 - Existing Sewer District 1.mxd

conveys the flow to a gravity sewer on South Street and ultimately to the WWTF. The original Copps Hill Pump Station was replaced in 2007 with a new duplex submersible wastewater pump station.

In 1997, the Ramapoo Road area was sewered through a network of gravity sewers conveying flow to a duplex submersible pump station located on Millstone Court. Collected wastewater is pumped through a 6 inch force main to the gravity sewer at the intersection of Gilbert Street and Ramapoo Road.

In 2003, the High School and the new Scotts Ridge Middle School were connected to the sewer system through construction of a duplex, two-stage pump station located behind the school complex and 4 miles of 6 inch force main that discharges directly to the South Street WWTF. No connections to the force main are permitted other than the school complex.

High Flow Issues

Due to the age of the collection system, Infiltration/Inflow (I/I) has historically been an issue at the WWTF since the 1960s. I/I is extraneous groundwater and surface water that enter the sewer system, occupying capacity and potentially overloading the collection system. The Town has undertaken previous efforts to locate and remove I/I sources. An initial effort was undertaken in the 1960s with smoke testing and subsequent grouting of leaking manholes and sewer pipe joints. The approximately 50 inflow sources identified by smoke testing in the 1960s were reportedly not removed, but the grouting program was found to be effective in reducing I/I.

In the mid-1980s, a television inspection program was undertaken on a portion of the system, and certain badly damaged sewer manhole to manhole reaches were replaced or lined. House-to-house inspections to locate sump pumps illegally connected to the sewer system were conducted in 1984 that identified 50 sump pumps connected to the sewer, and 20 had been confirmed to be removed. In the late 1980s, a number of leaking sewer mains in Sewer District 1 were lined using either a cured-in-place liner or a fold and form PVC liner. Manholes were also sealed and repaired to eliminate leakage.

During 2005 and 2006, due to the unusually wet weather in the fall of 2005 and the spring of 2006, the six-month moving average at the South Street WWTF average daily effluent flow reached 0.97 mgd. The NPDES permit for the South Street WWTF contains a requirement that if the six-month moving average daily flow exceeds 90 percent of the plant permitted capacity, in this case 0.9 mgd, the Town must prepare a plan to accommodate future increases in flow to the plant. To develop the plan, a district wide I/I analysis consisting of flow metering and television inspection of the district sewers was completed in 2005-2009. This was followed by a sewer rehabilitation project in 2010 that used a variety of methods to address defective and leaking pipes including cured in place liners, short liners for spot repairs, chemical grouting, and excavate and replace methods. In 2010, to address infrequent periods of high flows at the South Street WWTF Influent Pump Station, the Town installed a portable self priming pump that starts automatically as a supplement to the influent pumps. Since the sewer rehabilitation work has been completed, the frequency of operation of this backup pumping system has been reduced, indicating the effectiveness in reducing I/I.

South Street WWTF

From 1902 until 1973-74, collected wastewater in Sewer District 1 was treated using primary treatment and sand filtration at the location of the current WWTF on South Street. Treated effluent was discharged to the Great Swamp, the headwaters of the Norwalk River. The original primary treatment plant had an average daily flow capacity of 0.126 mgd.

In 1973-74, the WWTF was upgraded to secondary treatment using the extended aeration activated sludge process with an average daily flow capacity of 0.72 mgd. In the mid-1980s, the WWTF was periodically hydraulically overloaded due to excessive I/I, and failed to meet effluent permit limits. As a result, the Town was ordered by the State of Connecticut to undertake the necessary studies and plant upgrades to address the needs of the Town for the next 20 years. The State also issued a new NPDES permit for the facility which required seasonal ammonia removal, imposed more stringent limits on BOD_5

and total suspended solids, imposed a limit on chlorine residual and total phosphorus, and an effluent dissolved oxygen limit.

In response to the order issued by the State, the Town completed facilities planning which resulted in a recommendation to expand the South Street WWTF to a capacity of 1.0 mgd. During the period of the facilities plan and design of the expanded and upgraded plant, the State imposed a moratorium on new connections to the Sewer District 1 collection system from 1986 through 1989 when the Town executed the construction contract for the upgraded and expanded WWTF. In 1992 the upgraded and expanded WWTF was completed and is the system that exists today. Figure 2-2 shows a site plan of the existing South Street WWTF, Figure 2-3 presents a process flow diagram of the WWTF, and Figure 2-4 shows aerial photographs of the existing WWTF.

The South Street WWTF uses a single stage nitrification activated sludge process to provide advanced treatment. The WWTF consists of the following processes:

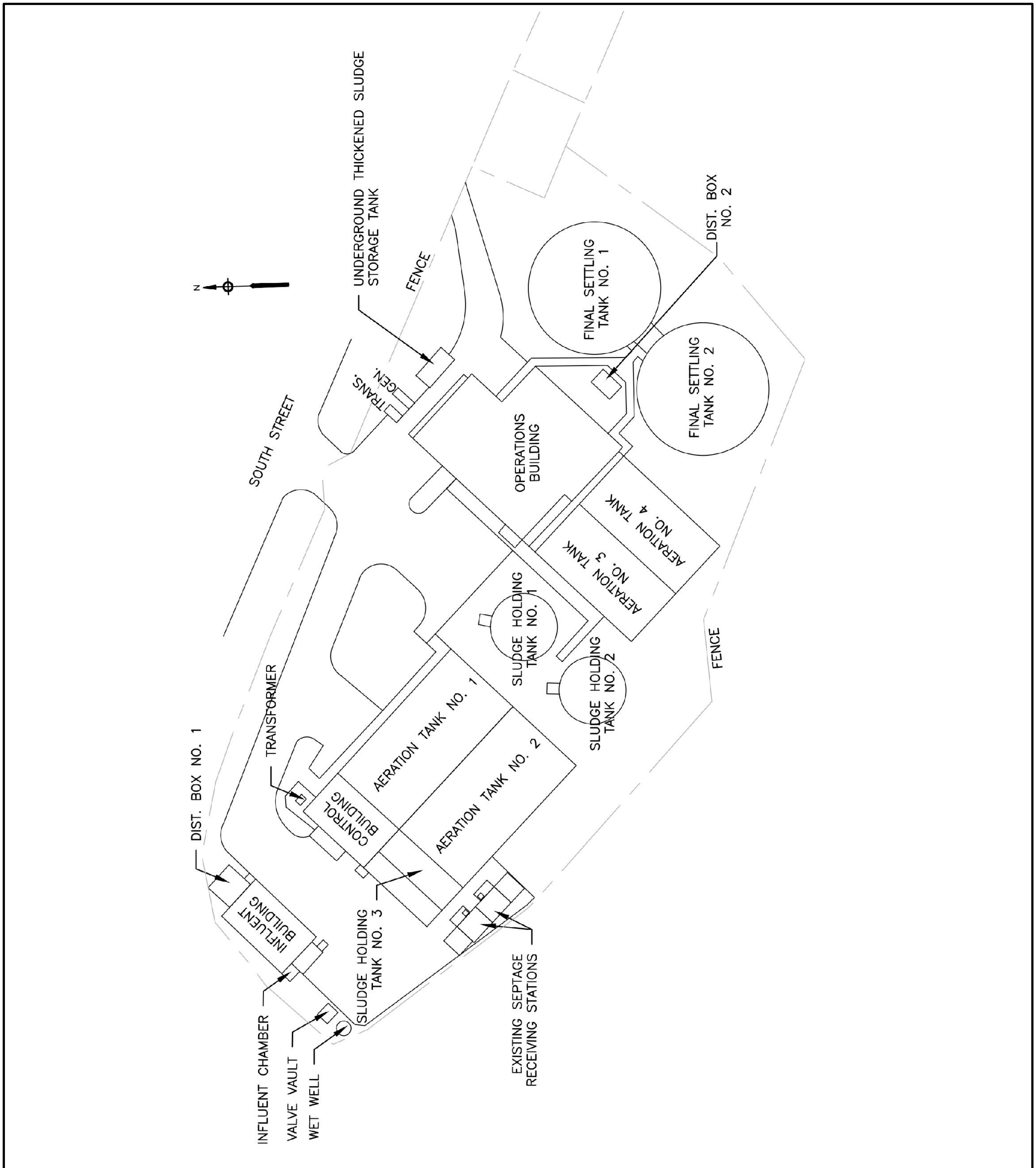
1. **Gravity Influent and Influent Pump Station**. Raw wastewater enters the WWTF by two means: gravity sewers and pumped discharges. Several gravity sewers convey wastewater collected from areas in town to the Influent Building. Some gravity sewers are lower in elevation than the Influent Building, and flows from these areas are pumped by the Influent Pump Station. The collected wastewater from the High School/Scotts Ridge Middle School is pumped directly to the Influent Building. The original Influent Pump Station was replaced in 2007 with a new submersible pump station due to deterioration of the concrete wet well. In addition to the Influent Pump Station, the Town has installed a trailer mounted portable self priming pump that starts automatically as a supplement to the influent pumps for the rare occasions when influent flows exceed the capacity of the Influent Pump Station.

2. **Influent Building**. Raw wastewater entering the Influent Building passes through a mechanically cleaned bar screen to remove large objects from the wastewater. The screened wastewater then enters a forced vortex grit chamber where grit, sand, and other abrasive materials are removed. Next, the wastewater flows through a comminutor which grinds and shreds larger solids so that they can be processed. Finally, the flow passes through a manually cleaned bar screen before it is discharged to the Flow Distribution Box No.1 where it is mixed with Return Activated Sludge (RAS) and can be directed to the Aeration Tanks.

3. **Aeration Tanks**. The heart of the activated sludge treatment process is the Aeration Tanks. In these tanks, microorganisms are continuously cultivated, circulated, and aerated to promote consumption of organic compounds in the wastewater. In the aeration tanks, biochemical oxygen demand is reduced, ammonia is converted to nitrate (nitrification) and nitrate is converted to nitrogen gas (denitrification). The plant has two aeration tanks, each divided into 4 bays, and equipped with mechanical surface aerators which transfer oxygen into the mixture of wastewater and microorganisms (termed mixed liquor) as well as mixing the contents of the aeration tanks. The mixed liquor flows over the aeration tank weirs and into Flow Distribution Box No. 2 for distribution to the Final Settling Tanks. To precipitate phosphorus present in the wastewater, aluminum sulfate (alum) is seasonally added to the mixed liquor.

4. **Final Settling Tanks.** The mixed liquor flows into the two circular final settling tanks. The low velocity in the settling tanks allows the separation of the solids and liquids in the mixed liquor through sedimentation. Settled solids are collected by the rotating circular collector mechanisms and a portion of the solids are pumped to Flow Distribution Box No. 1 as RAS where it is mixed with the incoming wastewater. Any floating scum or other materials are skimmed off and removed. The clarified wastewater flows over the weirs at the Final Settling Tanks and then enters the sand filters for further treatment.

5. **Sand Filters.** The wastewater from the Final Settling Tanks enters the sand filters in the basement of the Operations Building where it flows from the bottom up to the top, while at the same time sand is recirculated within the filter from the top to the bottom of the filters. Passing the



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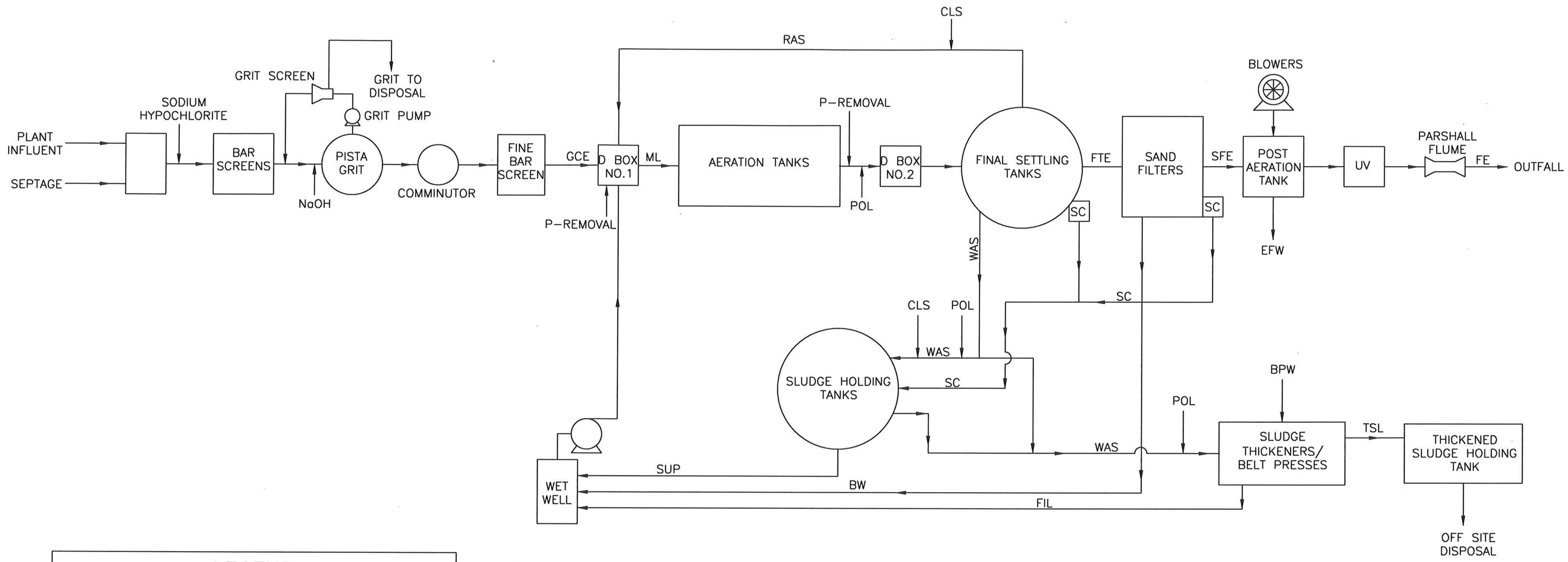
Drawn:	BC 4/24/2015
Approved:	AA 4/24/2015

FIGURE 2-2

SOUTH STREET WWTF
SITE PLAN

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT



LEGEND

FE	FINAL EFFLUENT
BW	BACKWASH
SC	SCUM
FIL	FILTRATE
POL	POLYMER
RAS	RETURN ACTIVATED SLUDGE
WAS	WASTE ACTIVATED SLUDGE
SUP	SUPERNATANT
GCE	GRIT CHAMBER EFFLUENT
ML	MIXED LIQUOR
FTE	FINAL SETTLING TANK EFFLUENT
SFE	SAND FILTER EFFLUENT
BPW	BELT PRESS WASH WATER
EFW	EFFLUENT FLUSHING WATER
CLS	CHLORINE SOLUTION
TSL	THICKENED SLUDGE

AECOM

Drawn: BC 4/14/2015

Approved: AA 4/14/2015

FIGURE 2-3

SOUTH STREET WWTF
PROCESS FLOW SCHEMATIC

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 2-3 - South Street WWTF Schematic.mxd

Photographs from Bing Maps

Figure 2-4. South Street WWTF Aerial Photographs

wastewater through the filters removes solids by trapping the solids in the sand media. The sand at the bottom of the filter is continuously cleaned and recirculated to the top of the filters. The plant has six filter cells, each equipped with two filter modules.

6. **Post Aeration.** After leaving the sand filters, the wastewater is aerated in the Post Aeration Tank in the basement of the Operations Building with submerged air diffusers to increase the dissolved oxygen content prior to the Ultraviolet Disinfection system.

7. **Ultraviolet (UV) Disinfection.** After the wastewater leaves the Post Aeration Tank, flow enters a long, horizontal, open channel and flows past racks of submerged bulbs in the basement of the Operations Building. These bulbs emit light in the ultraviolet spectrum which either kills or inactivates disease causing microorganisms. This system operates only in the warmer months of the year as required by the WWTF's NPDES permit. Following the UV Disinfection system, the effluent flow is measured in a Parshall Flume prior to the discharge of the treated effluent to the Great Swamp.

8. **Solids Handling.** Most of the activated sludge is returned to Distribution Box No.1 to mix with the incoming wastewater before entering the aeration tanks. Periodically, excess activated sludge must be removed (wasted) from the system to maintain the proper balance of the treatment process. There are two circular waste sludge holding tanks. Excess, or waste, sludge is pumped to a belt filter press/thickener unit in the Operations Building. Most of the water is removed from the sludge, and the thickened sludge is stored in a holding tank, until it is pumped into a tanker truck and disposed of at the incinerator at the Mattabassett District in Cromwell.

The South Street WWTF in its current form has been in operation since 1992 with minor equipment replacements as needed to keep the many mechanical systems operating.

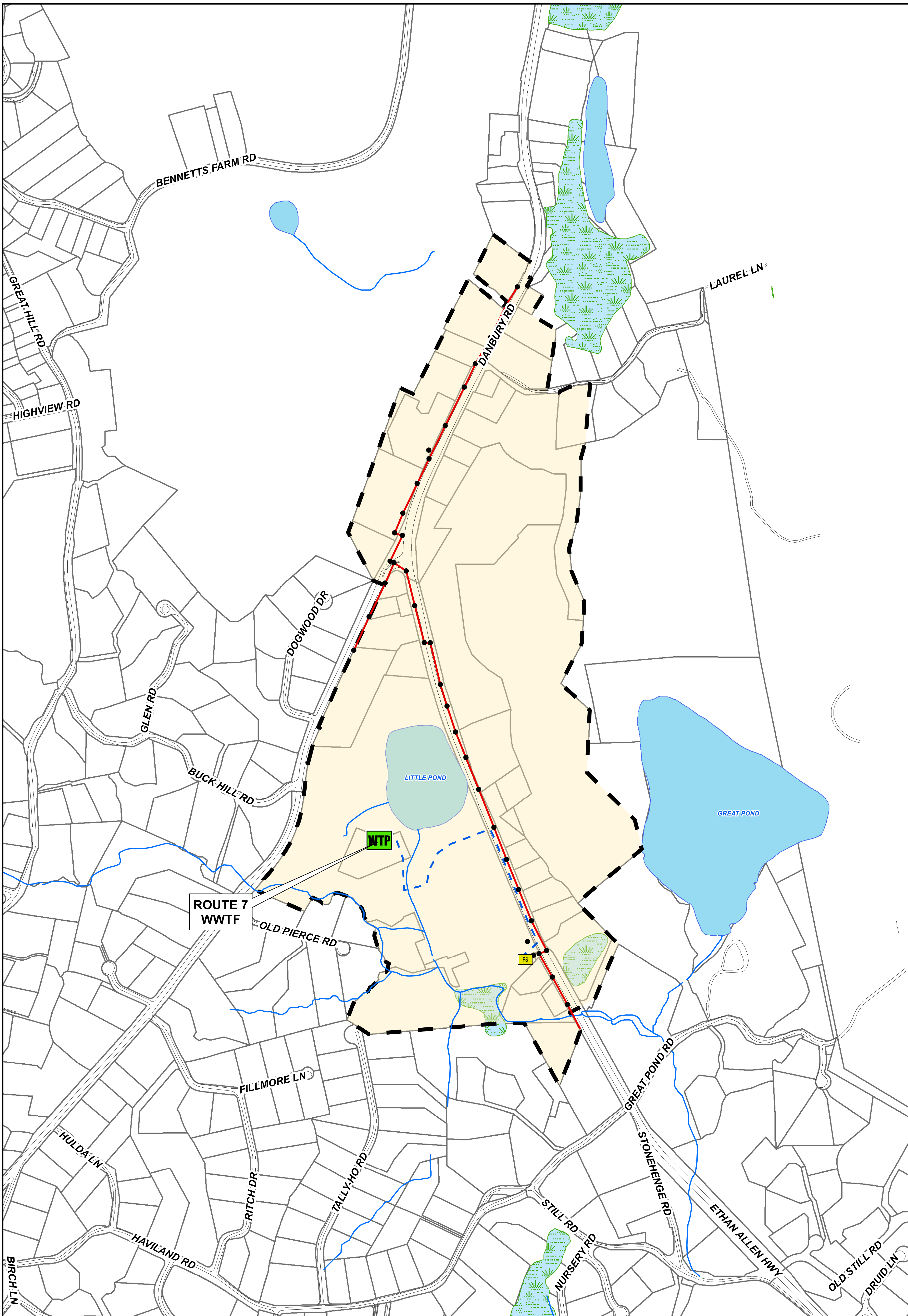
SEWER DISTRICT 2

Sewer District 2 is located near the intersection of Route 35 and Route 7. Figure 2-5 shows the extent of the collection system and service area for Sewer District 2. Sewer District 2 includes a wastewater collection system that collects wastewater from approximately 170 acres which represents less than 1 percent of the town's area. The gravity sewer system consists of approximately 6,300 feet of sewers ranging in size from 8 inches in diameter to 10 inches, with approximately 180 billed service accounts. Most of the collection system conveys wastewater by gravity to a single pump station near the Route 7 WWTF where the collected wastewater is pumped to the Route 7 WWTF. The Route 7 Pump Station houses duplex pumps in a prefabricated enclosure, and each pump has a capacity of 500 gallons per minute. This pump station has not been upgraded since it was constructed in 1985.

History

In 1978, the State of Connecticut issued the Town an order to abate pollution from failing on-site septic systems in the Route 7 and Route 35 area. At that time, the only wastewater treatment facility in this area was serving the Wells-Benrus (later Perkin-Elmer, now Ponds Edge Professional Park) facility. Other properties in this area were served by on-site septic systems. The Wells-Benrus WWTF was a 40,000 gallon per day extended aeration facility constructed in 1967. In 1979, in response to the State Order, a Facilities Plan was prepared by the Town for the Route 7/Route 35 Area. Subsequent to the Facilities Plan, the collection system, Route 7 Pump Station, and the Route 7 WWTF were constructed. The old Wells-Benrus WWTF was then abandoned, and the Wells-Benrus facility was connected into the new Route 7 WWTF.

The planning and funding mechanism for the existing Route 7 WWTF serving Sewer District 2 was completed using a different approach than that for Sewer District 1. To fund the construction of the sewer system and WWTF, all of the parcels to be served formed the basis for Sewer District 2, and each parcel was allocated a flow allowance. The owner of each parcel then purchased the allocated flow allowance which represented their share of the plant capacity. The State paid 55 percent of the cost for the WWTF,



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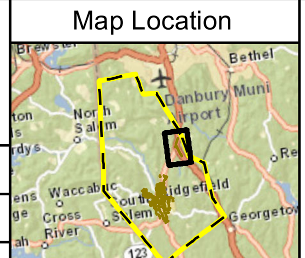
Approved: AA 4/23/2015

Map Location



Legend

WTP WWTF	Roads
PS Pump Station	Force Main
Manhole	Streams
Municipal Sewer	Swamps
	Waterbodies
	Parcel Polygon

N

0 300 600 1,200 Feet

FIGURE 2-5

WASTEWATER COLLECTION SYSTEM
SEWER DISTRICT 2

PHASE I WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 2-5 - Existing Sewer District 2.mxd

as well as 30 percent of the cost of the collection system serving the area. The remaining costs were borne by the property owners of the parcels to be served by the system. The Route 7 WWTF and collection system was then constructed by the Town. Nearly all of the parcels in Sewer District 2 have since connected to the sewer system, although many of the parcels have not been developed at the density of development permitted by current zoning of the District. As a result, all of the current Route 7 WWTF capacity has been allocated to the existing users, with no capacity available for extension of the collection system.

Route 7 WWTF

The Route 7 WWTF uses rotating biological contactors (RBCs) to provide advanced treatment and has an average daily flow capacity of 0.12 mgd. Figure 2-6 shows a site plan of the existing Route 7 WWTF, Figure 2-7 presents a process flow diagram of the WWTF, and Figure 2-8 shows aerial photographs of the existing WWTF. All flow that enters the Route WWTF is pumped by the Route 7 pump station through an eight inch force main to the headworks of the WWTF. The WWTF consists of the following processes:

1. **Headworks.** Influent wastewater pumped by the Route 7 Pump Station enters the Headworks. Grit is removed in an aerated grit chamber which is equipped with a mechanical grit collector system. Flow then passes through a comminutor which shreds and grinds large solids for subsequent treatment. Flow then enters a distribution box for distribution to the Primary Settling Tanks.

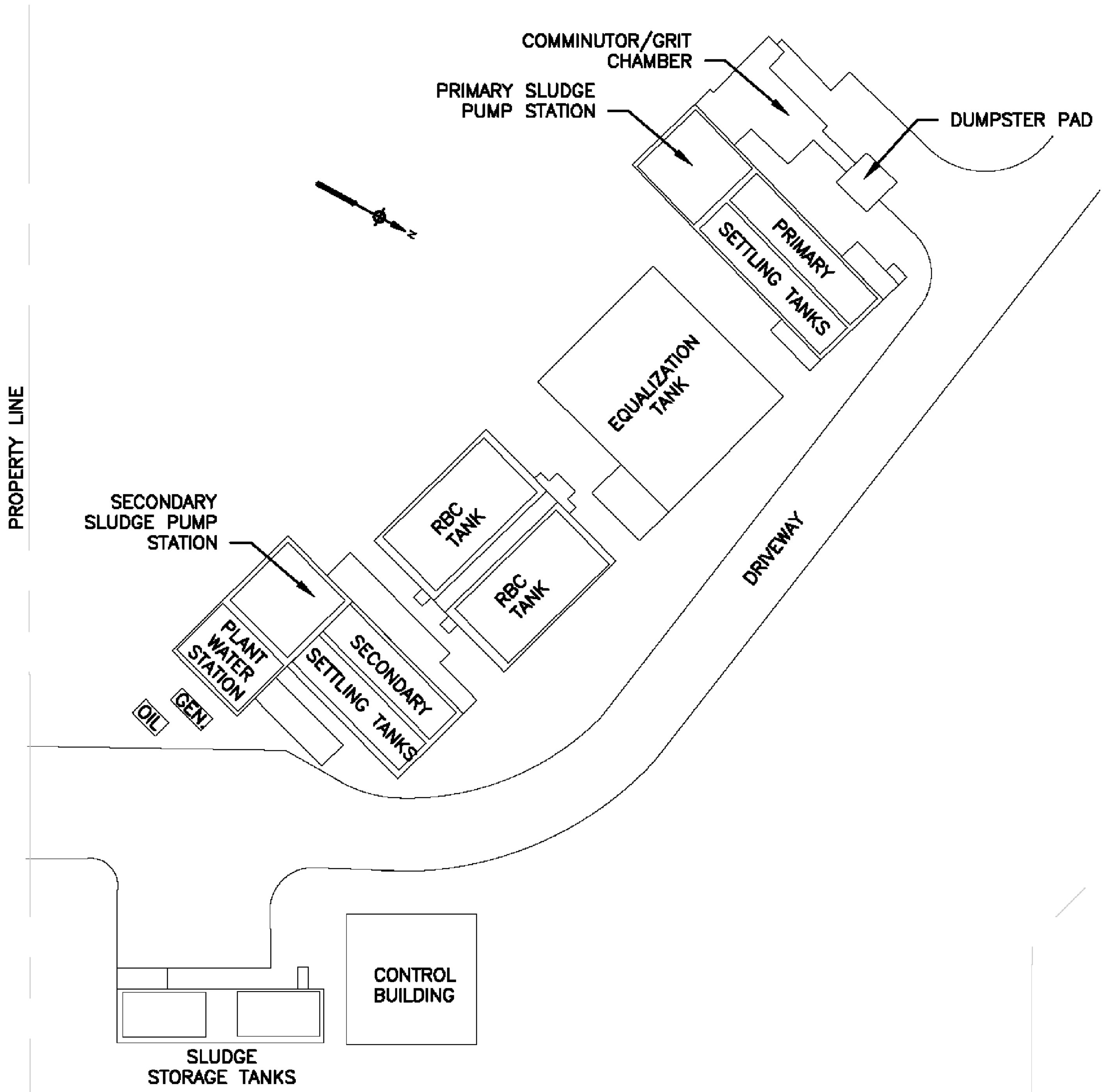
2. **Primary Settling Tanks.** From the distribution box, the flow enters the two rectangular Primary Settling Tanks. In these tanks, the low velocity allows the organic solids to separate through sedimentation. Chain and flight collectors at the top and bottom of the tanks collected floating scum and settled solids. Clarified effluent from the Primary Settling Tanks flows over weirs and is conveyed to the Flow Equalization Tank.

3. **Flow Equalization Tank.** Primary effluent enters the Flow Equalization Tank where it is aerated. The purpose of the tank is to dampen out the peak flows and provide a more consistent flow rate to the Rotating Biological Contactors downstream.

4. **Rotating Biological Contactors (RBCs).** The RBCs are the heart of the treatment process at the Route 7 WWTF. RBCs are a fixed film treatment process that consists of a series of discs mounted in parallel on a shaft, and the shaft slowly rotates the discs through the wastewater. The Route 7 WWTF has two trains of RBCs with each train having four RBC stages that are housed in fiberglass covered tanks. Microorganisms attach to the discs, and as the discs rotate in the air the microorganisms uptake oxygen, and when submerged in the wastewater the organism assimilate and remove organic matter from the wastewater. Excess microorganism growth sloughs off the discs and is removed in the Secondary Settling Tanks.

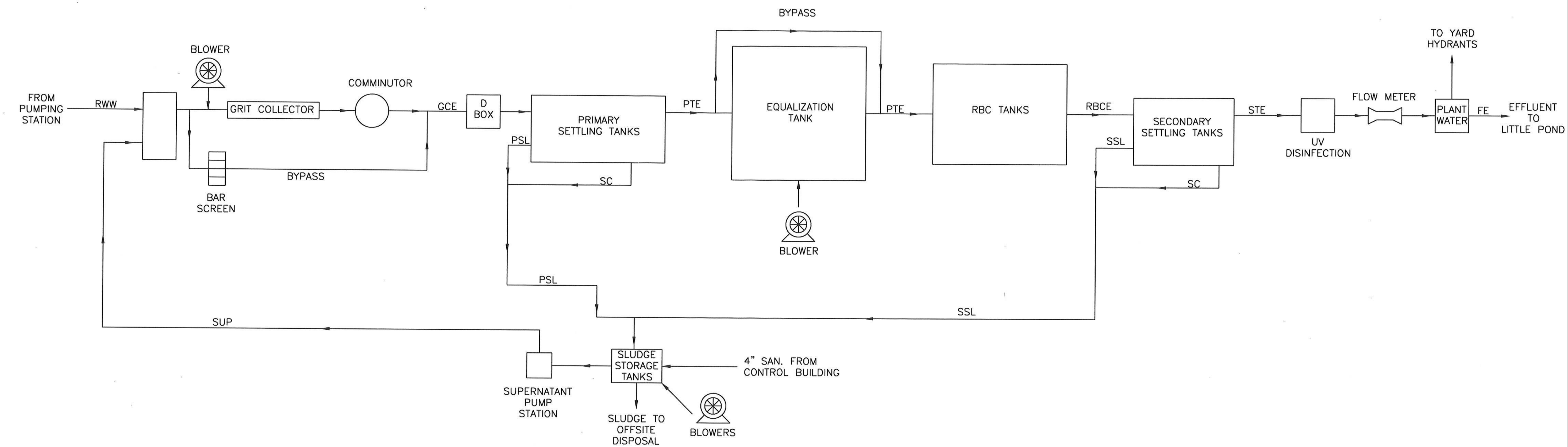
5. **Secondary Settling Tanks**. In the two rectangular Secondary Settling Tanks the low velocity allows the microorganisms to separate from the treated water through sedimentation. Chain and flight collectors at the top and bottom of the tanks collected floating scum and settled solids. Clarified effluent from the Secondary Settling Tanks flows over weirs and is conveyed to the UV Disinfection Unit.

6. **UV Disinfection.** The Secondary Settling Tank effluent is conveyed to the UV Disinfection system, a horizontal, open channel and flows past racks of submerged bulbs. These bulbs emit light in the ultraviolet spectrum which either kills or inactivates disease causing microorganisms. This system operates only in the warmer months of the year as required by the WWTF's NPDES permit. Treated effluent passes through a flow meter, and then is discharged to Little Pond, which ultimately discharges to the Norwalk River.



Drawn: BC 4/24/2015	FIGURE 2-6
Approved: AA 4/24/2015	ROUTE 7 WWTF SITE PLAN
	PHASE 1 WASTEWATER FACILITIES PLAN RIDGEFIELD, CT

X:\Ridgefield, CT\60299267 - Ridgefield Phase 1 Fac Plan\500 Deliverables\507 Facilities Plan Report\Section 2 Figures\Figure 2-6 - Route 7 WWTF Site Plan1 sme.mxd



LEGEND

RWW	RAW WASTEWATER
GCE	GRIT CHAMBER EFFLUENT
FE	FINAL EFFLUENT
PTE	PRIMARY SETTLING TANK EFFLUENT
STE	SECONDARY SETTLING TANK EFFLUENT
SC	SCUM
PSL	PRIMARY SLUDGE
SSL	SECONDARY SLUDGE
RBCE	ROTATING BIOLOGICAL CONTACTOR EFFLUENT
SUP	SUPERNATANT
SAN	SANITARY DRAIN

AECOM

Drawn: BC 4/14/2015
Approved: AA 4/14/2015

FIGURE 2-7

ROUTE 7 WWTF
PROCESS FLOW SCHEMATIC

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 2-7 - Route 7 WWTF Schematic.mxd

Photographs from Bing Maps

Figure 2-8. Route 7 WWTF Aerial Photographs

7. **Solids Handling.** Solids from the Primary Settling Tanks and the Secondary Settling Tanks is collected and pumped to sludge storage tanks which are aerated to mix and introduce oxygen into the stored solids. Periodically, the collected solids are removed and disposed of at the incinerator at the Mattabassett District in Cromwell.

The Route 7 WWTF in its current form has been in operation since 1985 with minor equipment replacements as needed to keep the many mechanical systems operating. In about the year 2000, the RBC units were replaced due to deteriorated conditions.

REGULATORY REQUIREMENTS

Discharges from both of the Town's WWTFs are regulated by the US Environmental Protection Agency (EPA) through the National Pollutant Discharge Elimination System (NPDES) permits. The EPA has delegated the responsibility for administering permits in Connecticut to the Connecticut Department of Energy and Environmental Protection (DEEP). The existing NPDES permits for both plants expired in 2009 and the Town submitted renewal applications as required by the program rules. The DEEP deferred issuing new permits for both WWTFs until DEEP's Phosphorus Reduction Strategy for Inland Non-Tidal Waters could be developed and finalized. In the meantime, the expired NPDES permits were administratively continued and remained in effect.

As a result of the implementation of the strategy, in October 2014 DEEP issued a new NPDES permit for the Route 7 WWTF that includes an effluent phosphorus limit that the existing treatment facility cannot meet without modifications. The permit also includes a compliance schedule that defers the implementation of the new limit until August 2019 to allow time for the Town to complete the ongoing facilities planning effort and implement modifications to the Route 7 WWTF to meet the new phosphorus limit. The new permit for the Route 7 WWTF also includes a change in the indicator organism used to monitor disinfection performance from fecal coliform to escherichia coli. It is anticipated that the South Street WWTF permit, once issued, will also contain a more stringent limit on effluent phosphorus as well as a compliance schedule to meet the new limit, and potentially, other changes from the current permit limits. It is anticipated that the existing WWTFs will not be able to meet their future permit limits without some modifications.

A condition of the NPDES permit for both plants requires if the 180 day rolling average for the plant average daily influent flow exceeds 90 percent of the design flow rate, the Town shall develop and submit to the DEEP a plan to accommodate future increases in flow to the plant. Historically, the South Street WWTF has operated below the design capacity of 1.0 mgd, except for occasional storm induced high flows. However, during extended wet periods with high groundwater conditions, the influent average daily flow at the South Street WWTF has exceeded both the 90 percent flow threshold as well as the design capacity of 1.0 mgd. The Route 7 WWTF has not experienced any periods where flows have approached the design capacity due to the sewer system being newer and constructed of PVC pipe with tighter joints than the older sewers in Sewer District 1.

Recognizing that to meet the lower effluent phosphorus limits in the DEEP Phosphorus Reduction Strategy will require significant financial investments by the affected communities in upgrading their wastewater treatment facilities, in 2013 the state created a new funding program. This program, which increases the available grant funds for phosphorus removal upgrades at WWTFs with low effluent phosphorus limits from the 30 percent of the eligible costs then available under the Clean Water Fund to 50 percent, is only available to a limited number of communities. Initially, only three communities of the eleven eligible communities with phosphorus limits of less than 0.2 mg/l were eligible to receive the 50 percent grant funds. The legislation creating this funding program was subsequently revised in 2014 to expand the number of communities that can be eligible for the 50 percent grant program. Ridgefield is one of the communities that will be eligible for the funding assistance under the current program provided it meets the schedule criteria in the legislation. The 50 percent funding assistance, as currently written in the legislation, will be made available to the communities that complete the design of the WWTF upgrade for phosphorus removal and execute a construction contract by July 1, 2018. If there are more communities that meet the schedule criteria, then the funds will be competitively awarded to the WWTFs with the lowest permitted discharge limit for phosphorus, and then to those WWTFs that will remove the greatest amount of phosphorus in pounds/year. However, there have been a number of bills filed in the current legislative session to further change and revise this funding program.

CHAPTER THREE
EXISTING WASTEWATER FLOWS AND LOADS

Three years of monthly operating reports, daily monitoring reports, and pump station data (from July 1, 2010 to June 30, 2013) from the South Street and Route 7 WWTFs and collection systems were collected and reviewed. In addition, for each WWTF, a one week in-plant sampling program was performed in the fall of 2013 to supplement the data noted above. All of this data was used to evaluate the flows, infiltration and inflow, pollutant loading and effluent concentrations for each WWTF and sewer district. This data is summarized in Technical Memorandum No. 7 - Future Flow and Loading Evaluation, in Appendix C, Technical Memorandum No. 6 – Plant Capacity Evaluation, in Appendix D, and In-Plant Sampling Data Summary in Appendix E. A summary of the existing wastewater flows, infiltration and inflow, and loads to each WWTF as well as a comparison to the NPDES discharge permit limits and design valves are summarized below.

SOUTH STREET WWTF EXISTING AND DESIGN FLOWS AND LOADS

Flows

The existing influent flows and concentrations of wastewater constituents for the South Street WWTF for the period between July 1, 2010 and June 30, 2013 were evaluated. The current annual average daily flow was approximately 0.85 mgd, the maximum month flow was 1.83 mgd, the maximum dally flow was 4.51 mgd, and the maximum instantaneous peak flow was 5.88 mgd. Based on the review of the three years of data, the WWTF exceeded the permitted threshold of the average flow of the last 180 days being greater than 90 percent of the design average WWTF flow 40 percent of the time.

Infiltration and Inflow

Infiltration is the leakage of groundwater into the collection system, and inflow is the entry of surface water into the collection system. The amount of I/I in a collection system depends on the length of the sewer, and the number of joints and manholes, and condition of the system. The I/I rate also varies depending on the groundwater level, the proximity of water courses, the porosity of the soil, and other topographic and geological features. Infiltration and inflow (I/I) for Sewer District 1 were assessed using flows recorded at the WWTF and pump station data throughout the collection system. Significant I/I was shown to be present in the collection system based on the flow records. Table 3-1 presents a summary of the wastewater, infiltration, and inflow received at the South Street WWTF. More detailed information on the infiltration and inflow can be found in Technical Memorandum No. 7 – Future Flows and Loads included in Appendix C.

TABLE 3-1. SOUTH STREET WWTF CURRENT INFILTRATION AND INFLOW SUMMARY

Flow Component	Average Daily Flow (gpd)	Peaking Factor	Peak Flow (gpd)
Current Wastewater	592,000	2.8	1,658,000
Current Infiltration	201,000	1.81	363,000
Current Inflow	57,000	-	3,859,000
Total	850,000		5,880,000

Pollutant Loads

A preliminary analysis of influent concentration and loading data for the primary pollutants (BOD_5, TSS, Total Kjeldahl Nitrogen (TKN), and TP) for the July 1, 2010 and June 30, 2013 period showed a great deal of variability. This variability was believed to contain some unrepresentative data that was potentially attributed to the septage received at the WWTF and its impact on the influent composite samples. As a result, the data for the reporting period was truncated based on the review of plotted data histograms, engineering judgment, and textbook references. Table 3-2 presents the range of reported concentration data and the truncated concentration data set used to eliminate potentially unrepresentative data. The impact of septage and the data truncation are described in more detail in Technical Memorandum No. 6 in Appendix D.

TABLE 3-2. SOUTH STREET WWTF PRIMARY POLLUTANT DATA TRUNCATION

Primary Pollutant	Data Range Reported	Truncated Data Set
BOD_5	53 mg/l - 480 mg/l	100 mg/l - 400 mg/l
TSS	11 mg/l - 2,420 mg/l	75 mg/l - 500 mg/l
TKN	10 mg/l - 73 mg/l	10 mg/l – 50 mg/l
TP [1]	1.2 mg/l - 9.5 mg/l	1.2 mg/l - 9.5 mg/l

1. The total phosphorus data was not truncated

Table 3-3 summarizes the South Street WWTF flow and wastewater constituent data for the July 1, 2010 to June 30, 2013 reporting period for the WWTF's truncated influent, primary effluent, and final effluent including annual average day and maximum month conditions.

TABLE 3-3. SOUTH STREET WWTF FLOW AND LOADING SUMMARY (JULY 2010 TO JUNE 2013)

Parameter	Annual Average Day	Max Month Peaking Factor	Max Month [1]
Influent			
Flow (mgd)	0.85	2.15	1.83
TSS (mg/l)	232		181
TSS (lb/d)	1,643	1.69	2,776
BOD_5 (mg/l)	219		158
BOD_5 (lb/d)	1,550	1.55	2,405
Total Kjeldahl Nitrogen (mg/l)	24.8		16.3
Total Kjeldahl Nitrogen (lb/d)	176	1.41	249
Total Phosphorus (mg/l)	4.0		3.1
Total Phosphorus (lb/d)	28.4	1.67	47.4
Zinc (kg/d)	0.799	1.81	1.446
Effluent Discharged			
TSS (mg/l)	2.1		2.3
TSS (lb/d)	14.8	2.34	34.7
BOD_5 (mg/l)	2.2		2.1
BOD_5 (lb/d)	15.3	2.14	32.7
Ammonia Nitrogen (mg/l)	0.5		1.0
Ammonia Nitrogen (lb/d)	3.8	3.87	14.7
Total Nitrogen (mg/l)	5.9		4.2
Total Nitrogen (lb/d)	40.7	1.58	64.3
Total Phosphorus (mg/l)	0.2		0.3
Total Phosphorus (lb/d)	1.4	3.29	4.6
Zinc (kg/d)	0.147	1.33	0.196

1. Due to the limited number of daily samples collected for analysis, the maximum month loading conditions were based on the 92nd percentile of all of the data while the maximum month concentration data was back calculated from the maximum month loading conditions and the maximum month flow.

Design Flow and Loading Comparison

Flows. During the evaluation period of July 1, 2010 to June 30, 2013, the average annual flow was 0.85 mgd, or 15 percent below the design flow of 1.0 mgd. The maximum month flow was 1.83 mgd versus the design maximum month of 1.9 mgd. Also, there were two instances in March 2011 where the total daily flow exceeded the 4.1 mgd peak design flow rate, and twenty-one instances where the maximum recorded daily flow exceeded 4.1 mgd peak design flow rate.

Loads. The loads from the three years of data evaluated are presented below in Table 3-4 and compared to the design loadings of the WWTF. Based on the comparison, the plant is slightly under loaded organically and more significantly under loaded from a solids and nitrogen standpoint. Based on the current flows to the WWTF, the influent organic concentrations are similar to the design concentrations while the TKN and TSS concentrations are slightly less. This may have been the result of the data truncation as discussed previously.

TABLE 3-4. SOUTH STREET WWTF DESIGN VERSUS CURRENT LOADING COMPARISON

Pollutant	Design Load [1]	Current Loads	Current Percent of Design Load
BOD_5			
Annual Average	2,000 lbs/day	1,550 lbs/day	78%
Maximum Month	3,000 lbs/day	2,405 lbs/day	80%
TSS			
Annual Average	2,900 lbs/day	1,643 lbs/day	57%
Maximum Month	4,300 lbs/day	2,776 lbs/day	65%
TKN			
Annual Average	360 lbs/day	176 lbs/day	49%
Maximum Month	500 lbs/day	249 lbs/day	50%

1. Design loads from the November 1987 Report on Wastewater Treatment and Sewer System Rehabilitation Needs prepared by Stearns and Wheler.

Effluent Concentration and Permit Limit Comparison

The following is a summary of the comparison of the WWTF effluent concentrations/loads and the NPDES discharge limits by specific wastewater constituent.

BOD_5. The effluent BOD_5 concentrations were well below permitted concentrations (average monthly concentration of 10 mg/l between April 1st to October 31st and 20 mg/l between November 1st and March 31st) as well as the required 85% removal for the period reviewed.

TSS. The effluent TSS effluent concentrations were also well below the permit concentration limits (average monthly concentration of 10 mg/l between April 1st to October 31st and 20 mg/l between November 1st and March 31st) as well as the required 85 percent removal for the period reviewed.

Ammonia. The effluent ammonia concentrations were also well below the permit concentration limits (varies by month between 1.6 mg/l to 7.3 mg/l during the April 1st to October 31st ammonia permit season) for the period reviewed.

Total Phosphorus. The effluent total phosphorus concentrations were well below permitted concentrations (average monthly concentration of 1.0 mg/l between May 1st to September 30th) for the period reviewed

Zinc. The effluent zinc loads were well below permitted concentrations (average monthly load of 1.95 kg/d and maximum day load of 0.326 kg/day) for the majority of the reporting period. There were two average month exceedances in March 2011 and March 2013 with average monthly concentrations of

0.21 kg/day and 0.20 kg/day, respectively. It should be noted that these were spring high flow months (average flow of 1.73 mgd and 1.11 mgd, respectively) which contributed to the high mass in the effluent. There was also one maximum day load exceedance on March 12, 2013 with a load of 0.345 kg/day when the daily flow to the WWTF was 2.07 mgd.

ROUTE 7 WWTF EXISTING AND DESIGN FLOWS AND LOADS

Flows

The existing influent flows and concentrations of wastewater constituents for the Route 7 WWTF for the reporting period between July 1, 2010 and June 30, 2013 were evaluated. The current annual average daily flow was approximately 0.053 mgd with maximum month flow of 0.079 mgd, a maximum dally flow of 0.162 mgd and an instantaneous peak flow of 0.357 mgd. Based on the review of the data, the WWTF never exceeded the permitted threshold of the average flow of the last 180 days being greater than 90 percent of the design average WWTF flow.

Infiltration and Inflow

Infiltration and inflow (I/I) were assessed using flows recorded at the WWTF and the Route 7 Pump Station data. Significant I/I was shown to be present in the Sewer District 2 collection system based on the flow records. Table 3-5 presented a summary of the wastewater, infiltration, and inflow received at the Route 7 WWTF.

TABLE 3-5. ROUTE 7 WWTF CURRENT INFILTRATION AND INFLOW SUMMARY

Flow Component	Average Daily Flow (gpd)	Peaking Factor	Peak Flow (gpd)
Current Wastewater	33,000	3.0	99,000
Current Infiltration	21,000	1.79	37,600
Current Inflow	-	-	223,400
Total	54,000		360,000

Pollutant Loads

Table 3-6 summarizes the Route 7 WWTF flow and wastewater constituent data for the July 1, 2010 to June 30, 2013 reporting for the WWTF's influent, primary effluent, and final effluent including annual average day and maximum month conditions.

Design Flow and Loading Comparison

Flows. The current flows to the WWTF are significantly lower than the design flows to the plant of 0.12 mgd and a peak hourly flow of 0.72 mgd for the processes upstream of the equalization tank and a peak hourly flow of 0.30 mgd downstream of the equalization tank. It should be noted that the WWTF does not currently operate the equalization tank in an equalization mode but allows the flow to pass through the tank and exit the tank through an overflow pipe at the top of the tank.

Loads. Due to the limited records on the WWTF's design conditions, a direct comparison of the current influent loadings to the design influent loading could not be made. However, the contract specifications from the 1984 construction include design criteria for the rotating biological contactors. Per the contract documents, the RBCs were specified to treat 0.12 mgd of primary effluent with BOD_5 and TSS concentrations of 275 mg/l. Comparing that design criteria to the current primary effluent concentrations as presented in Table 3-6, the RBCs are currently under loaded for both TSS and BOD_5. Also assuming the primary settling tanks were intended to remove approximately 35 percent of the influent BOD_5

and 50 percent of the influent TSS, this would have meant the original influent wastewater constituent concentrations for BOD_5 and TSS would have been approximately 425 mg/l and 550 mg/l. Again, comparing these back- calculated design concentrations with the current influent wastewater concentrations in Table 3-6 reinforces that the WWTF is currently under loaded.

TABLE 3-6. ROUTE 7 WWTF FLOW AND LOADING SUMMARY (JULY 2010 TO JUNE 2013)

Parameter	Annual Average Day	Max Month Peaking Factor	Max Month[1]
Influent			
Flow (mgd)	0.053	1.49	0.079
TSS (mg/l)	226		199
TSS (lb/d)	102	1.28	131
BOD_5 (mg/l)	280		263
BOD_5 (lb/d)	124	1.40	173
Total Phosphorus (mg/l)	5.98		5.84
Total Phosphorus (lb/d)	2.71	1.42	3.85
Ortho-Phosphate (mg/l)	3.28		2.94
Ortho-Phosphate (lb/d)	1.46	1.33	1.94
Primary Effluent			
TSS (mg/l)	109		139
TSS (lb/d)	49.3	1.86	91.5
BOD_5 (mg/l)	180		182
BOD_5 (lb/d)	81.8	1.47	120
Ammonia Nitrogen (mg/l)	19.7		17.8
Ammonia Nitrogen (lb/d)	8.91	1.31	11.7
Effluent Discharged			
TSS (mg/l)	2.62		4.46
TSS (lb/d)	1.17	2.51	2.94
BOD_5 (mg/l)	4.20		5.42
BOD_5 (lb/d)	1.89	1.89	3.57
Ammonia Nitrogen (mg/l)	0.52		0.90
Ammonia Nitrogen (lb/d)	0.24	2.46	0.59
Total Phosphorus (mg/l)	5.09		5.00
Total Phosphorus (lb/d)	2.29	1.44	3.29
Ortho-Phosphate (mg/l)	4.05		3.79
Ortho-Phosphate (lb/d)	1.82	1.37	2.50

1. Due to the limited number of daily samples collected for analysis, the maximum month loading conditions were based on the 92nd percentile of all of the data while the maximum month concentration data was back calculated from the maximum month loading conditions and the maximum month flow.

Effluent Concentration and Permit Limit Comparison

The following is a summary of the comparison of the WWTF effluent concentrations/loads and the NPDES discharge limits by specific wastewater constituent. Note data on total phosphorus was not in the Route 7 WWTF NPDES permit for the data set reviewed since there was no phosphorus limit previously.

BOD_5. The effluent BOD_5 concentrations were well below permitted concentrations (average monthly concentration of 20 mg/l) as well as the required 90 percent removal for the data period reviewed

TSS. The effluent TSS effluent concentrations were also well below the permit concentration limits (average monthly concentration of 20 mg/l) as well as the required 90 percent removal for the data period reviewed.

Ammonia. The effluent ammonia concentrations were also well below the permit concentration limits (varies by month between 2.5 mg/l to 6.7 mg/l during the June 1st to October 31st ammonia permit season) for the data period reviewed.

CHAPTER FOUR
COLLECTION SYSTEM EVALUATIONS

In order to provide reliable wastewater service for the next 20 years there are collection system issues in Sewer District 1 that should be considered. These include infiltration and inflow (I/I), and sewer capacity issues (bottlenecks). Technical Memorandum No. 1, 2, and 3 contained in Appendix F, G, and H provide details of the evaluations conducted. A summary of these evaluations is provided below.

INFILTRATION AND INFLOW

Infiltration and inflow contribute additional flow to the Town's wastewater systems which needs to be conveyed in the collection systems and ultimately treated. I/I consumes valuable capacity in both the collection system and the WWTFs which can result in the need to provide additional or larger pipes, equipment, tankage, and treatment unit processes. This is an important component of total wastewater flow at the South Street WWTF that impacts the current and projected future flows.

Collection system I/I is an ongoing issue that needs to be monitored and addressed on a regular basis as new I/I sources can often develop from deterioration of the system components to offset the flow decreases achieved from I/I rehabilitation efforts. The recent sewer rehabilitation project, completed in 2010, addressed known defects in the main line sewer in Sewer District 1, but comprehensive investigations to identify other I/I sources such as smoke testing, manhole inspections, and house to house inspections to locate sump pump connections, had not been undertaken recently. Smoke testing and manhole inspections were conducted in Sewer District 1 as part of the Phase 1 Wastewater Facilities Plan.

Background

In 2007, the Town completed an I/I analysis of the wastewater collection system (Sewer Districts 1 and 2). The purpose of the investigation was to estimate the amount of I/I entering the wastewater collection system and to develop a prioritized program of additional investigations to identify sources of I/I for subsequent rehabilitation.

As a result of the 2007 I/I analysis, a number of recommendations were made. These recommendations included:

- Continued yearly TV inspection of the sewers to prioritize the rehabilitation of I/I sources and defects
- Sewer District 1 Inflow Investigations including:
    - Smoke testing
    - Dye testing and dye water flooding (rainfall simulation) of suspect and indirect sources based on the findings of the smoke testing.
    - Manhole Inspections
    - House-to-house inspections
- Sewer rehabilitation of specific sewer infiltration sources or defects which included:
    - Repair of leaking joints
    - Repair of cracked and broken pipes
    - Reduction of root intrusion by chemical root treatment
    - Rehabilitation of 75 lateral service connections
    - Excavation and replacement or lining of 1,000 linear feet of sewer

Subsequent to the 2007 I/I analysis, a sewer rehabilitation contract was undertaken to address the identified pipeline defects. The project was completed in May 2010 by the National Water Main Cleaning Company.

The recommended inflow investigations from the previous I/I analysis for Sewer District 1 consisting of house-to-house inspections and dye water testing and dye water flooding are anticipated to be undertaken during the Phase 2 Facilities Plan. Smoke testing (all of Seer District 1) and manhole inspections (Subarea 1 in Sewer District 1 only) were conducted as part of Phase 1 of the Wastewater Facilities Plan.

Data Collection

Smoke Testing. Smoke testing is performed primarily to detect inflow sources such as downspouts, catchbasins, cellar drains and area drains by introducing smoke into sewer manholes and visually observing its discharge points. Technical Memorandum No. 1, Smoke Testing contained in Appendix F, provides the details of this evaluation. A summary of this evaluation is provided herein.

Smoke testing services were provided by Stacy DePasquale Engineering (SDE) under subcontract to AECOM. Smoke testing was performed throughout the six subareas of Sewer District 1 as shown on Figure 4-1. Approximately 96,000 linear feet of sewers were smoke tested between September and October, 2013. As smoke was introduced into the wastewater collection system, the surrounding area was inspected for locations emitting smoke, indicating an inflow source. The subcontractors smoke testing report which provides location sketches and photographs of inflow sources, is included in Appendix F as Attachment A.

The smoke testing program identified both positive and suspect inflow sources. A positive inflow source is identified through smoke testing by smoke emanating from that source. Suspect inflow sources are potential sources of inflow which based on observed characteristics did not smoke during smoke testing but may be expected to be connected to the sanitary sewer.

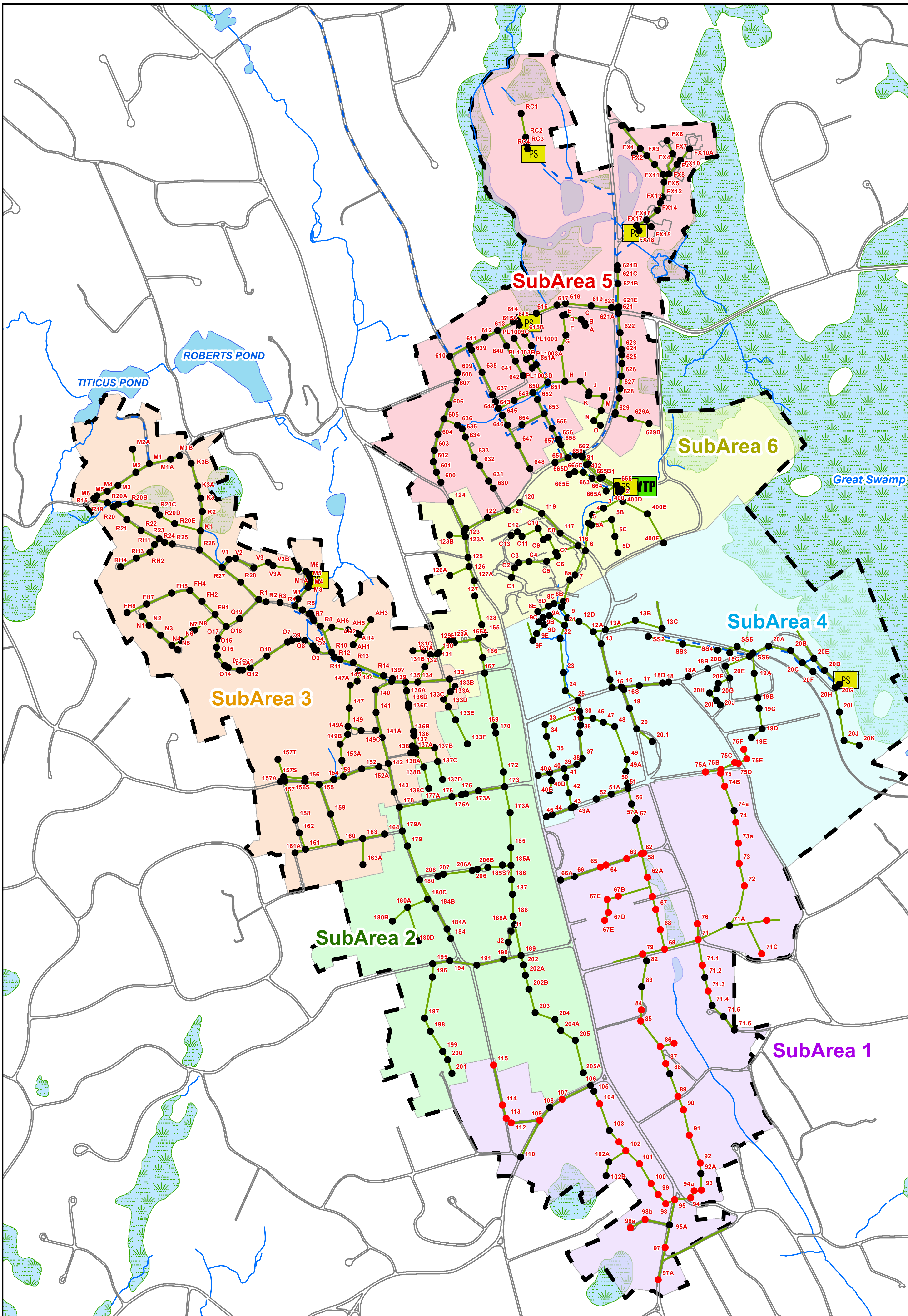
Manhole Inspections. Physical inspection of manholes is performed to identify manholes with active infiltration, inflow, signs of previous leakage, or physical defects. Technical Memorandum No. 3, Manhole Inspections, contained in Appendix H, provides the details of this evaluation that was conducted in Subarea 1 in Sewer District 1. A summary of this evaluation is provided herein.

Manhole inspections were performed by Stacy DePasquale Engineering (SDE) under subcontract to AECOM. 63 manholes were inspected in November of 2013 in Subarea 1 of Sewer District 1. Field technicians entered each manhole to conduct the inspection. For each sewer manhole inspected, a manhole inspection log presenting the data collected was completed by the field crew. The manhole inspection logs are included in the manhole inspection report included as Attachment A to Technical Memorandum No. 3 in Appendix H.

Analysis

Smoke Testing. Table 1 of Technical Memorandum No. 1 lists a total of 78 inflow sources (45 direct and 33 indirect), where smoke was observed during smoke testing operations. Direct inflow sources are those where significant smoke was observed during the smoke testing operations. Indirect inflow sources are those which smoked lightly during the smoke testing. Direct inflow sources identified include open service connection cleanouts, downspouts and catchbasins. Five sump pumps discharging to the sewer system have also been identified by these investigations. Indirect sources of inflow identified include catchbasins, a drain culvert, manholes that smoked in the area surrounding the corbel frame and cover, and areas where smoke emanated from soil seams. Further investigation including manhole inspections and dye water flooding in conjunction with television inspection of the adjacent sewer (dye water tracing) is warranted to identify the source of indirect sources identified.

Where possible, estimates of the peak inflow rate (gallons per day) entering the sewer system were calculated for each inflow source identified. Further investigation of some sources is needed to estimate the potential inflow. A total peak inflow rate of approximately 287,500 gallons per day (gpd) in a one year 6 hour frequency storm is estimated to be contributed by inflow sources identified by smoke testing.

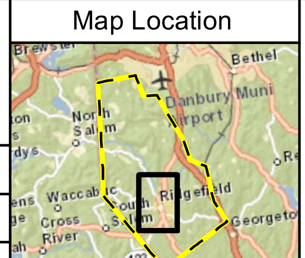


AECOM

Drawn: BC 4/24/2015

Approved: AA 4/24/2015

Map Location



Legend

- Manholes
- Manholes - Inspected
- PS Pump Station
- Sewers Smoke Tested
- Force Main
- Streams
- Road Edge

SubAreas

- Subarea 1
- Subarea 2
- Subarea 3
- Subarea 4
- Subarea 5
- Subarea 6
- Swamps
- Waterbodies

N

0 200 400 800 Feet

FIGURE 4-1

2013 SMOKE TESTING AND MANHOLE INSPECTIONS

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

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Table 2 of Technical Memorandum No. 1 lists those properties identified with suspect inflow sources. Suspect inflow sources include driveway or yard drains, downspouts piped underground, and buildings with flat roofs which may be connected to the sanitary sewer. A total of 784 suspect inflow sources were identified. Of the 784 suspect sources identified, 684 were located at the Casagmo, Quail Ridge, and Fox Hill condominium complexes. Another 13 suspect sources were identified at the Ridgefield Recreation Center. Generally suspect sources identified during smoke testing warrant follow-up investigation, including dye water testing for sewer connection verification.

Manhole Inspections. Based on the inspections conducted, the sewer manholes within Subarea 1 in Sewer District 1 were generally found to be in fair condition. Some manholes were observed to have sediment/debris build up, loose or misaligned frames and covers, non-concealed pick holes in the covers, evidence of leaking walls, deteriorated benches and inverts, and missing benches. 20 manholes were observed with evidence of leakage during the inspection, however, little quantifiable infiltration was observed. A total of 54 manholes of the 63 manholes inspected were identified as having at least one of the defects noted above.

Two other observations not recorded in the manhole inspection logs, but of significance, are the number of manholes in areas subject to flooding that are not equipped with watertight, locking manhole covers, and those that have raised chimneys that are not watertight due to deteriorated masonry.

Covers located in flood prone areas that are not watertight have the potential to be sources of inflow. It is recommended that these manhole covers be made watertight by either replacing them with watertight, locking manhole covers or installing manhole inserts to reduce the potential for inflow during periods of inundation.

Raised manholes located in flood prone areas that are not watertight have the potential to be sources of inflow. Based on conditions observed during AECOM's field visits it is recommended that these raised manhole chimneys be wrapped with a watertight material to reduce leakage during periods of inundation.

In addition, during the field investigations a number of manholes in Subarea 1 could not be inspected. These manholes were either buried, paved over, on private property or unable to be located. To fully complete the inspection of all of the manholes in Subarea 1 the Town should take the necessary steps to locate and inspect the manholes identified in Table 3 of Technical Memorandum No. 3.

Summary and Recommendations – Smoke Testing and Manhole Inspections

The smoke testing program located and documented a total of 78 inflow sources. These sources are estimated to contribute inflow to the Ridgefield Sewer District 1 collection system at a peak rate of 0.287 mgd from a one year 6 hour storm. Of the 78 inflow sources identified, 45 have been identified as direct and 33 have been identified as indirect inflow sources. Additionally, 784 suspect inflow sources were identified. Further investigation of 160 of these suspect sources is recommended to verify whether or not they are sources of inflow to the wastewater collection system. Finally, with the discovery of 5 sump pumps during smoke testing, a house-to-house internal building inspection program is recommended to identify the presence of sump pumps connected to the wastewater collection system.

Through the manhole inspections a total of 54 manholes which were identified as requiring repair of defects and/or cleaning to remove sediment and debris accumulated on the bench or in the invert of the manhole. To repair the defects, it is recommended that the Town incorporate the design and construction of manhole rehabilitation measures into a manhole rehabilitation project. Furthermore, as summarized in Technical Memorandum No. 2, it is recommended that the remaining manholes in Sewer District 1, approximately 550, be inspected to identify additional sources of leakage and to assess the general condition of manholes in Sewer District 1.

Based on the smoke testing and manhole inspections performed, it is recommended that the Town implement a program to eliminate the inflow sources identified. The program would consist of three components:

- Capping and redirection of direct inflow sources
- Manhole rehabilitation
- Further investigations of indirect and suspect sources identified during smoke testing and locate additional direct inflow sources such as sump pumps.

Each of the recommended rehabilitation components are summarized below.

Capping and Redirection of Direct Inflow Sources. 45 direct inflow sources have been identified and are recommended for repair. Recommended repairs consist of capping open or broken cleanouts and the redirection of downspouts and sump pumps. Given the varied nature of sump pump configurations, the estimated cost of disconnecting and rerouting sump pumps is based on a licensed plumber disconnecting the sump pump from the sewer system and hard piping the discharge to the closest location outside of the building. Similarly, the estimated cost of disconnection and rerouting of downspouts is based on a licensed plumber disconnecting the downspouts, capping the sewer connection, and installing a bend and splash pad.

Table 4-1 presents a summary of the recommended capping and redirection of these direct inflow sources along with updated estimated costs, including an allowance for engineering and contingencies. The total estimated cost of the capping and redirection of direct inflow sources is approximately $40,000.

TABLE 4-1. SUMMARY OF RECOMMENDED CAPPING AND REDIRECTION OF DIRECT INFLOW SOURCES

Component	Quantity	Estimated Cost
Cap and Seal Cleanout	34	$25,000
Disconnect and Reroute Downspouts	4	$3,000
Disconnect and Reroute Sump Pumps	5	$12,000
Disconnect and Reroute Catchbasins	2	*(1)*
Total Estimated Cost		$40,000

Notes: 1. Catchbasins have reportedly been disconnected.

Manhole Rehabilitation. 13 manholes were identified as sources of inflow during smoke testing. To determine the full extent of repairs necessary, inspection of these structures is warranted and is discussed in the section of this report entitled Further Investigations, below. Minimum recommendations for repair of the identified inflow sources have been made, where possible. Additionally, manhole inspections identified 54 manholes as having at least one defect requiring repair.

Table 4-2 presents a summary of the recommended manhole repairs along with updated estimated costs, including an allowance for engineering and contingencies. The manhole repairs generally include resetting or replacing frames and covers, chemical sealing and/or interior coating of walls, and repairs to the chimney, bench and invert areas. The total estimated cost of the manhole repairs is approximately $201,000.

TABLE 4-2. SUMMARY OF RECOMMENDED MANHOLE REPAIRS

Recommended Repair	Quantity	Estimated Cost
Manhole Cleaning	12	$9,000
Repair Defective Chimney	8	$8,000
Wrap Chimney	6	$18,000
Reset Frame and Cover	16	$17,000
Raise Frame and Cover	10	$11,000
Replace Defective Frame and Cover	10	$16,000

TABLE 4-2. SUMMARY OF RECOMMENDED MANHOLE REPAIRS (CONT.)

Recommended Repair	Quantity	Estimated Cost
Install Manhole Insert	38	$19,000
Root Control	15	$20,000
Chemical Sealing	18	$28,000
Chemical Sealing & Coating	13	$41,000
Rebuild Bench & Invert	4	$6,000
Chemical Sealing Connection	8	$8,000
Total Estimated Cost		$201,000

Further Investigations. 13 manholes were identified as inflow sources. These manholes have been recommended to be inspected to assess their condition and identify rehabilitation measures as necessary. 5 of these manholes have already been inspected. Technical Memorandum No. 3 details the inspections. Technical Memorandum No. 2 recommended that the remaining manholes in Sewer District 1, approximately 550, also be inspected.

20 inflow sources were also identified which require further investigation to verify the presence of the connection and to quantify the inflow of the sources identified. These sources include 9 catchbasins, 1 drain culvert (Technical Memorandum No. 1, Table 1, indirect source number 68) and 10 locations where smoke was observed emanating from seams in the soil. It is recommended that these sources be dye water flooded in conjunction with television inspection of the adjacent sewer (dye water tracing).

784 suspect inflow sources were identified during the smoke testing. Of the 784 suspect sources identified, 160 suspect sources are recommended to be dye water tested.

Finally, the discovery of 5 sump pumps during the smoke testing provides evidence of the presence of sump pumps in Sewer District 1. It is therefore recommended that the Town undertake a two phase program to identify and remove sump pumps from the wastewater collection system in District 1. In the first phase, the Town should undertake the necessary investigations to locate sump pumps. This will involve conducting a house-to-house internal building inspection program to identify sump pumps connected to the sewer collection system. The second phase will involve development and implementation of a program to redirect sump pump discharges out of the sanitary sewer system.

Table 4-3 presents a summary of the recommended further investigations along with estimated costs, including an allowance for engineering and contingencies. The total estimated cost of the recommended further investigations is approximately $332,000.

TABLE 4-3. SUMMARY OF RECOMMENDED FURTHER INVESTIGATIONS

Component	Quantity	Estimated Cost
Inspect Manholes	556	$93,000
Inspect Structures	2	$2,000
Dye Water Tracing of Indirect Sources	20	$23,000
Dye Water Testing of Suspect Sources	160	$30,000
House to House Inspections	1,760	$184,000
Total Estimated Cost		$332,000

COLLECTION SYSTEM BOTTLENECK EVALUATION

An evaluation was conducted to identify the hydraulic limitations in the Sewer District 1 collection system which contribute to surcharging of sewers during wet weather, high flow conditions. Technical

Memorandum No. 2, Collection System Bottleneck Evaluation contained in Appendix G, provides the details of this evaluation. A summary of this evaluation is provided below.

Background

There are a number of areas in the collection system that are reported to have capacity issues as certain sewers surcharge or backup under wet weather conditions. Surcharges are typically indicative of a sewer that is undersized (or overloaded) or that has a defect or blockage that is not allowing it to convey wastewater as intended. These areas are "bottlenecks" in the collection systems which have the potential to cause sewage backups into homes or businesses and sewer system overflows (SSOs). Grove Street in Subarea 4 of Sewer District 1 is an area that is reported to have surcharging issues. Other areas of surcharging, identified through previous investigations, include:

- Governor Street
- New Street
- Arnolds Way Easement
- Olcott Way at the Casagmo Condominiums

Data Collection

To estimate the hydraulic capacities of the collection system under investigation a field survey of the manholes on the sewers in the areas under investigation was conducted to collect information on sewer sizes, and gather rim and invert elevations. AECOM obtained the services of Land Resource Consultants, Inc., of Cromwell, CT to conduct the field survey. Existing television inspection data provided by United Water was reviewed to identify the condition of the sewer reaches under investigation.

Continuous flow monitoring at nine locations on the sewers under investigation was conducted from April 17, 2013 through July 10, 2013. Rainfall gauging at the South Street WWTF was also conducted during this period. AECOM also obtained the services of ADS Environmental Services, of Congers, NY, to conduct continuous flow monitoring. The flow monitoring report is included as Attachment A to Technical Memorandum No. 2. The flow data was collected to identify areas of surcharging and compare against the theoretical capacity of the collection system under investigation.

The extent of the field survey as well as the meter and rain gauge locations are shown on Figure 4-2. For ease of reference, the collection system under analysis is referred to as the East Branch and the West Branch.

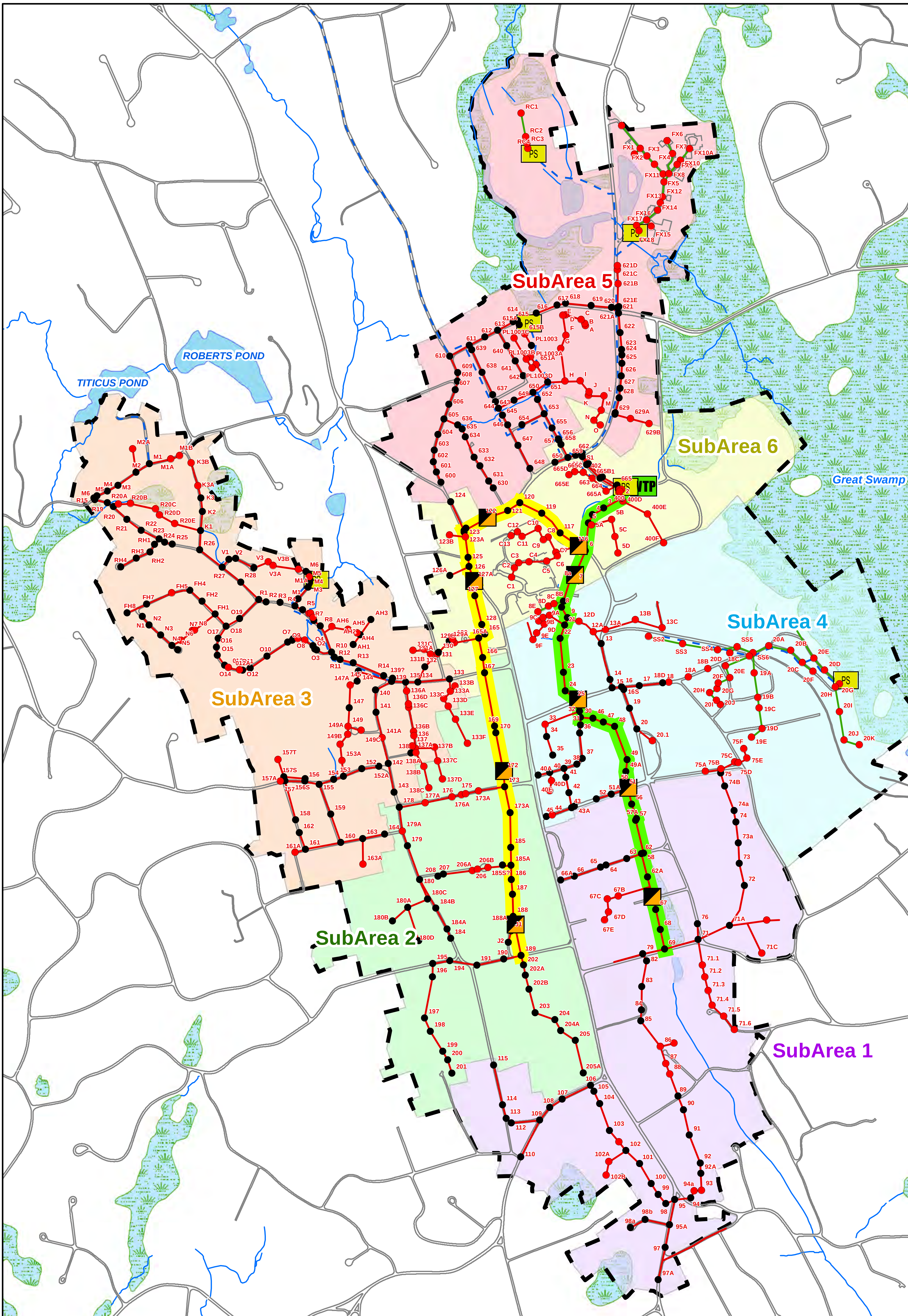
Analysis

The collected data was analyzed to identify potential bottlenecks in the collection system. Below is a brief summary of the analysis.

Review of Television Inspection Data. Existing television inspection data was reviewed to identify the condition of the pipe, pipe materials, and to estimate the roughness coefficient (Manning's "n") for use in the hydraulic analysis.

Much of the collection system under review is old. Many of the original vitrified clay piping sections have been lined or have undergone some type of rehabilitation to reduce I/I or prolong their useful life. However, the system continues to exhibit signs that extraneous flows enter the collection system. This may be from inflow sources such as sump pumps, downspouts, or open abandoned service connections or cleanouts. Typical of many old clay sewers is that they have bends and sags which can inhibit the theoretical capacity of the pipeline.

Table 4-4 is a summary of the pipe materials and corresponding roughness coefficients (Manning's "n") used in the analysis.



Drawn:	BC 4/13/2015
Approved:	AA 4/13/2015

Map Location

Legend

- Manhole
- Manhole - Not Field Located
- PS Pump Station
- 2013 Meter Locations
- Updated Sewer (From Smoke Testing)
- Municipal Sewer
- Private Sewer
- Force Main
- Streams
- West Branch Sewer Manholes Surveyed
- East Branch Sewer Manholes Surveyed

N

0 200 400 800 Feet

FIGURE 4-2

SURVEYED MANHOLES AND METER LOCATIONS

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 4-2 - Surveyed Manholes and Meter Locations.mxd

TABLE 4-4. SUMMARY OF PIPE MATERIALS AND ROUGHNESS COEFFICIENTS

Pipe Material	Roughness Coefficient (Mannings's "n")
Liner (LNR)	0.0120
Cast Iron Pipe (CIP)	0.0140
Polyvinylchloride (PVC)	0.0130
Vitrified Clay Pipe (VCP)	0.0150

Hydraulic Capacities. A hydraulic capacity spreadsheet model of the collection system under investigation was prepared using the survey data collected and the roughness coefficients estimated to calculate the theoretical capacities of the sewers. Tables 1 and 2 of Technical Memorandum No. 2 summarize the hydraulic characteristics (i.e. pipe diameters, slopes, etc.) of each manhole to manhole reach of the sewers under investigation. The tables also report the theoretical full flow conveyance capacity of these sewers.

Sewer design criteria typically require that a sewer be constructed with a slope equal to or greater than the minimum slope required to provide a full flow velocity of 2.0 feet per second to keep solids in suspension. Sewers constructed with less than minimum slope may result in problems with sediment deposition and back-ups due to poor flow velocities in the sewer. These sewers can also act as "bottlenecks", restricting the flow.

The majority of the wastewater collection system under investigation appears to have been constructed with slopes greater than or equal to the minimum slope recommended. However, two sewer reaches have been identified as having less than minimum slopes. They are sewer reach MH 57A to MH 56, between Market Street and Governor Street, and sewer reach MH 67 to MH 67A between Rowland Lane and Branchville Road. The only recourse to those lines identified as having a less than minimum slope is to either clean the line regularly to remove any sediment build-up or replace the line if there is the ability to increase the slope. The invert elevations of the upstream and downstream manholes on those lines are fixed. It is therefore recommended that the Town regularly monitor these two sections for sediment build-up and clean as necessary.

Continuous Flow Monitoring. Continuous monitoring of wastewater flows was performed from April 17, 2013 through July 10, 2013 to help identify bottlenecks in the existing collection system. Rainfall data was also collected for this period. Both the rainfall and flow data were collected and reported in 15-minute increments. Meters were installed on the East Branch and the West Branch in an effort to capture data upstream and downstream of suspected problem areas.

During the monitoring period a total of 13.56 inches of rainfall were recorded. Recorded rain events are summarized in Table 4-5.

TABLE 4-5. SUMMARY OF RAINFALL EVENTS

Rainfall Event Date and Time		Total Rainfall (inches)	Duration (hours)	Average Rainfall Intensity (in/hr)	Peak Rainfall Intensity (in/hr)
Start	End				
4/19/2013 23:45	4/20/2013 7:30	0.32	8.00	0.04	0.11
5/8/2013 8:15	5/8/2013 16:15	0.51	8.25	0.06	0.27
5/9/2013 12:45	5/9/2013 15:30	0.16	3.00	0.05	0.10
5/11/2013 2:00	5/11/2013 5:30	0.14	3.75	0.04	0.12
5/11/2013 16:30	5/11/2013 17:15	0.11	1.00	0.11	0.11
5/19/2013 5:45	5/19/2013 23:00	0.33	17.50	0.02	0.07
5/23/2013 10:15	5/24/2013 9:30	1.00	23.50	0.04	0.20
5/24/2013 16:30	5/25/2013 22:30	1.12	30.25	0.04	0.21

TABLE 4-5. SUMMARY OF RAINFALL EVENTS (CONT.)

Rainfall Event Date and Time		Total Rainfall (inches)	Duration (hours)	Average Rainfall Intensity (in/hr)	Peak Rainfall Intensity (in/hr)
Start	End				
5/28/2013 14:30	5/29/2013 3:45	0.29	13.50	0.02	0.10
6/2/2013 20:45	6/3/2013 9:00	0.79	12.50	0.06	0.32
6/6/2013 18:15	6/8/2013 4:15	3.14	34.25	0.09	0.31
6/10/2013 13:45	6/11/2013 4:15	1.34	14.75	0.09	0.22
6/11/2013 15:15	6/11/2013 17:30	0.19	2.50	0.08	0.14
6/13/2013 8:30	6/14/2013 6:00	2.12	21.75	0.10	0.35
6/17/2013 13:30	6/17/2013 14:00	0.10	0.75	0.13	0.10
6/18/2013 16:45	6/18/2013 19:00	0.18	2.50	0.07	0.09
6/24/2013 20:00	6/24/2013 20:45	0.20	1.00	0.20	0.20
6/26/2013 19:30	6/26/2013 20:15	0.08	1.00	0.08	0.08
6/27/2013 22:30	6/28/2013 1:00	0.61	2.75	0.22	0.56
7/1/2013 10:45	7/1/2013 20:00	0.44	9.50	0.05	0.22
7/10/2013 13:00	7/10/2013 15:00	0.10	2.25	0.04	0.05

Notes:

1. *Highlighted rows indicate dates of sewer surcharges measured by the flow meters.*

Based on a review of the data collected on flow depth, four rainfall events triggered surcharging at various meter locations. A sewer is considered surcharged when the depth of flow in the pipe rises above the crown of the pipe at a manhole. Surcharge events recorded during the flow metering period are summarized in Table 4-6.

TABLE 4-6. SUMMARY OF METERING DATA SURCHARGE EVENTS

Meter Location Manhole Number	Pipe Diameter (in)	*(1)* Average Dry Weather Flow (mgd)	Maximum Flow (mgd)	*(2)* Maximum Flow Depth (in)	*(3)* Maximum Surcharge Depth (in)	Surcharge Dates
MH 67A	10	0.10	1.44	50	40	5/23/13, 6/7-8/13, 7/1/13
MH 51	12	0.14	1.36	30	18	5/23/13, 6/8/13, 7/1/13
MH 25 *(4)*	11	0.13	1.71	11	N/A	N/A
MH 07	12	0.25	2.19	8	N/A	N/A
MH 188A	12	0.02	0.74	4	N/A	N/A
MH 172 *(4)*	11	0.09	1.73	7	N/A	N/A
MH 127A *(4)*	11	0.20	2.45	16	5	5/23/13, 6/7-8/13, 6/14/13, 7/1/13
MH 122	12	0.25	2.01	6	N/A	N/A
MH 116	12	0.31	2.89	89	77	5/23/13, 6/7-8/13, 7/1/13

Notes:

1. *Dry weather flows recorded April 25, 2013 through May 7, 2013.*
2. *Maximum flow depth is the depth of flow above the invert of the pipe.*
3. *Maximum surcharge depth is the depth of flow above the crown of the pipe.*
4. *Effective diameter of pipe reduced by liner.*

Figures 4-3 and 4-4 compare the estimated dry and peak wet weather flows to the theoretical capacities for each reach of the collection system under evaluation.

During dry weather conditions the flows recorded at each of the metered locations on both the East Branch and the West Branch collection systems are well within the theoretical capacity of the existing sewers. This indicates that the collection system has adequate capacity to convey the collected wastewater in dry weather. However, estimated peak flows during wet weather conditions reach or exceed the capacity of the collection system at a number of locations. These portions of the collection system may be referred to as collection system bottlenecks. Figure 4-5 highlights these collection system bottlenecks.

Collection System Bottleneck Relief Options

Options available to relieve the identified bottlenecks generally include identifying and reducing Infiltration/Inflow (I/I), diverting flow around the bottlenecks, conducting collection system upgrades by constructing new sewers, and performing routine operation and maintenance (O&M).

Identify and Reduce I/I. Through previous investigations and recent field work, the Town has identified numerous public and private sources of inflow to its wastewater collection system such as catch basins, a drainage culvert, defective manholes, sump pumps, and roof leaders. Inflow entering the system through these sources is a contributing factor to the flow related problems that occur within the collection system under investigation during wet weather conditions.

One way to reduce I/I is to proceed with recommendations arising from the 2013 smoke testing and manhole inspection efforts. These efforts are summarized above and are detailed in Technical Memorandum No. 1. Rehabilitation of the inflow sources identified and quantified during smoke testing have the potential of removing an estimated peak inflow rate of 0.29 mgd in a one year, 6 hour storm from the collection system.

It is also recommended that further efforts be made to identify and eliminate sources of inflow. These efforts should consist of:

- Manhole inspections as recommended in the 2007 Infiltration/Inflow Analysis Report and summarized in Technical Memorandum No. 1.
- Conducting house-to-house inspections as recommended in the 2007 Infiltration/Inflow Analysis Report and summarized in Technical Memorandum No. 1.
- Conducting public education and outreach as summarized in Technical Memorandum No. 1. The Town should sponsor public education activities, including the preparation of a brochure to mail to residents, posting the brochure on the town's web site, bill stuffing and newspaper articles to inform the public. Costs associated with this effort should be included in the WPCA's operating budget.

Divert Flows Around Bottlenecks. Another relief option is to divert flows around the bottlenecks identified. As detailed in Technical Memorandum No. 4 and summarized in Chapter 5, it has been recommended that the Quail Ridge Pump Station be relocated from its current location to the intersection of South Street and Old Quarry Road. Figure 4-6 illustrates the location of the new pump station site and the conceptual alignment of the gravity sewer that would be required to convey flows from the current pump station location to that of the new location. The pump station currently discharges to the gravity sewer on Sunset Lane, upstream of a portion of the East Branch of the collection system which has been identified as a bottleneck. The relocation of the pump station would divert flows around the area identified as a bottleneck and discharge flows either directly to the South Street WWTF or to a gravity sewer upstream of the South Street WWTF.

The relocation of the Quail Ridge Pump Station alone will not alleviate the bottlenecks however it would reduce flows upstream of a section of the wastewater collection system that has been identified as a bottleneck.

Figure 4-3. Theoretical Capacity vs. Estimated Flows - East Branch

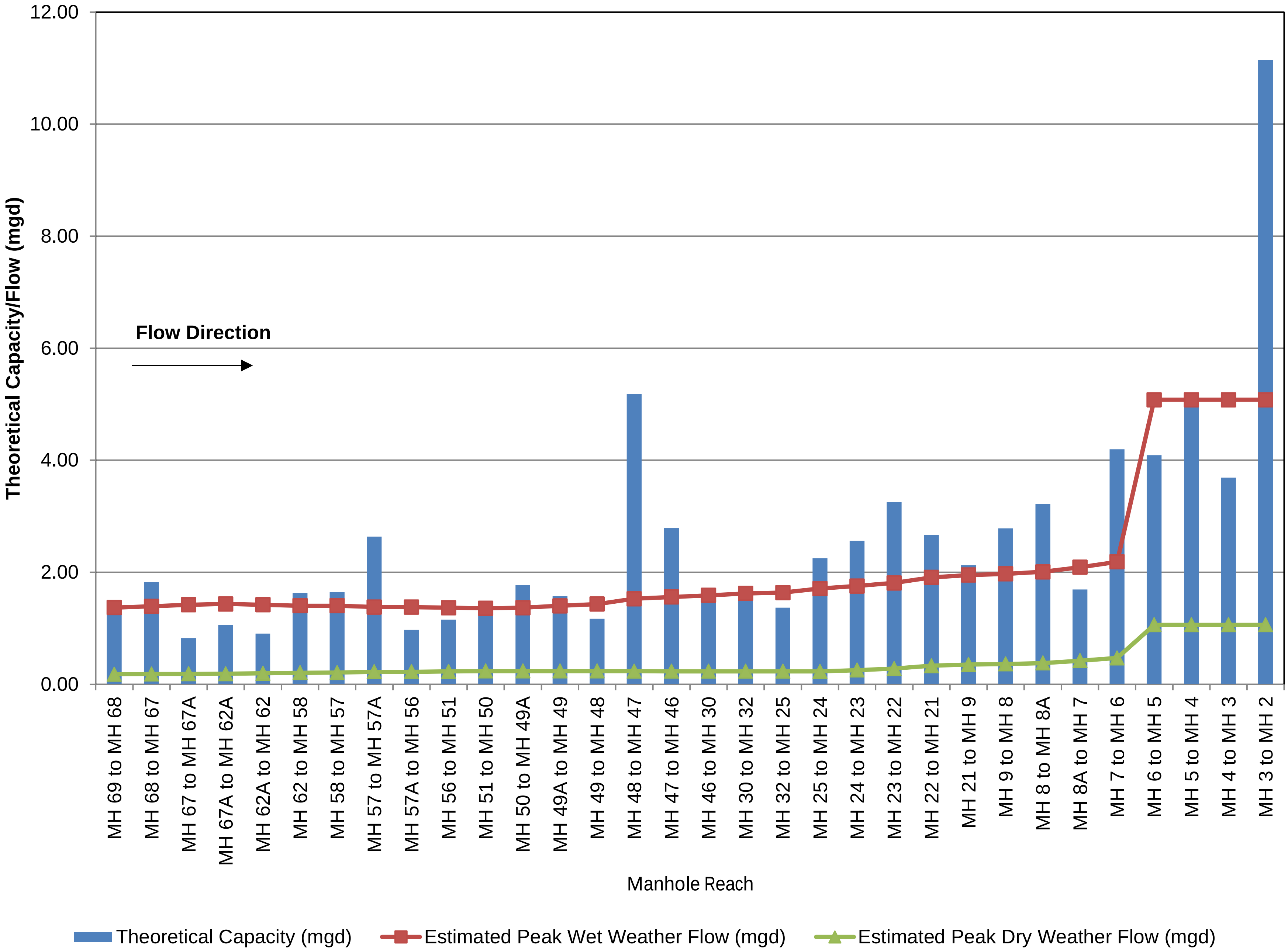
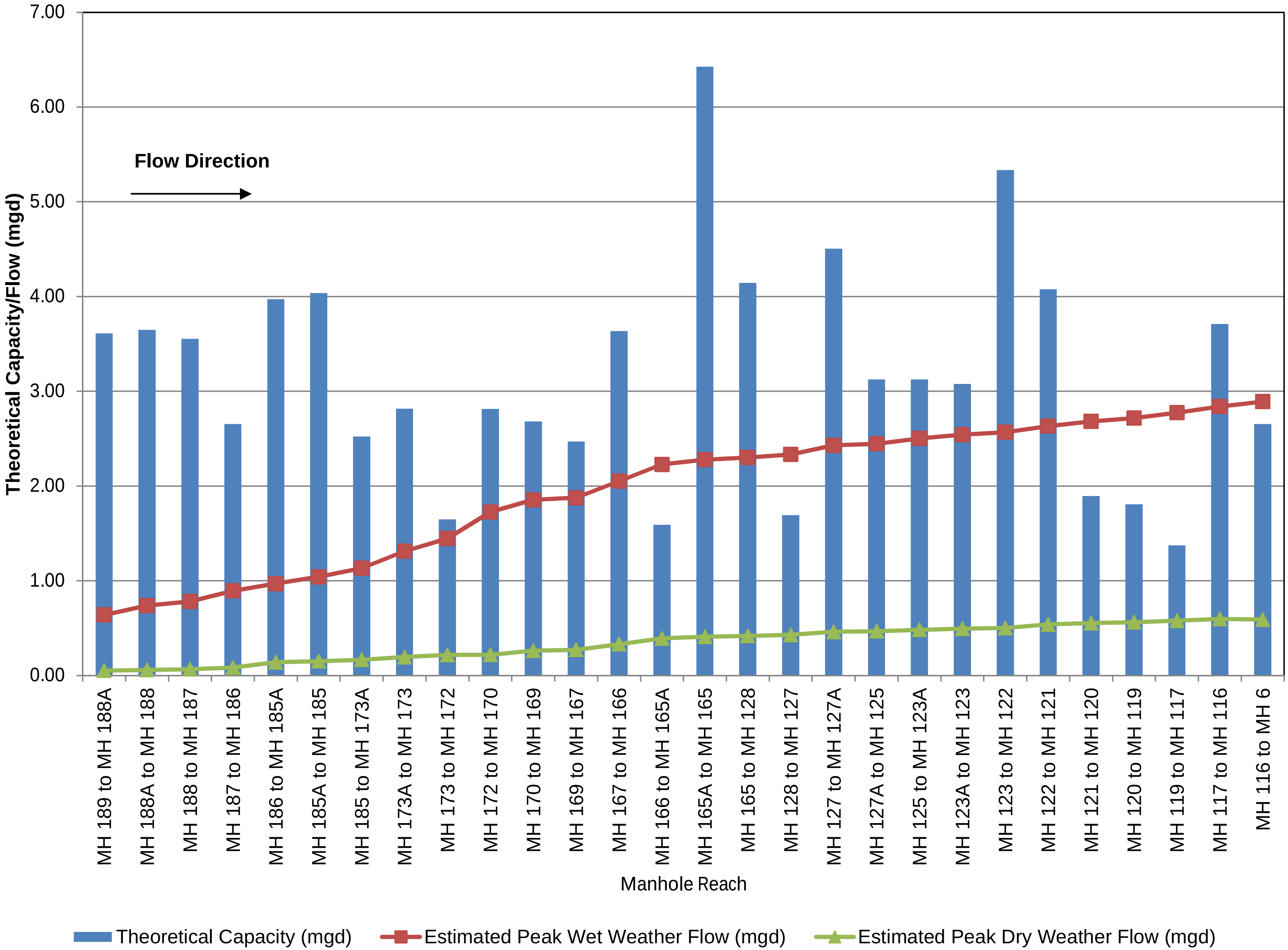
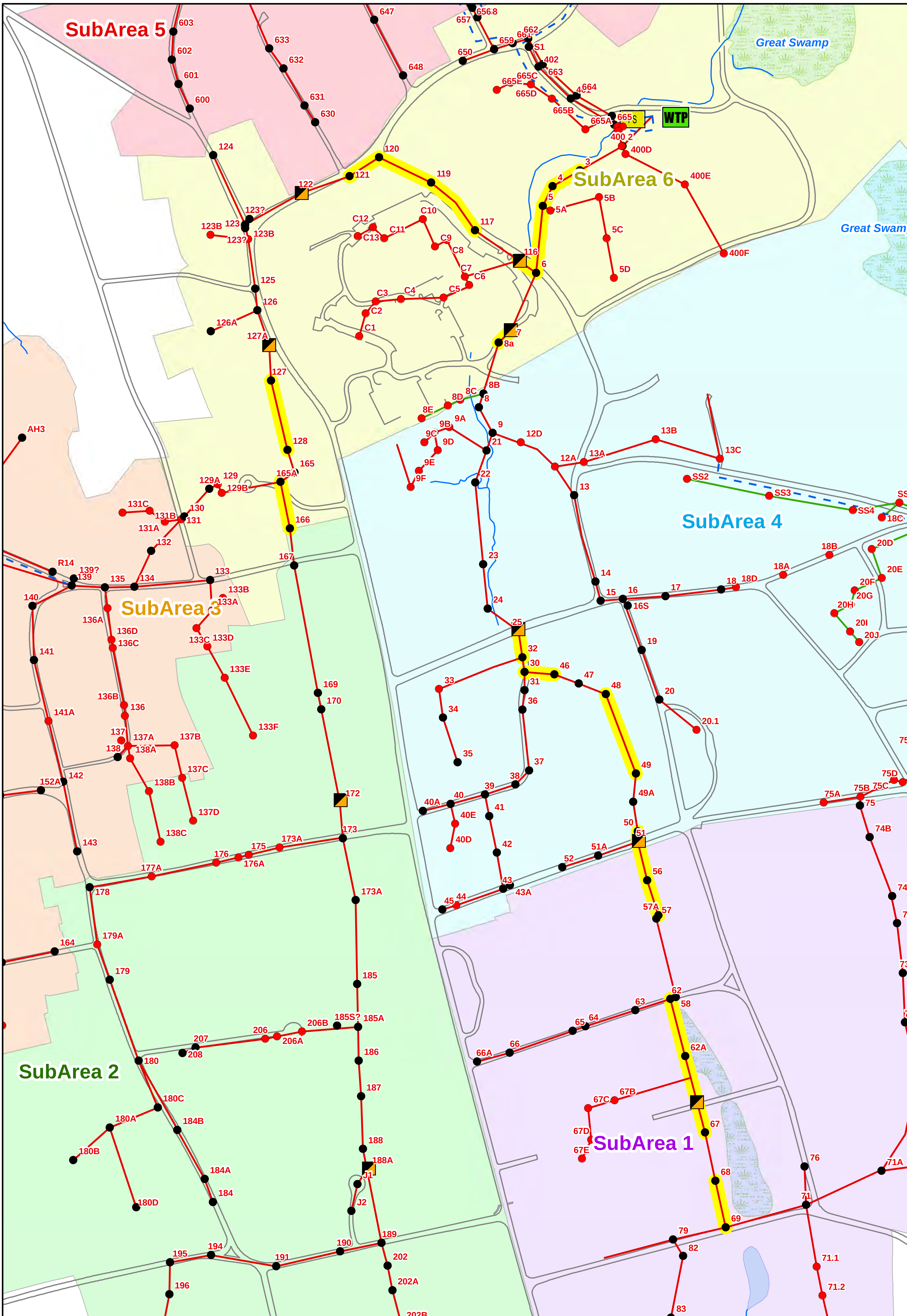


Figure 4-4. Theoretical Capacity vs. Estimated Flows - West Branch



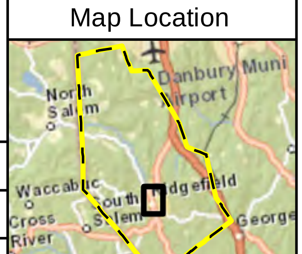


Drawn: BC 4/13/2015

Approved: AA 4/13/2015

Project #: 60284509

Map Location



Legend

- Manhole
- Manhole - Not Field Located
- Force Main
- Updated Sewer (from Smoke Testing)
- Municipal Sewer
- Private Sewer
- Estimated Flows Exceed Theoretical Capacity
- Road Edge
- Streams

SubAreas

- Subarea 1
- Subarea 2
- Subarea 3
- Subarea 4
- Subarea 5
- Subarea 6
- 2013 Meter Locations
- PS Pump Station

N

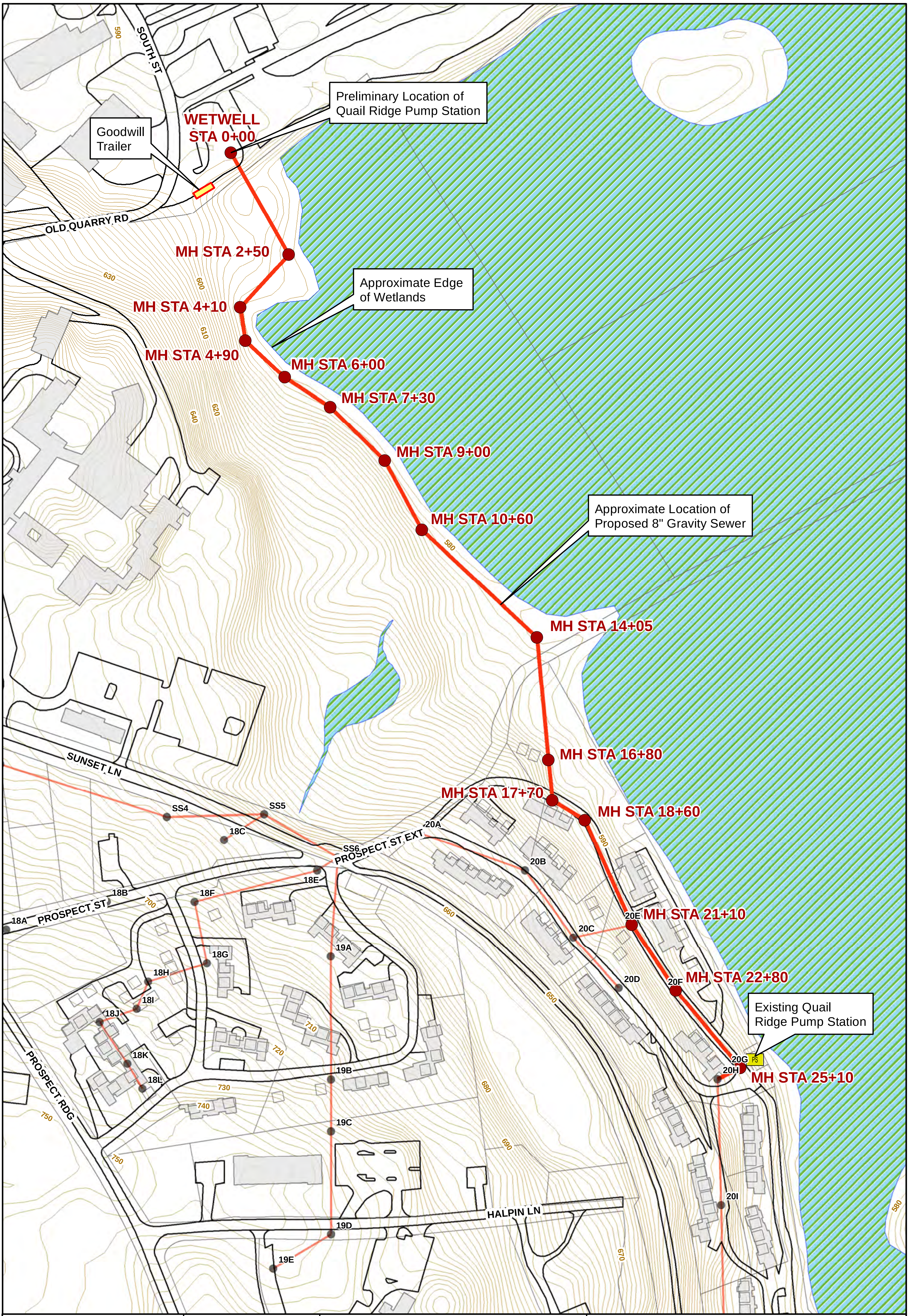
0 200 400 800 Feet

FIGURE 4-5

COLLECTION SYSTEM BOTTLENECKS

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 4-5 - Collection System Bottlenecks.mxd



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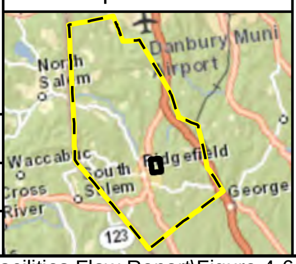
Approved:	AA	4/13/2015

Map Location



Legend

- PS Existing Pump Station
- Proposed Manholes
- Proposed Sewer
- Manhole - Existing
- Existing Sewer
- Swamps
- Edge of Pavement
- Buildings
- Contour

N

0 100 200 400 Feet

FIGURE 4-6

QUAIL RIDGE GRAVITY SEWER ALIGNMENT

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 4-6 - Quail Ridge Pump Station Relocation.mxd

Collection System Upgrades. A third relief option would be to accommodate the estimated peak wet weather flow. Under this option, upgrades to the collection system would be necessary. Upgrades would consist of replacing portions of sewers in both the East Branch and West Branch collection systems. Pipelines with insufficient capacity to accommodate estimated peak wet weather flows would be increased in size to handle the additional flow. As an alternative to the replacement of sewers with insufficient capacity to accommodate estimated peak wet weather flows, the construction of parallel relief sewers could be considered during design if sufficient space and elevation is available.

Upgrades to the East Branch of the collection system under investigation include replacing the entire length of the East Branch consisting of approximately 5,800 linear feet of existing sewers. Approximately 3,100 linear feet of existing sewers on the West Branch of the collection system, approximately 48% of the total length, would be replaced. The majority of the East Branch and some of the West Branch of the collection system upgrades are located in off road areas, behind residential and through commercial properties. Replacement of these sewers would have impacts to residents, businesses, and traffic as well as adjacent wetlands. The total estimated cost associated with the collection system upgrades, including allowances for engineering and contingencies, is approximately $5,300,000 ($3,350,000 for East Branch upgrades and $1,950,000 for West Branch upgrades).

Because of the significant impacts that construction would have and the high capital costs, the upgrades presented above are not recommended at this time. It is recommended that the Town first continue to identify and eliminate sources of I/I as previously described. Following the efforts to identify and eliminate sources of I/I, then upgrades to specific sections of the East and West Branches that continue to exhibit signs of bottlenecking should be revisited.

Perform Routine Operation and Maintenance (O&M). Based on the results of the capacity analysis presented, the sewers between MH 57A to MH 56 and MH 67 to MH 67A that have been constructed at slopes less than the minimum typically used in the design of sanitary sewers. These sewers should be given priority when conducting routine O&M procedures. Because these sewers are more likely to experience flow related problems, the Town should implement a program of cleaning the sewers and inspecting the manholes for evidence of surcharging on a regular basis.

Summary and Recommendations – Collection System Bottleneck Evaluation

The collection system has sufficient capacity to accommodate dry weather flows, but during wet weather conditions infiltration and inflow entering the system consume much of the system's capacity. Given the magnitude of inflow in the system, known inflow sources should be removed, and additional investigation efforts should be made to further identify and subsequently remove sources of inflow. Generally, inflow sources are the most cost effective flows to remove. It is therefore recommended that the elimination of inflow sources be pursued prior to implementing collection system upgrades to accommodate these flows

To alleviate bottlenecks identified in the collection system it is recommended that the Town take the following measures:

- Reduce I/I
    - Rehabilitate and further investigate inflow sources identified during 2013 Smoke Testing
    - Conduct additional Manhole Inspections as recommended in 2007 I/I Report
    - Conduct House-to-House Inspections as recommended in 2007 I/I Report
    - Conduct Public Education and Outreach
- Divert Flows Around Bottlenecks
    - Relocate Quail Ridge Pump Station
- Perform Routine O&M
    - Include sewer reaches MH 57A to MH 56 and MH 67 to MH 67A on a list of sewers that should be cleaned and inspected on a regular basis

Implementation of the recommendations contained herein may not eliminate all of the bottlenecks identified in the collection system. However, it presents an approach to reduce peak flow rates which contribute to the bottlenecking by eliminating known inflow sources and diverting flows around bottlenecks. It also presents a methodical approach to identifying additional sources of inflow and identifies areas which should be monitored for routine maintenance.

Following the recommended investigations (rainfall simulation, manhole inspections, and house-to-house inspections) rehabilitation recommendations will be made to further reduce extraneous flows entering the collection system. Implementation of the rehabilitation recommendations resulting from these investigations is not included in the estimated costs as the extent of rehabilitation work is not known at this time. After implementation of the rehabilitation recommendations are performed the Town should evaluate if collection system upgrades are required in addition to the elimination of inflow and diversion of flows around bottlenecks.

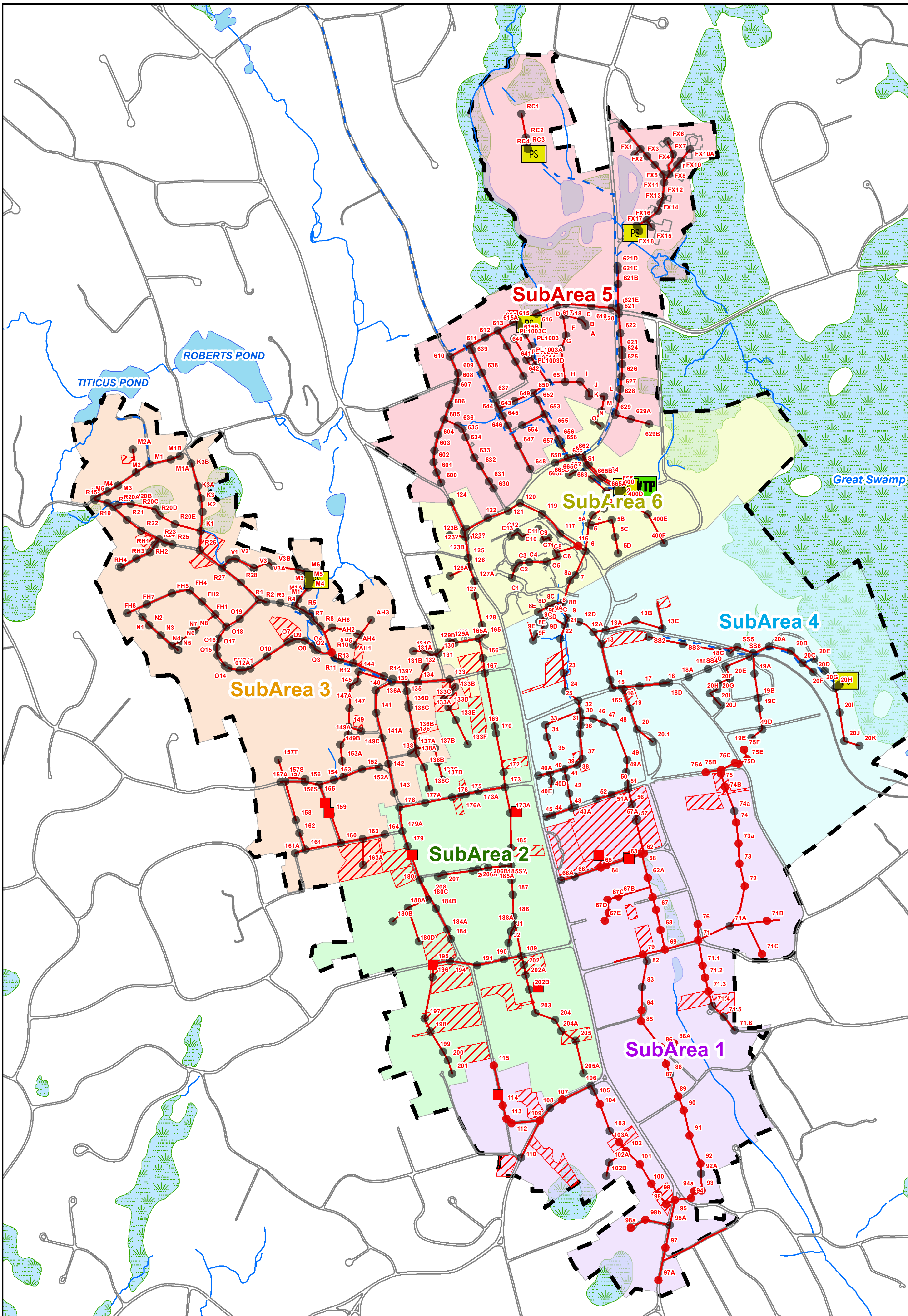
As noted in Chapter 5 the estimated costs to divert flows around bottlenecks by relocating the Quail Ridge Pump Station is $2,200,000. The costs presented are planning level cost estimates for budgeting purposes. A more accurate estimate of the anticipated costs may be determined during subsequent phases of the recommended program.

RECOMMENDED APPROACH TO IDENTIFY AND/OR ADDRESS ISSUES

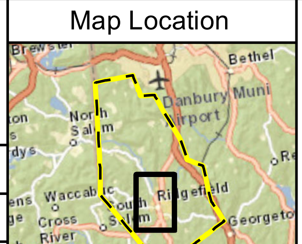
Figure 4-7 indicates the location of inflow sources and defects identified during the smoke testing and manhole inspection efforts conducted in 2013, as summarized in Technical Memorandum No. 1 and 3, included in Appendix F and H, respectively. A review of this information shows that the majority of the defects identified during the 2013 smoke testing and manhole inspection efforts are located in older parts of the wastewater collection system (only manholes in Subarea 1 were inspected). Approximately 76% of the inflow sources identified through smoke testing are located on private property. With the large percentage of inflow sources identified as being located on private property, it is recommended that one of the first steps that the Town undertake to address the identified inflow sources is to develop a policy and procedure regarding correction of privately owned inflow sources.

A variety of approaches have been used by other municipalities to address private inflow sources. A decision will need to be made on whether these sources will be addressed by the Town, or whether the Town will require the owners of the property where the privately owned sources were identified to implement removal of the sources. For example, sump pumps discharging to the sanitary sewer are illegal under Ridgefield's Sewer Use Regulations. Some communities have taken the approach that property owners with identified sump pump connections are in violation of the Sewer Use regulations, and the connection must be permanently removed from sewer system at the homeowner's expense. The Town typically requires a post removal inspection to confirm that the sump pump discharge has been redirected with rigid piping, and not flexible hose that can be easily redirected back to the sewer. At the other end of the spectrum, other communities have taken the approach where the Town undertakes a project to redirect the sump pump discharges as a Town administered and funded project. This approach involves the development of a Town funded construction project where a contractor completes the sump pump discharge redirection in the individual buildings with sump pumps. Agreements for access between the Town and the homeowner are necessary to allow this work to be completed.

Based on AECOM's experience, the most successful sump pump redirection programs have involved the use of some form of an Amnesty Program. Using this approach, the Town undertakes a public relations/education program through the local paper, mailings, and the Town website regarding the sump pump and other private inflow source problems and its effects on the efficiency of the treatment plant and the operating cost to pump and treat clean water. As part of the public relations/education program, it is noted that any homeowner that has a sump pump connected to the system has a certain period of time to notify the Town of the connection, and to have the pump discharge permanently redirected to an alternate location, typically to a dry well, a storm drain, or an adjacent low lying area on their property. Once the work is completed, the homeowner notifies the Town, and an inspection of the redirection is completed by Town staff. The most successful of these programs involve some payment to the homeowner for completing the redirection. Some communities have elected to set a fixed amount for homeowner



Map Location



Drawn: BC 4/24/2015

Approved: AA 4/24/2015

Legend

- Catch Basins with Sewer Defects Identified During Smoke Testing
- Parcels with Sewer Defects Identified During Smoke Testing
- Existing Sewer
- PS Pump Station
- Force Main
- Road Edge
- Streams

SubAreas

- Subarea 1
- Subarea 2
- Subarea 3
- Subarea 4
- Subarea 5
- Subarea 6
- Waterbodies
- Swamps

- Manholes with Defects
- Manholes - Existing

0 200 400 800 Feet

N

FIGURE 4-7

SUMMARY OF 2013 MANHOLE INSPECTION AND SMOKE TESTING SEWER DEFECTS

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 4-7 - Manhole Inspection - Updated.mxd reimbursement, say $500 or $1,000. Other communities have elected to reimburse the homeowner for the full amount of the redirection, with the cost supported by invoices for the completed work from the homeowner's contractor. Other communities have elected to undertake the work as noted above either with their own forces or using contracted services. Once the "amnesty" period ends, if a homeowner is discovered to have a sump pump connected to the sewer system, the Town levies a fine, the homeowner is required to permanently redirect the sump pump with no funding assistance from the Town, and a follow-up redirection inspection is conducted. To implement this approach, Ridgefield's Sewer Use Regulations would need to be amended to incorporate the fine. The amnesty programs have been successful because of the financial incentive typically offered to homeowners. A similar program could be used to address other types of private sources of inflow.

Once the Town has adopted a policy and procedure to correct private inflow sources, then it is recommended that the Town undertake a program to identify and remove sump pumps from the wastewater collection system in District 1. The Town should undertake the necessary investigations to locate sump pumps. This will involve conducting a house-to-house internal building inspection program to identify sump pumps connected to the sewer collection system.

Following these efforts, it is recommended that the Town undertake the dye water testing and tracing outlined in Technical Memorandum No. 1 to verify the presence of the connections of suspect sources identified during smoke testing.

Finally, the implementation of a program to correct private inflow sources should be undertaken. The above recommended steps to address private inflow sources are outlined in Table 4-7.

TABLE 4-7. RECOMMENDED STEPS TO CORRECT PRIVATE INFLOW SOURCES

Step	Description
1	Develop a Policy and Procedure Regarding Correction of Privately Owned Inflow Sources
2	Conduct Public Education and Outreach Program
3	Conduct House-to House-Inspections
4	Conduct Dye Water Testing and Tracing
5	Implement Procedure to Correct Private Inflow Sources

There is one additional step that is recommended to further address I/I sources – selected television inspection. The latest television inspection data by United Water was collected from 2005 - 2010, and most of the TV inspections were conducted in the late summer or early fall, when groundwater levels are typically lower than average. There has been previous discussion that some of the observed I/I in the collection system may be entering through some of the unusually long lateral service connections present in Sewer District 1, particularly on both sides of Main Street. Due to topography, there is no sewer in Main Street. All of the sewered properties on Main Street are connected through lengthy lateral service connections to sewers in low lying easements to the east and west of Main Street. To assess the potential for laterals to contribute significant I/I, it is recommended that a representative number of manhole to manhole segments, 8-10 segments, be television inspected during the spring high groundwater season to observe leakage from both the mainline sewer and the service laterals. A lateral inspection camera can then be deployed to further observe leakage within the service lateral connections that may be observed to be leaking. It would also be valuable to confirm whether the buildings served by apparent leaking laterals do not have a sump pump that could be contributing the observed clean water flow. The extent of potential service lateral inspection varies with the number of bends and condition of the service lateral piping. If significant leakage is observed, the lateral can either be lined or replaced to eliminate the leakage.

Table 4-8 presents a summary of the recommended steps to address ongoing I/I and collection system bottleneck issues. It also presents a summary of the estimated costs for the components of the recommended program to address I/I and collection system bottlenecks. The costs presented in this table include an allowance for engineering and contingencies and are planning level cost estimates for budgeting purposes. A more accurate estimate of the anticipated costs may be determined during subsequent phases of the recommended program.

TABLE 4-8. RECOMMENDED STEPS TO ADDRESS I/I AND COLLECTION SYSTEM BOTTLENECK ISSUES

Step	Description	Estimated Cost
1	Divert Flows Around Bottlenecks – Relocate Quail Ridge Pump Station	$2,200,000
2	Conduct Routine Maintenance of the Collection System *(1)*	N/A
3	Develop a Policy and Procedure Regarding Correction of Privately Owned Inflow Sources *(1)*	N/A
4	Conduct Public Education and Outreach Program	$30,000
	Conduct Further Investigations Including:	
	Manhole and Structure Inspections (558)	$95,000
	Dyed Water Testing and Tracing	$53,000
	House-to House-Inspections (1,760) *(2)*	$184,000
5	Selected Television Inspection	$28,000
6	Implement Procedure to Correct Private Inflow Sources *(2)*	$38,000
7	Conduct Manhole Rehabilitations *(2)*	$201,000

Notes:

1. *Costs associated with this effort should be included in the WPCA's operating budget.*
2. *Identified during 2013 Smoke Testing and Manhole Inspections.*

Technical Memorandum No. 3, contained in Appendix H, recommends that the remaining manholes in Sewer District 1 be inspected and that the defective manholes identified be repaired. These efforts may be completed simultaneously with the work outlined above. However, there may be an advantage to completing the recommended investigative work prior to beginning rehabilitation work. By completing the investigative work prior to beginning the rehabilitation work the Town may be able to take advantage of economies of scale by packaging the recommended rehabilitation work in one or two construction contracts, instead of multiple contracts as the investigative work is completed.

CHAPTER FIVE
PUMP STATION EVALUATION UPDATE

INTRODUCTION

The majority of the Ridgefield collection system pump stations have been upgraded in the past few years. However, the two oldest pump stations, the Quail Ridge Pump Station and the Route 7 WWTF Influent Pump Station, have not received significant upgrades in many years. Due to the age of these pump stations there is a concern about their ability to provide reliable service for the next 20 years.

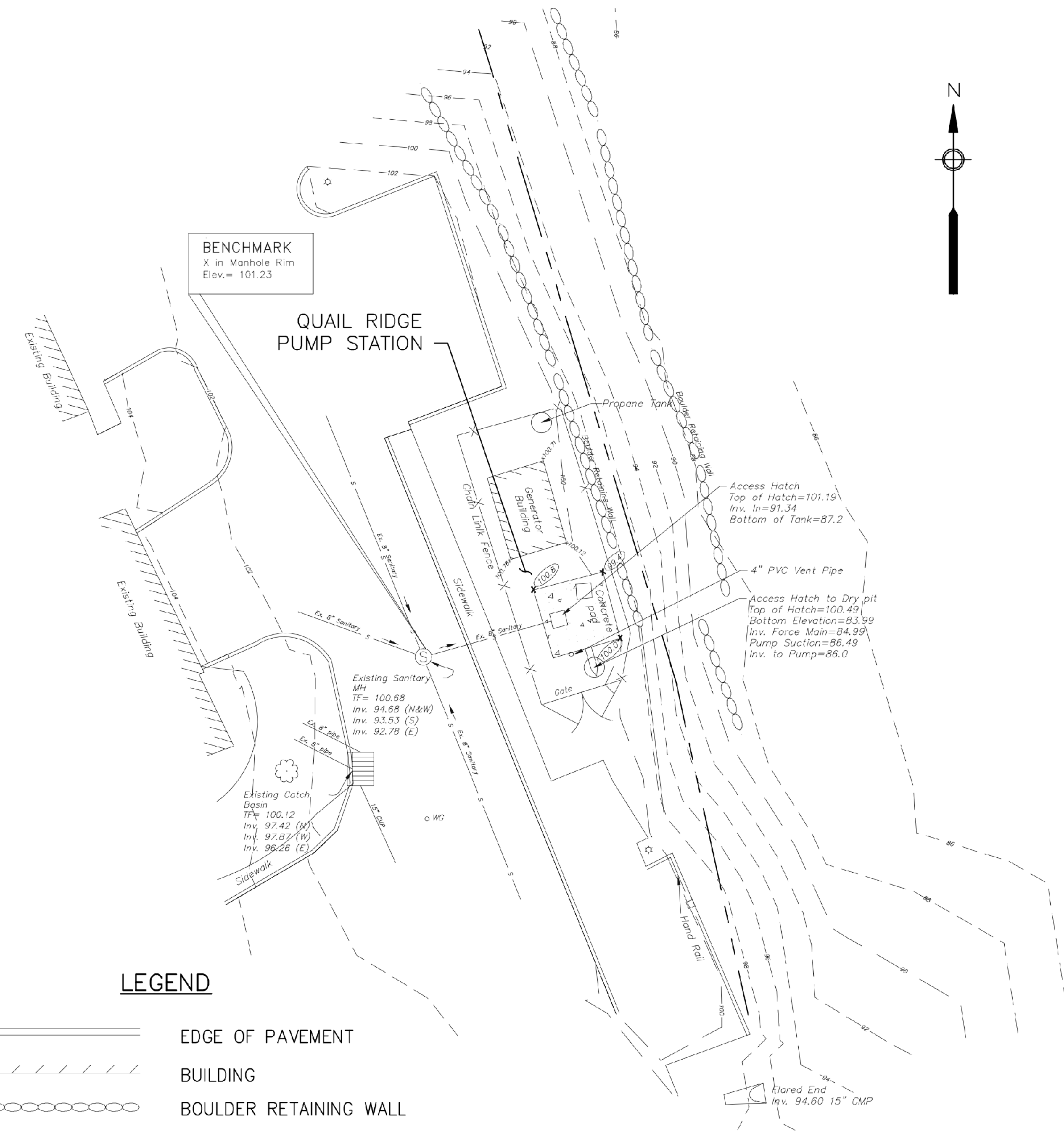
The Quail Ridge and Route 7 Pump Stations were evaluated and their upgrade needs were defined as part of the 2003 Final Pump Station Preliminary Design Report (2003 Report). The 2003 Report concluded that the mechanical and electrical equipment at each station was reaching the end of its design life and that it was in need of replacement to meet the projected design flows. As part of the Phase 1 Facilities Plan, the evaluation and upgrade needs for the Quail Ridge Pump Station and Route 7 WWTF Influent Pump Station were revisited and the estimated upgrade project costs updated. In addition, the potential to eliminate the Quail Ridge Pump Station by construction of a gravity sewer to the South Street WWTF was assessed. A summary of these evaluations are provided below. More detailed descriptions of the evaluations are provided in Technical Memorandum No. 4 – Pump Station Evaluation Upgrade which is included in Appendix I

QUAIL RIDGE PUMP STATION

Description

The Quail Ridge Pump Station is a Smith & Loveless package pump station consisting of a steel dry well and a precast concrete wet well. The pump station is about 29 years old, and is equipped with duplex 15 HP pumps each with a rated capacity of approximately 100 gallons per minute (gpm). Both pump motors were rebuilt approximately 16 years ago. The pump stands, suction elbows and gate valves were replaced in 2001. The pump station is equipped with a bubble tube level detection system. A dehumidifier is located in the dry well on a wall-mounted shelf. A 30-kW Empire standby propane generator is housed in an above ground wood framed structure at the site. A 6-foot chain-link fence is located around the entire pump station. The chain link fence is surrounded by a 6-foot wooden fence which provides a visual barrier for the pump station. Three-phase power is not available at the site and a Ronk phase converter is located in the generator building. An existing conditions site plan is shown on Figure 5-1.

As noted above, the Quail Ridge Pump Station was evaluated and its upgrade needs were defined as part of the 2003 Final Pump Station Preliminary Design Report (2003 Report). Since the 2003 Report, zoning changes have been implemented in the Quail Ridge Pump Station service area. These zoning changes impact the projected flows for the pump station. In addition to the zoning changes, the potential to eliminate the Quail Ridge Pump Station by construction of a gravity sewer to the South Street WWTF was assessed. The potential to eliminate the Quail Ridge Pump Station by construction of a gravity sewer to the South Street WWTF was assessed previously in Technical Memorandum No. 2 – Collection System Bottleneck Evaluation included as Appendix G**.** While it was found that it would not be possible to construct a gravity sewer between the existing Quail Ridge Pump Station and the South Street WWTF, the feasibility of eliminating the Quail Ridge Pump Station by constructing a new pump station closer to the South Street WWTF was considered.



LEGEND

- EDGE OF PAVEMENT
- BUILDING
- BOULDER RETAINING WALL
- CHAIN LINK FENCE
- PROPERTY LINE
- LAMP POST
- SANITARY MANHOLE
- CATCH BASIN

0 20 40 60

SCALE FEET

VICINITY MAP

NOTES:

1. THIS PLAN WAS COMPILED FROM OTHER MAPS, RECORD RESEARCH OR OTHER SOURCES OF INFORMATION. IT IS NOT TO BE CONSTRUED AS HAVING BEEN OBTAINED AS THE RESULT OF A FIELD SURVEY, AND IS SUBJECT TO SUCH CHANGE AS AN ACCURATE FIELD SURVEY MAY DISCLOSE.
2. THIS SURVEY AND MAP HAVE BEEN PREPARED PURSUANT TO THE REGULATIONS OF CONNECTICUT STATE AGENCIES SECTIONS 20–300b–1 THROUGH 20–300b–20 AND THE "STANDARDS FOR SURVEYS AND MAPS IN THE STATE OF CONNECTICUT" AS ADOPTED BY THE CONNECTICUT ASSOCIATION OF LAND SURVEYORS, INC., ON SEPTEMBER 26, 1996.
3. THE TYPE OF SURVEY PERFORMED IS AND MAPPED FEATURES DEPICTED HEREON ARE IN ACCORDANCE WITH THE REQUIREMENTS OF A COMPILATION PLAN.
4. THERE IS NO BOUNDARY DETERMINATION.
5. WITH RESPECT TO THE PROPERTY LINES AND MAPPED FEATURES, THIS SURVEY CONFORMS TO CLASS D. WITH RESPECT TO THE TOPOGRAPHY, THIS SURVEY CONFORMS TO CLASS T–2.
6. THIS SITE WAS FIELD SURVEYED ON MARCH 5, 2002 AND THE CONTOURS DEPICTED HEREON ARE IN TWO FOOT INTERVALS AND ELEVATIONS ARE BASED ON ASSUMED DATUM.
7. LOCATION OF UNDERGROUND UTILITIES DEPICTED HEREON ARE APPROXIMATE AND ARE BASED ON FIELD LOCATION OF VISIBLE STRUCTURES SUCH AS CATCH BASINS, MANHOLES, WATER GATES, ETC., AND COMPILING INFORMATION FROM PLANS SUPPLIED BY RESPECTIVE UTILITY COMPANIES AND GOVERNMENT AGENCIES. ALL CONTRACTORS ARE REQUIRED BY STATE REGULATIONS TO CONTACT CALL–BEFORE–YOU–DIG AT 1–800–922–4455 FOR LOCATION AND STAKEOUT OF UTILITIES PRIOR TO ANY EXCAVATION.

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Drawn:	BC	4/24/2015
Approved:	AA	4/24/2015

FIGURE 5-1

QUAIL RIDGE PUMP STATION
EXISTING CONDITIONS SITE PLAN
PUMP STATION EVALUATION UPDATE

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 5-1 Quail Ridge Pump Station1 sme.mxd

Feasibility to Eliminate the Existing Quail Ridge Pump Station

Eliminating the existing Quail Ridge Pump Station would involve the construction of approximately 2,500 linear feet of gravity sewer from the current Quail Ridge Pump Station location, through easements across the northeastern portion of the former Schlumberger property, along the edge of the wetlands, to a new pump station located generally in the vicinity of the Goodwill trailer on South Street. From the new pump station location, a new force main would convey the wastewater flows either indirectly into the existing gravity sewer on Old Quarry Road which discharges to the South Street WWTF (Route A) or directly to the South Street WWTF (Route B). Figure 5-2 presents the location of the new gravity sewer for the existing Quail Ridge Pump Station, the new Quail Ridge Pump Station and the two force main routes being considered. Force main Routes A and B are 650 linear feet and 1,300 linear feet, respectively.

Quail Ridge Pump Station Upgrade Alternatives

The option of updating the existing Quail Ridge Pump Station at its current location (Alternative 1) was evaluated comparatively to eliminating the Quail Ridge Pump Station by providing a gravity sewer, new pump station and either of the two force main routes noted above (Alternative 2). This evaluation is described below.

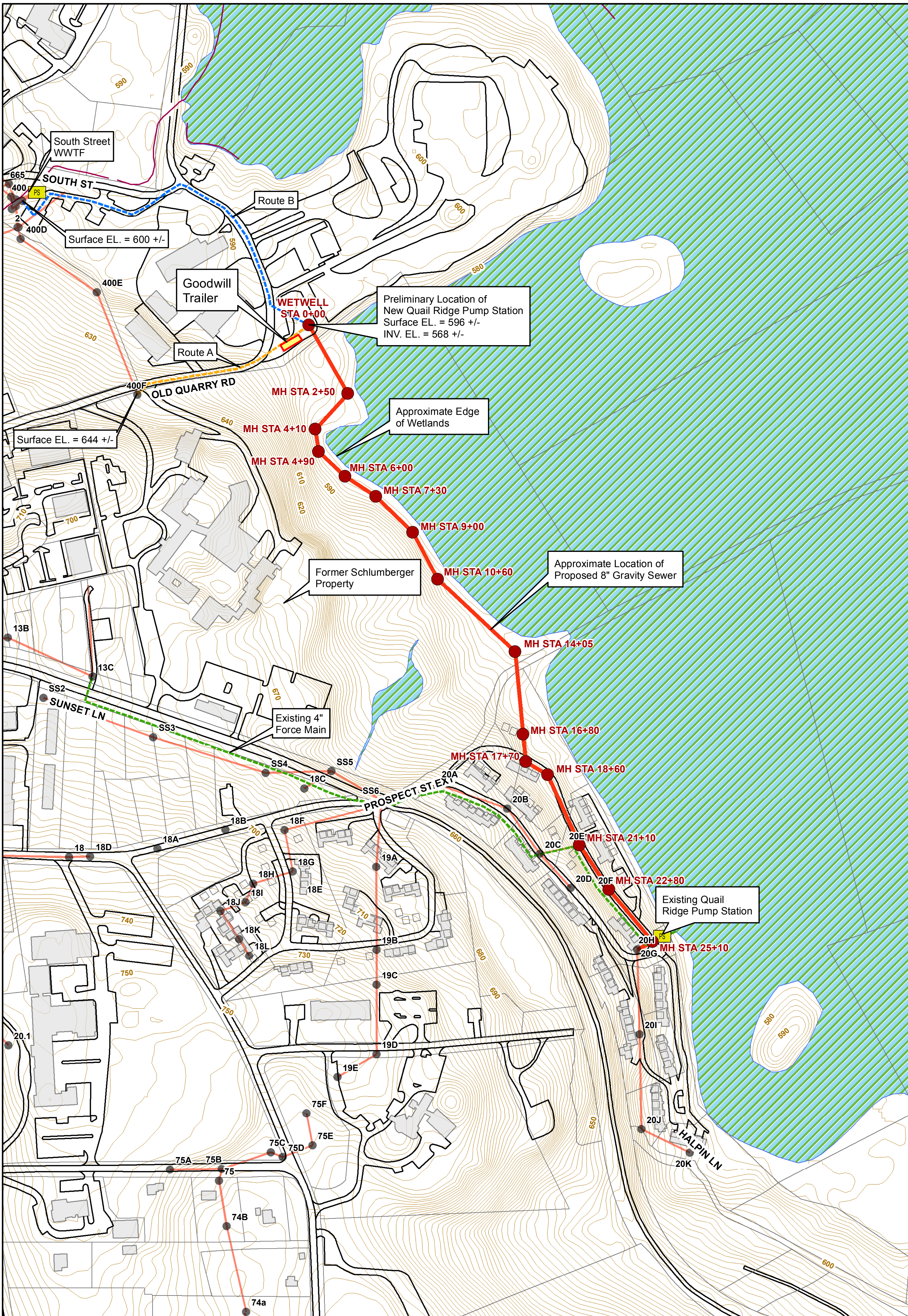
Preliminary Design Criteria for Quail Ridge Pump Station Upgrade Alternatives

Existing and Projected Flows. The current average daily and peak flows for Alternative 1 and the projected average daily and peak flows for Alternative 2 were estimated. The estimated existing average daily and peak flows to the Quail Ridge Pump Station (alternative 1) are approximately 58,000 gal/day and 180,000 gal/day, respectively or 40 gpm and 125 gpm. The estimated peak flow of 125 gpm exceeds the existing pump capacity. The projected future average daily and peak flows for the new pump station located near the Goodwill trailer on South Street (Alternative 2) are approximately 116,000 gal/day and 362,000 gal/day, respectively or 85 gpm and 260 gpm.

Pump Capacity. Pump selections were evaluated for both Alternatives 1 and 2 (for both route) using both 4 inch and 6 force mains and increasing the pumping flow capacities as needed to provide recommended pipe velocities . The energy losses due to friction at the design pumping rate combined with the static head energy losses for the alternatives and force main routes were also considerd. The evaluation concluded that a 4 inch force main was preferred for Alternative 1 and a 6 inch force main was preferred for Alternative No. 2. For Alternative 1 the pump selection would be two constant speed pumps with a capacity of 136 gpm at 139 feet of total dynamic head. For Alternative 2 the pump selection would be two constant speed pumps with a capacity of 309 gpm at 80 feet of total dynamic head for Route A and a capacity of 309 gpm at 42 feet of total dynamic head for Route B. While Route A minimizes the length of force main construction, the total dynamic head is greater than that of Route B. Because a low total dynamic head translates into lower operating costs, Route B with a 6-inch force main was recommended for Alternative 2.

Alternative 1 – Existing Pump Station Upgrade Recommendations

Based on an evaluation of the existing pump station the following components are recommended to upgrade the existing Quail Ridge Pump Station:



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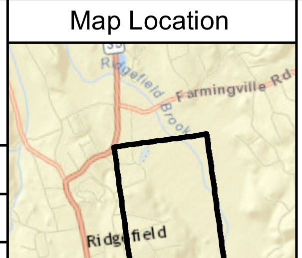
Drawn:	BC 4/23/2015
Approved:	AA 4/23/2015

Map Location



Legend

- PS Existing Pump Station
- Proposed Manholes
- Proposed Sewer
- Existing Manholes
- Existing Sewer
- Proposed Force Main Route A
- Proposed Force Main Route B
- Existing Force Main
- Swamps
- Edge of Pavement
- Buildings
- Contour

0 100 200 400 Feet

N

FIGURE 5-2

NEW QUAIL RIDGE PUMP STATION, GRAVITY, SEWER AND FORCE MAIN ROUTE ALTERNATIVES

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 5-2 - Quail Ridge Route Alternatives.mxd

1. Construct a new wet well and valve pit in the location of the existing generator building.
2. Provide two constant speed solids handling pumps.
3. Provide a new diesel-fired generator located in a weather proof, sound attenuating enclosure. The new generator would include a new transfer switch to automatically transfer power supply to/from the generator during a power outage and a belly tank to provide fuel storage for up to 48 hours of generator operation at full load. The generator would be located where the existing wet well and dry pit are currently situated. As a result temporary standby power would be required throughout the construction period.
4. Extend the sewer and force main to the location of the new wet well.
5. Provide a new three phase electrical service consisting of 1,400 ft of underground cable, and a transformer.
6. Replace electrical power supply components, such as the disconnect switch, motor starters, and site wiring. These electrical components would be located outside in all-weather panels.
7. Replace the perimeter fence and loam and seed the site.

Alternative 2 – Relocated Pump Station Recommendations

Based on an evaluation the following components are recommended for the new Quail Ridge Pump Station in the vicinity of the Goodwill trailer on South Street:

1. Provide a new 8-inch gravity sewer to be extended from the existing Quail Ridge Pump Station
2. Construct a new submersible pump station on town owned property in the vicinity of the Goodwill trailer on South Street.
3. Provide two constant speed submersible solids handling pumps.
4. Provide new three phase electrical power supply components, such as the disconnect switch, motor starters, and site wiring. These electrical components would be located outside in all-weather panels.
5. Provide a diesel-fired generator located in a weather proof, sound attenuating enclosure. The generator would include a transfer switch to automatically transfer power supply to/from the generator during a power outage and a belly tank to provide fuel storage for up to 48 hours of generator operation at full load.
6. Provide grading, paving and site restoration, including a perimeter fence around the new pump station.
7. Provide a new 6-inch force main extending from the new wet well to the South Street WWTF (Route B).

Under both alternatives, wet well level monitoring would be accomplished using a new submerged pressure transducer located in the wet well with backup float switches for high and low wet well level alarms. Flow metering and programmable logic controller (PLC) based pump controls would also be provided. The flow meter would have a local instantaneous and totalizer flow readout. Alarms from the pump station would be routed through a telemetry system to the town's alarm service. Backup float switches would be provided for high and low wet well level alarms.

Alternative Costs and Recommendations

The estimated cost of Alternative 1 is $1,400,000 and the estimated cost of Alternative 2 is $2,200,000. Both costs include a 40% allowance for engineering and contingencies.

An advantage of Alternative 2 is that the new pumps would operate at a lower total dynamic head than those at the current pump station. This would translate into lower operating costs. The existing Quail Ridge Pump Station force main currently discharges to an existing 12-inch gravity sewer in the easement adjacent to Grove Street. Another advantage of Alternative 2 would be the diversion of flows from this portion of the collection system that at times is overburdened. Alternative 2 also provides the opportunity to eliminate the Highway Department pump station by intercepting flows from the municipal buildings, which currently discharge to it, and redirecting them to the new pump station. A disadvantage of Alternative 2 is the higher capital costs associated with the construction of a new cross country gravity sewer and a new force main. Based on the discussion presented above, AECOM recommends Alternative 2.

ROUTE 7 PUMP STATION

Description

The Route 7 Pump Station is a Smith & Loveless package pump station consisting of a steel dry well and a precast concrete wet well. The pump station, which has duplex 15 HP pumps each with a rated capacity of approximately 500 gpm at 73 feet total dynamic head (TDH), is about 29 years old and contains most of its original equipment. The pump station has a 60 kW Kohler standby emergency generator that is diesel powered, and consists of a day tank, a subsurface fuel oil tank and a fiberglass generator enclosure located partially below ground. An existing conditions site plan is shown on Figure 5-3.

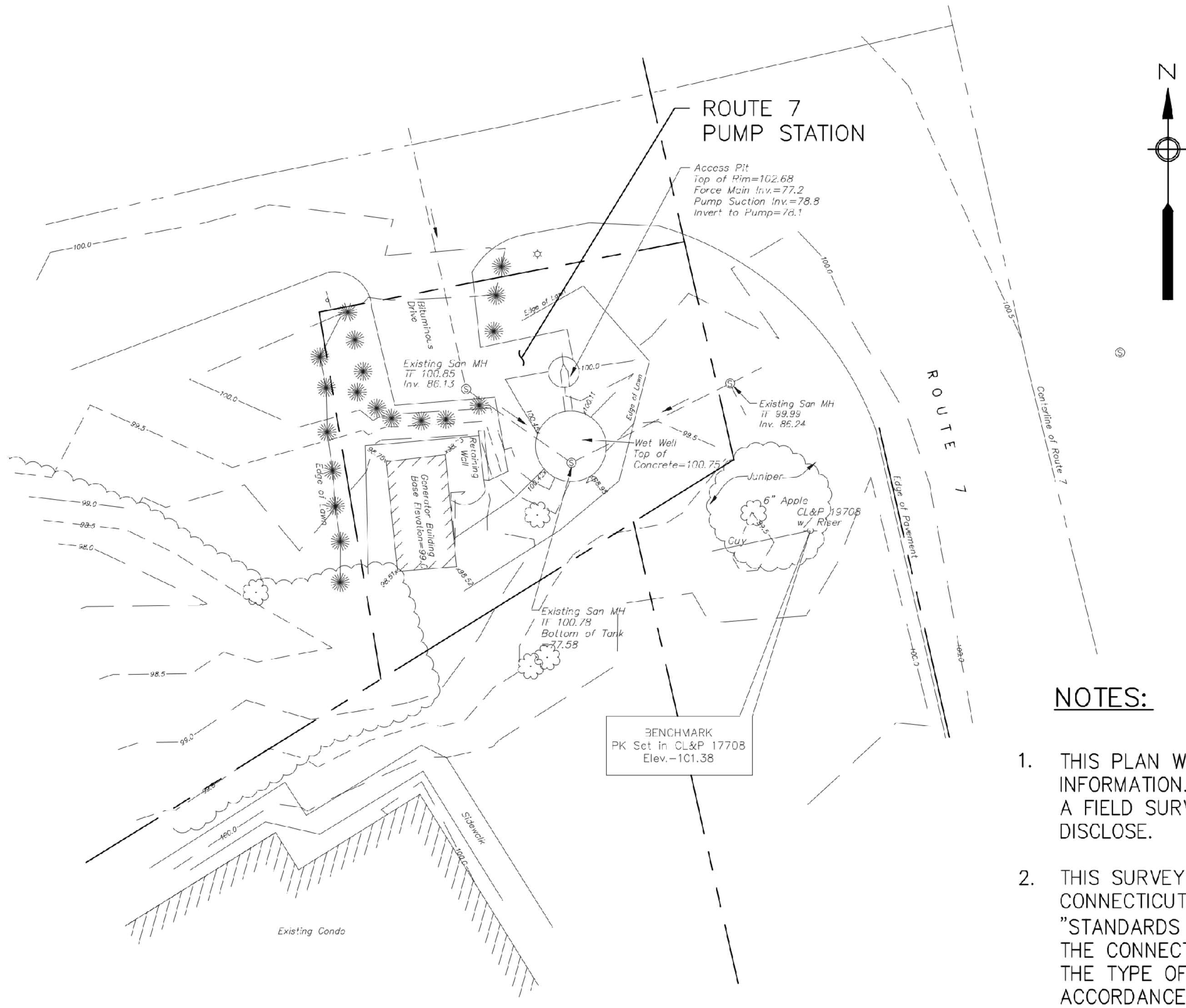
Existing and Projected Flows

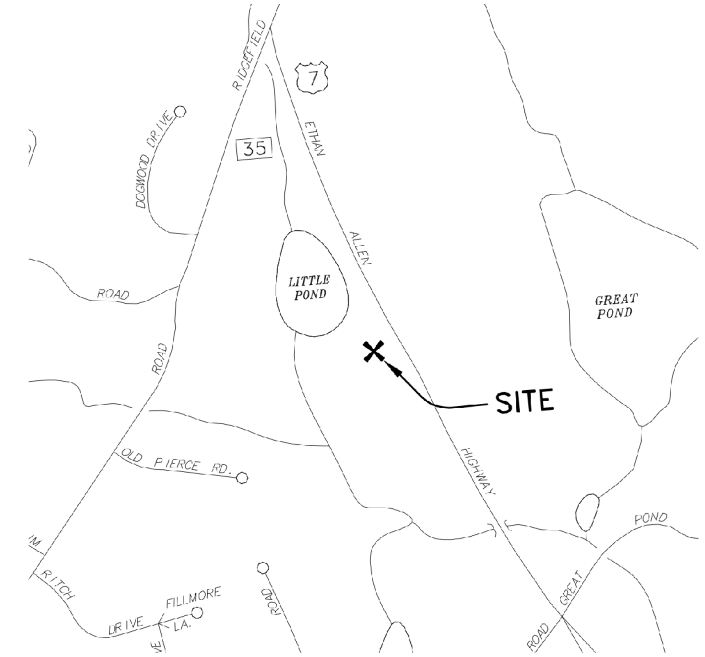
The evaluation estimated both the current and projected average daily and peak flows for the Route 7 Pump Station. The estimated current average daily and peak flows are approximately 54,000 gal/day and 357,000 gal/day respectively, or 38 gpm and 250 gpm. The projected average daily and peak flows are therefore approximately 120,000 gal/day and 720,000 gal/day respectively, or 85 gpm and 500 gpm. The existing and project peak flows do not exceed the existing pump capacity of 500 gpm.

Preliminary Design Criteria for the Route 7 Pump Station

Pump Capacity. The peak design flow for the Route 7 Pump Station is 500 gpm. It is recommended that the pumps be equipped with variable frequency drives (VFD’s) to reduce flow surges at the Route 7 WWTF. It is recommend that in lieu of providing two large pumps each sized for 500 gpm, three smaller pumps each sized to accommodate half of the design flow are recommended to be provided. It is anticipated that one pump would be adequate to keep up with pump station demand most of the time and two pumps would be required to convey peak wastewater flows with the third pump as an installed standby.

Existing Force Main. The existing Route 7 force main is 8-inch ductile iron, approximately 2,500 feet long and runs from the pump station directly to the Route 7 WWTF. The elevation rise from the wet well to the force main discharge is approximately 50 feet. The energy losses due to friction at the design pumping rate combined with the static head energy losses at this pump station would result in a total dynamic head of approximately 75 feet with the existing 8-inch force main.





VICINITY MAP

NOTES:

1. THIS PLAN WAS COMPILED FROM OTHER MAPS, RECORD RESEARCH OR OTHER SOURCES OF INFORMATION. IT IS NOT TO BE CONSTRUED AS HAVING BEEN OBTAINED AS THE RESULT OF A FIELD SURVEY, AND IS SUBJECT TO SUCH CHANGE AS AN ACCURATE FIELD SURVEY MAY DISCLOSE.

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3. THERE IS NO BOUNDARY DETERMINATION.

4. WITH RESPECT TO THE PROPERTY LINES AND MAPPED FEATURES, THIS SURVEY CONFORMS TO CLASS D. WITH RESPECT TO THE TOPOGRAPHY, THIS SURVEY CONFORMS TO CLASS T-2.

5. THIS SITE WAS FIELD SURVEYED ON MARCH 5, 2002 AND THE CONTOURS DEPICTED HEREON ARE IN TWO FOOT INTERVALS AND ELEVATIONS ARE BASED ON ASSUMED DATUM.

6. LOCATION OF UNDERGROUND UTILITIES DEPICTED HEREON ARE APPROXIMATE AND ARE BASED ON FIELD LOCATION OF VISIBLE STRUCTURES SUCH AS CATCH BASINS, MANHOLES, WATER GATES, ETC., AND COMPILING INFORMATION FROM PLANS SUPPLIED BY RESPECTIVE UTILITY COMPANIES AND GOVERNMENT AGENCIES. ALL CONTRACTORS ARE REQUIRED BY STATE REGULATIONS TO CONTACT CALL-BEFORE-YOU-DIG AT 1-800-922-4455 FOR LOCATION AND STAKEOUT OF UTILITIES PRIOR TO ANY EXCAVATION.

LEGEND

EDGE OF PAVEMENT
BUILDING
TREELINE
CHAIN LINK FENCE
PROPERTY LINE
LAMP POST
SANITARY MANHOLE

0 20 40 60
SCALE FEET

AECOM

Drawn:	BC 4/24/2015
Approved:	AA 4/24/2015

FIGURE 5-3

ROUTE 7 PUMP STATION
EXISTING CONDITIONS SITE PLAN

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

Rehabilitation Recommendations

Based on an evaluation of the condition of the pump station and the ability to accommodate the recommended three replacement pumps the following the rehabilitation items are recommended:

1. Abandon the existing dry pit installation and replace with a new submersible wet well with an adjacent valve pit located on the existing site.
2. Replacement of the two existing pumps with three VFD driven solids handling centrifugal submersible pumps installed in a new concrete wet well.
3. Provide wet well level monitoring using a submersible pressure transducer, flow metering, and PLC based pump controls. The flow meter would have a local instantaneous and totalizer flow readout.
4. Provide alarms from the pump station to be routed through a telemetry system to the town's alarm service. Backup float switches would be provided for high and low wet well level alarms.
5. Replace electrical power supply components, such as the disconnect switch, motor starters, and site wiring. Locate new electrical components, such as the pump control panel and VFDs outside in all-weather panels.
6. Replace the existing generator with a new diesel-fired generator located in a pad mounted enclosure. The new generator would include a transfer switch to automatically transfer power supply to/from the generator during a power outage and a belly tank would provide diesel fuel storage for up to 48 hours of generator operation at full load.
7. A perimeter fence would be provided around the new pump station and the site would be loamed and seeded.

Rehabilitation Costs

The estimated cost of the Route 7 Pump Station Rehabilitation is $1,500,000, including a 40% allowance for engineering and contingencies.

CHAPTER SIX
LAND APPLICATION FEASIBILITY EVALUATION

INTRODUCTION

Based on the anticipated more restrictive NPDES permit limit for phosphorus in the South Street WWTF effluent, an alternative to upgrading the WWTF to meet the more stringent phosphorus limit would be to land apply all or a portion of the WWTF effluent. Since it is anticipated that the WWTF effluent concentration limits for land application of effluent will not be as stringent as the surface water discharge limits, there is the potential to reduce the overall expenditures and remain in compliance with the effluent discharge requirements with land application of all or a portion of the WWTF effluent. This alternative approach to meeting the limits was evaluated and the methodologies used as well as the results of the evaluation are presented in this chapter.

BACKGROUND

There are a number of treatment alternatives to remove phosphorus from a WWTF effluent discharge prior to entering surface waters. Since the proposed phosphorus limit for the South Street WWTF is mass based, and not a concentration limit, an alternative to a surface water discharge and higher treatment levels is discharging all or a portion of the WWTF effluent to groundwater through land application. Land application allows the discharge to flow through subsurface soils prior to the discharge entering surface waters. Phosphorus has an affinity for soil particles which limits its travel in the groundwater. In other words, the soils treat the phosphorus in the effluent instead of at the WWTF. The result is very low levels of phosphorus reaching the surface water with potential savings in treatment costs.

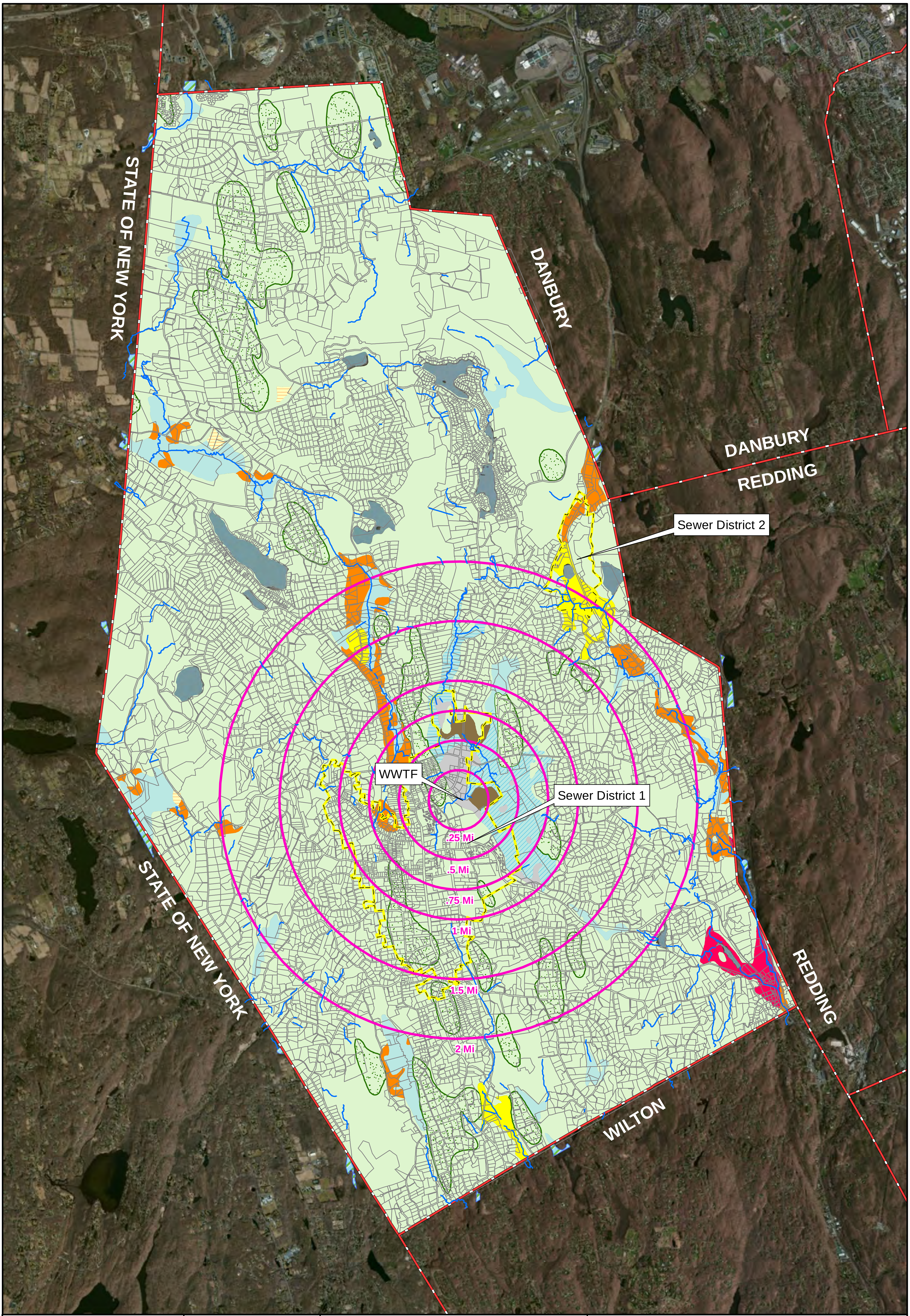
Land application of WWTF effluent would require new infrastructure such as pump stations, conveyance piping, and disposal infrastructure such as rapid infiltration basins or soil absorption systems. Land application can be a cost effective method to reduce phosphorus when compared to other treatment methods. However, as with other technologies there are variables that factor into the cost effectiveness of its application. In part, these variables include:

- Soils Type
- Acreage of Favorable Soils
- Estimated Depth to Groundwater
- Parcel Ownership
- Parcel Development
- Distance from WWTF

There are other factors that can influence the cost effectiveness of land application; however those listed above are generally the most significant factors in cost.

REVIEW OF EXISTING SOILS DATA

Soils data provided by USGS and USDA Natural Resources Conservation Service reports were reviewed to identify areas where sand or sand and gravel outwash deposits had been mapped. The review was conducted to identify parcels with soil characteristics conducive to land application of WWTF effluent. Only those parcels that appear to contain sand or sand and gravel soils were considered for further review. Parcels underlain only by till and/or bedrock, over 90 percent of the parcels reviewed, were eliminated from further review. Parcels mapped as having favorable soils such sand, or sand and gravel, were identified in GIS and mapped for further evaluation. Figure 6-1 shows areas mapped as having soils potentially favorable for land application.



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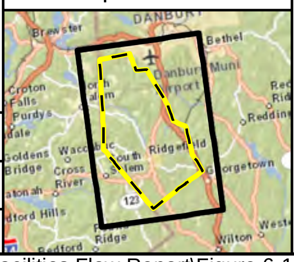
Drawn:	BC	4/13/2015
Approved:	MO	4/13/2015

Map Location



Legend

- Streams
- NWI Wetlands
- Town Boundaries
- Sewer District Boundary
- Parcels
- WWTP Buffer Distance

GLACIAL ICE-LAID DEPOSITS

- Thin Till
- Thick Till
- End Moraine Deposits

GLACIAL MELTWATER DEPOSITS

- Gravel
- Sand and Gravel
- Sand
- Sand Overlying Fines

POSTGLACIAL DEPOSITS

- Alluvium Overlying Undifferentiated Coarse Deposits
- Swamp Deposits
- Swamp Deposits Overlying Fines
- Swamp Deposits Overlying Sand Overlying Fines
- Artificial Fill

0 2,000 4,000 8,000 Feet

N

FIGURE 6-1

SURFICIAL GEOLOGY

PHASE I WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 6-1 - Surficial Geology.mxd

GIS DATABASE

The GIS database consisted of the Town's GIS parcel database along with data from USDA (soils data), CT DEEP (surficial geology), and National Wetlands Inventory (NWI) (wetland areas). The GIS database was used to determine property ownership, parcel development, parcel acreage, acreage of each soil type per parcel, a straight-line distance between potential land application sites and the WWTF, as well as other parcel specific data.

PARCEL RATING CRITERIA AND ANALYSIS

Data for parcels identified with favorable soils characteristics were summarized in a matrix for further evaluation. Within the Town of Ridgefield there are 9,077 parcels. Of those parcels, 59 were identified as having some soils favorable for land application. The following criteria were used to evaluate and further shortlist these parcels by eliminating unfavorable parcels for further review.

1. Soil Type - Most parcels in Ridgefield are underlain by till and bedrock. Till contains a mixture of soils including clay, silt, sand gravel, cobbles and boulders. The unsorted mixture of these soils holds water, generally resulting in a high water table that is often only a few feet below the ground surface. Even in areas where there is a significant depth to groundwater, till is poor at allowing groundwater to flow through it. As a result, till can be suitable for small wastewater discharges such as single family septic systems, but not for larger WWTF discharges. As a result, sites underlain only by till and bedrock were eliminated for further review.

2. Developed Small Parcels - Also eliminated from review were developed, privately owned parcels less than 5 acres in size. A majority of these parcels are residential with little land remaining for land application. These smaller, developed parcels could be reevaluated if they abut or are located near a potential land application site.

3. Distance from the WWTF - The distance of the parcel from the WWTF was also used to eliminate parcels from further review. Significant infrastructure and cost are necessary to pump effluent from a WWTF to a land application site. In general, the greater the distance, the greater the cost, and the less cost effective land application becomes. At a point, the cost benefit of land application exceeds the additional treatment necessary to discharge directly to surface water. Parcels located over 2 miles from the South Street WWTF were generally eliminated from further consideration.

Once the criteria above were applied, 38 parcels out of the original 59 remained for review. These shortlisted parcels were further evaluated using a matrix to rank sites. Parcel data for each was summarized in a matrix, described below.

MATRIX

Using GIS, a matrix was created to summarize data for each of the shortlisted parcels. The matrix summarizes the data by parcel includes the following:

- Street Address
- Owner Name
- GIS Parcel ID
- Developed or Undeveloped Property
- Parcel Ownership Category
- Acreage of Parcel
- USDA Soils Grade
- USDA Soils Description
- Soils Drainage Classification
- Hydric Soils Classification

- Acreage of Soils Type by Parcel
- Distance of Potential Discharge from WWTF
- Map, Lot and Block Number
- Zoning Code
- Estimated Depth to Groundwater

As noted above, all shortlisted parcels having potentially favorable soils for land application were rated using several criteria. The GIS parcel data used to rank each parcel for land application are as follows:

- Parcel Ownership
- Parcel Development
- Soils Type
- Acreage of Favorable Soils
- Distance from WWTF
- Estimated Depth to Groundwater

Each parcel was ranked between 1 and 5 for each of the above criteria. A value of 5 was considered very favorable while a value of 1 was considered not favorable.

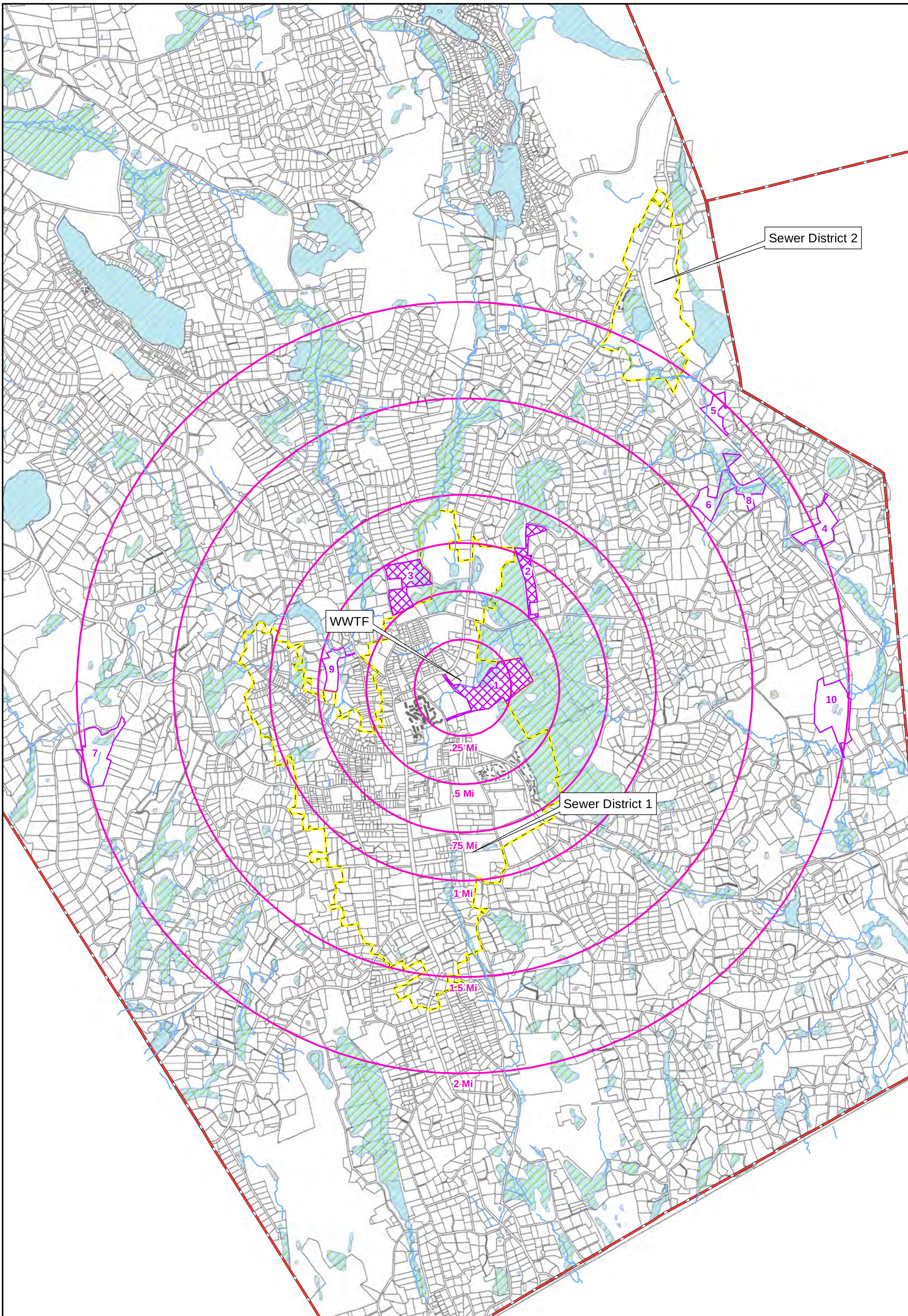
RANKING OF PARCELS

The criteria described above were used to rank the shortlisted parcels in the matrix. The potential range of values for ranked parcels is between 6 and 30. Most parcels scored below 12. The highest ranking parcel was 25. The remainder of the shortlisted parcels ranked between 13 and 22. Overall these are relatively low rankings as typically there would be several parcels with rankings above 25. The lower overall ranking is due to several factors including few parcels with sand and gravel deposits, most sand and gravel deposits are low-lying valley deposits with little elevation above the water table, and many of these parcels are smaller parcels that have already been developed.

The ten highest ranking shortlisted parcels of the 38 reviewed are summarized in Table 6-1 and shown on Figure 6-2. The pros and cons of the three highest ranking parcels are summarized in Table 6-2.

TABLE 6-1. RANKING OF MOST FAVORABLE PARCELS

Number	Street Address	Parcel Ownership	Parcel ID Number	Total Rating
1	45 South Street	Ridgefield Town Of	E14-0158	25
2	Norrans Ridge Drive	Ridgefield, Town Of	F13-0037	22
3	North Street	St. Marys Corp	E13-0056	22
4	Bobbys Court	Ridgefield Town Of	H12-0074	19
5	Ethan Allen Highway	Ridgefield, Town Of	G11-0064	17
6	Stonehenge Road	Ridgefield, Town Of	G12-0016	17
7	Peaceable Hill Road	Ridgefield, Town Of	C14-0021	17
8	Ethan Allen Highway	Ridgefield Town Of	G12-0048	16
9	15 Sawmill Hill Road	Tighe Maureen	E13-0014	16
10	323 Florida Hill Road	Julian Alexander	H14-0014	13



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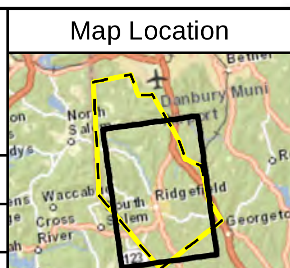
Drawn: BC 4/13/2015

Approved: AA 4/13/2015

Map Location



Legend

- Streams
- Favorable Site
- Most Favorable Site
- WWTP Buffer Distance
- Sewer District Boundary
- Town Boundaries
- Waterbodies
- NWI Wetlands
- Parcels

0 1,250 2,500 5,000 Feet

N

FIGURE 6-2

LOCATION OF TEN HIGHEST RANKED PARCELS

PHASE I WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 6-2 - Ten Highest Ranked Parcels.mxd

TABLE 6-2. PROS AND CONS OF THREE MOST FAVORABLE PARCELS

Number	Parcel ID	Pros	Cons
1	E14-0158	• Town owned Parcel • Parcel approximately 36.5 acres • Approximately 8.5 acres of parcel mapped as favorable soils • Located within 0.25 miles of the WWTF • Located inside of the Sewer District	• Partially developed • Mapped as fill material • Moderate depth to groundwater under portions of site • The depth to groundwater may limit the discharge rate
2	F13-0037	• Town owned parcel • Undeveloped • Parcel approximately 14 acres • Approximately 12.5 acres of parcel mapped as favorable soils • Located within 0.5 miles of the WWTF	• Wetland and private property setbacks will likely limit the discharge area • Moderate depth to groundwater under portions of the site • Located outside of the Sewer District
3	E13-0056	• Parcel approximately 22.5 acres • Approximately 6.5 acres of parcel mapped as favorable soils • Located approximately 0.5 mile from WWTF • Moderate to high elevation over nearby surface waters	• Privately owned • Commercial property • Partially developed • Cemetery located on the southern portion of the property • Uncertain if northern half of parcel area is being used or if there are plans for its use • Located outside of the Sewer District

SUMMARY, WPCA REVIEW, AND RECOMMENDATIONS

The review of available soils data indicate that a majority of the parcels within the area of interest are underlain by till and or bedrock. Other parcels have topography too steep for groundwater discharge. The steep slopes can also be an indication of thin till over bedrock.

A majority of the remaining parcels are low-lying, privately owned, or developed properties, not favorable for a groundwater discharge. Most of these properties are less than an acre in size, and are developed residential sites. Of the 9,077 parcels within the area of interest, only 38 parcels were identified as being a potential groundwater discharge location. The ten highest ranking of these parcels were summarized in Table 6-1. The ranking of these properties was between 13 and 25 out of a potential score of 30.

Prior to proceeding with a site visit or field investigations, the highest ranked parcels were presented to the WPCA for review and comment at the January 2015 WPCA meeting. The top ranked site on South Street was confirmed to be the former Town landfill. The landfill has been capped and according to the WPCA, CT DEEP has forbidden any future excavation on the site so that the cap is not disturbed. This eliminates this South Street site from further consideration as a potential discharge location.

The second highest ranked site is located off of Norrans Ridge Drive. At present, this parcel is the location of Aquarion's Water Company's Beechwood wellfield. The close proximity of the water supply to the potential groundwater discharge of treated WWTF effluent would likely mean that land application of treated wastewater would not be permitted, precluding the site from consideration.

North Street, the third highest ranked site, is part of St. Mary's cemetery. The WPCA did not believe that the property owner would consent to using land intended as a cemetery for land application. For this reason, the site was eliminated from further consideration.

Six of the remaining seven parcels (Bobby's Court, Stonehenge Road, Peaceable Hill Road, Florida Hill Road, and two locations on Ethan Allen Highway) are located between 1.5 and 2.0 miles from the WWTF as shown on Figure 6-2; the outside limit of the area of investigation. The actual length of infrastructure necessary to convey the treated effluent to any of these sites would be greater than two miles as existing roads would be the primary means of installing the conveyance infrastructure. Land application on these sites was therefore determined not to be cost effective and eliminated from consideration.

The remaining parcel, Sawmill Hill Road, was ranked relatively low and could not accommodate discharge of a significant portion of the WWTF's effluent due to the sites limited acreage of favorable soils, and presence of surface waters and wetlands. As there were no other sites where land application was feasible, the Sawmill Hill Road site was also determined not to be cost effective and eliminated from consideration.

Based on the WPCA's review and input, it was decided there were no sites that warrant a field visit or further site investigation. It was further agreed that AECOM would complete the technical memorandum on the evaluation of land application (Technical Memorandum No. 5 included as Appendix J) and conclude that no feasible sites within a reasonable distance of the WWTF were identified. AECOM therefore recommends that the WPCA consider other options for lowering effluent phosphorus levels to meet the CT DEEP proposed effluent total phosphorus limits, as necessary.

CHAPTER SEVEN
WASTEWATER TREATMENT FACILITY CAPACITY EVALUATION

The capacities of the Route 7 WWTF and the South Street WWTF were evaluated under current conditions, design conditions, and increased flow and loading conditions to determine which unit processes are limiting the WWTFs' overall capacities. At each WWTF, both the hydraulic capacity and the pollutant removal capacity were evaluated. At each WWTF an opinion was offered on both its hydraulic capacity and pollutant removal capacity to identify which unit processes were limiting the overall capacity at each WWTF. After these capacity limitations were established, potential modifications to relieve these limitations were then identified with consideration of the current permit limits at both WWTFs and the potential future permit limits at the South Street WWTF. An opinion of the potential to "re-rate" the WWTFs to a higher capacity has been provided as part of the analyses. A summary of the results from this evaluation is provided below for the Route 7 WWTF followed by the South Street WWTF. Technical Memorandum No. 6 – WWTF Capacity Evaluations contained in Appendix D provides the details of this evaluation.

ROUTE 7 WWTF CAPACITY EVALUATION

Background

The Route 7 WWTF was constructed in 1985 to serve the needs of Sewer District 2 that included flows from the Wells-Benrus facility. The Route 7 WWTF provides advanced wastewater treatment using rotating biological contactors (RBCs), has an average daily design flow of 0.120 mgd, and discharges treated wastewater to the Little Pond which in turn discharges to the Norwalk River. Figure 7-1 provides a layout of the existing Route 7 WWTF and Figure 7-2 presents a process flow schematic of the existing Route 7 WWTF.

Route 7 WWTF Hydraulic Capacity

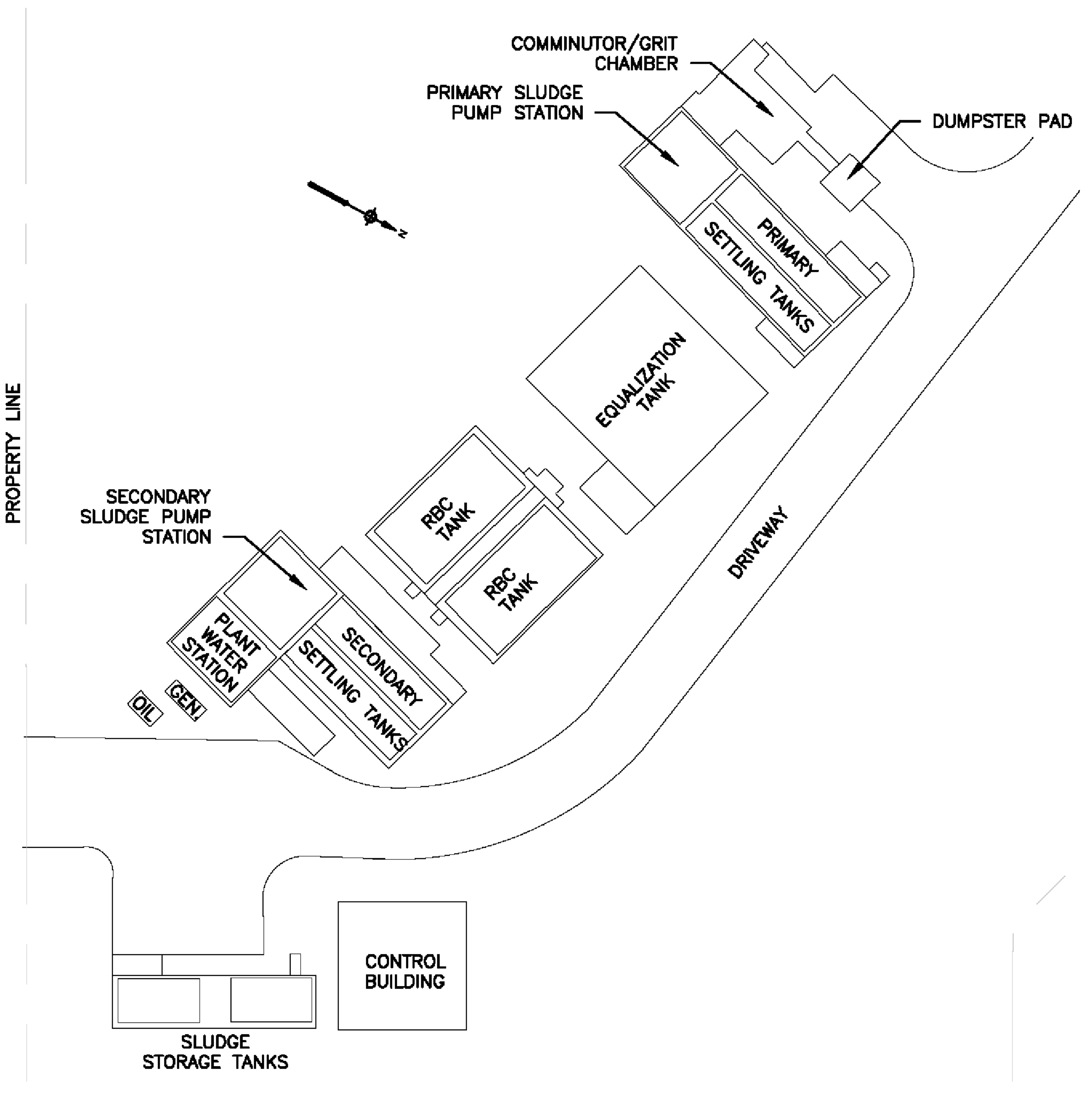
The hydraulic capacity of the Route 7 WWTF was evaluated and an opinion of the current hydraulic capacity of each unit process in the facility has been provided. The hydraulic capacity was evaluated with one of the redundant units out of service for each of the applicable processes (for example one primary settling tank, one RBC, and one final settling tank) based on the requirements of "TR-16 Guides for the Design of Wastewater Treatment Works" published by the New England Interstate Water Pollution Control Commission in 2011.

For the purposes of this evaluation, unit processes or structures are considered to have limited hydraulic capacity if either of the following conditions occurs:

1. There is less than 1 foot of difference (freeboard) between the top of the structure (top of concrete) and the hydraulic grade line (water elevation).
2. There is less than 3 inches between a flow control weir in a structure and the water surface elevation downstream of the weir.

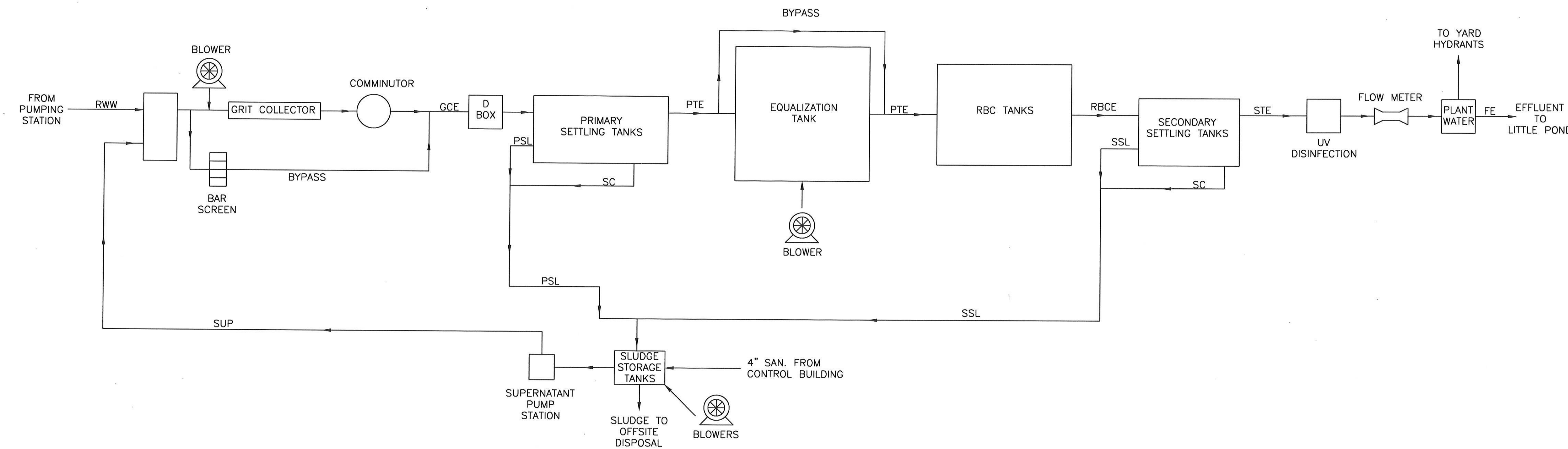
Based on the model runs performed, the hydraulic capacity of each of the unit process was evaluated. The summary of the hydraulic capacity of each of the unit process is summarized in Table 7-1. It should be noted that the hydraulic capacity of the rotating biological contactors and the UV disinfection system are indicated in the table as 0.0 mgd. This is the result of the weirs in both of these unit processes being located less than one foot below the top of the wall or structure therefore limiting the freeboard to less than 1 foot under all conditions. It should be noted that these structures have not been reported to have overtopped in the past and have been able to convey the flows received at the WWTF.

Based on the evaluation there are hydraulic limitations in the RBCs, UV system, and secondary settling tank effluent troughs.



AECOM		FIGURE 7-1 ROUTE 7 WWTF SITE PLAN PHASE 1 WASTEWATER FACILITIES PLAN RIDGEFIELD, CT
Drawn: BC 4/24/2015		
Approved: AA 4/24/2015		

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 7-1 - Route 7 WWTF Site Plan aa.mxd



LEGEND

RWW	RAW WASTEWATER
GCE	GRIT CHAMBER EFFLUENT
FE	FINAL EFFLUENT
PTE	PRIMARY SETTLING TANK EFFLUENT
STE	SECONDARY SETTLING TANK EFFLUENT
SC	SCUM
PSL	PRIMARY SLUDGE
SSL	SECONDARY SLUDGE
RBCE	ROTATING BIOLOGICAL CONTACTOR EFFLUENT
SUP	SUPERNATANT
SAN	SANITARY DRAIN

AECOM

Drawn: BC 4/14/2015

Approved: AA 4/14/2015

FIGURE 7-2

ROUTE 7 WWTF
PROCESS FLOW SCHEMATIC

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

G:\Projects\MUNI\60299267RID\Maps\Facilities Flow Report\Figure 7-2 - Route 7 WWTF Schematic.mxd

TABLE 7-1. ROUTE 7 WWTF UNIT PROCESS HYDRAULIC CAPACITY WITH ONE UNIT OUT OF SERVICE

Treatment Unit	Unit Process Capacity	Comment
Plant Influent Chamber	Between 0.60 mgd and 0.72 mgd	
Grit Chamber	Between 0.60 mgd and 0.72 mgd	
Comminutor	Between 0.50 mgd and 0.60 mgd	
Primary Settling Tanks	Greater than 0.72 mgd	Simulations capped at 0.72 mgd
Primary Settling Tank Effluent Trough	Less than 0.30 mgd	
Equalization Tank	Less than 0.72 mgd	
Rotating Biological Contactors	0.0 mgd [1]	RBC effluent weir is 6 in. below the top of tank
Secondary Settling Tanks	Greater than 0.3 mgd	Simulations capped at 0.3 mgd downstream of EQ
Secondary Settling Tank Effluent Trough	Less than 0.12 mgd	
UV Disinfection	0.0 mgd [1]	UV effluent weir is less than 12 inches below UV channel top
Plant Water Station	Greater than 0.30 mgd	Simulations capped at 0.3 mgd downstream of EQ

1. This structure has not been reported to have overtopped in the past and has been able to convey the flows recorded at the WWTF.

Route 7 WWTF Loading Capacity

Each unit process was examined and the design capacity was reviewed against standards provided in "Guides for the Design of Wastewater Treatment Works (TR-16)" prepared by the New England Interstate Water Pollution Control Commission, 2011 edition and other industry standards including Wastewater Engineering, Treatment and Reuse 4th edition (Metcalf and Eddy). The loading capacities for each unit process were evaluated based on the existing wastewater constituent data (see Chapter 3), the estimated or calculated constituent removal by unit process, unit process design data, and WWTF mass balances. Table 7-2 presents process capacity of each unit process. The secondary settling tanks are the limiting process for average daily flow, and the UV system is the peak flow's limiting process. Also it is not believed that the WWTF will be able to meet their new total phosphorus permit limit without WWTF modifications.

Based on the evaluation the grit chamber does not have enough detention time at the design peak flow of 0.72 mgd. Finally, according to the manufacturer the UV system does not have the capacity to handle the design peak flow of 0.30 mgd.

Opinion to Re-Rate the Route 7 WWTF

Based on results of the hydraulic and loading capacity analysis the potential to "re-rate" the Route 7 WWTF to a higher capacity was evaluated. Based on the evaluation it does not appear the Route 7 WWTF can be re-rated to a higher capacity without modifications.

TABLE 7-2. ROUTE 7 WWTF UNIT PROCESS LOADING CAPACITY

Treatment Unit	Unit Process Capacity [1]	Limitation Comment
Grit Chamber	Peak Hour Flow - 0.58 mgd	Hydraulic Detention Time Limitation
Primary Settling Tanks	Ave Daily Flow - 0.27 mgd Peak Hour Flow – 1.12 mgd	Capacity in Excess of Maximum Influent Conditions
Rotating Biological Contactors	Ave Daily Flow - 0.18 mgd	Capacity in Excess of Maximum Influent Conditions
Secondary Settling Tanks	Ave Daily Flow - 0.16 mgd Peak Hour Flow - 0.32 mgd	Capacity in Excess of Maximum Influent Conditions and attenuated peak flow conditions
UV Disinfection	Peak Hour Flow - 0.20 mgd	Capacity per information from the manufacturer

1. The loading was based on increasing flows at current WWTF influent concentrations.

Options to Remove Route 7 WWTF Hydraulic Capacity Limitations

As noted above there are hydraulic limitations in the RBCs, UV system, and secondary settling tank effluent trough. The following is a summary of potential modification to remove the hydraulic capacity limitations by unit process:

Rotating Biological Contactors (RBCs). In order to provide the desired recommend one foot of freeboard at the 0.30 mgd peak flow, possible modifications include:

1. Lowering of the RBC weir to allow for one foot of freeboard. This alternative would need to be evaluated in more detail, as lowering the water surface in the RBC reactors could have an impact of the treatment performance/capacity of the reactors.
2. Increasing the wall height of the RBC tanks.
3. Discussing the freeboard requirements with the Connecticut DEEP to see if this unit process could be grandfathered from the one foot freeboard guideline.

Ultraviolet (UV) Disinfection. In order to provide the desired recommend one foot of freeboard at the 0.30 mgd peak flow, possible modifications include:

1. Increasing wall height of the UV channel (not recommended due the potential to submerge the UV lamp ballasts).
2. Discussing the freeboard requirements with the Connecticut DEEP to see if this unit process could be grandfathered from the one foot guideline.
3. It should also be noted that when the WWTF is upgraded, the UV system would likely be replaced to provide a system that would provide reliable service for the next 20 years. As a result of that potential replacement, other UV system configurations could be examined at that time. These include a channel UV system with higher channel walls or the use of a pressurized (closed pipe) UV system where freeboard is not an issue.

Secondary Settling Tanks Effluent Troughs. In order to provide the desired 0.25 feet between the settling tank weir and the downstream effluent trough water surface the following modification could be considered:

1. Upsizing the 4 inch discharge pipe on the effluent trough.
2. Adding a second discharge pipe to the effluent trough.
3. Increasing the width of the effluent trough.

Options to Remove Route 7 WWTF Loading Capacity Limitations

As noted above the grit chamber does not have enough detention time at the design peak flow of 0.72 mgd and the UV system does not have the capacity to handle the design peak flow of 0.30 mgd. The following is a summary of potential modification to remove these loading capacity limitations by unit process. In addition, modifications for the WWTF to meet new nutrient permit limits are also summarized.

Grit Chamber. Modifications to increase the grit chamber capacity above 0.58 mgd include:

1. Replacing the grit chamber with one with a larger volume to increase the hydraulic detention time.
2. Another option would be to consider no change in the grit chamber configuration. This should be given consideration due to the limited duration of high flows to the WWTF coupled with the fact that the flow is generated from a small separated collection system with minimal grit. In addition, slightly more grit passing the grit chamber, while undesirable, should have little or no impact on the WWTF effluent and the financial impact associated with the additional wear that might occur in the primary sludge pumps should be significantly less than the installation cost of a new larger grit chamber.

UV Disinfection System. Modifications to increase the UV system capacity above 0.20 mgd include:

1. Replacement of the UV system with a higher capacity system.
2. The addition of a second UV system to operate in parallel or in series with the existing system.
3. Modification of the existing system to increase the number of lamps or modules (need to confirm with manufacturer).

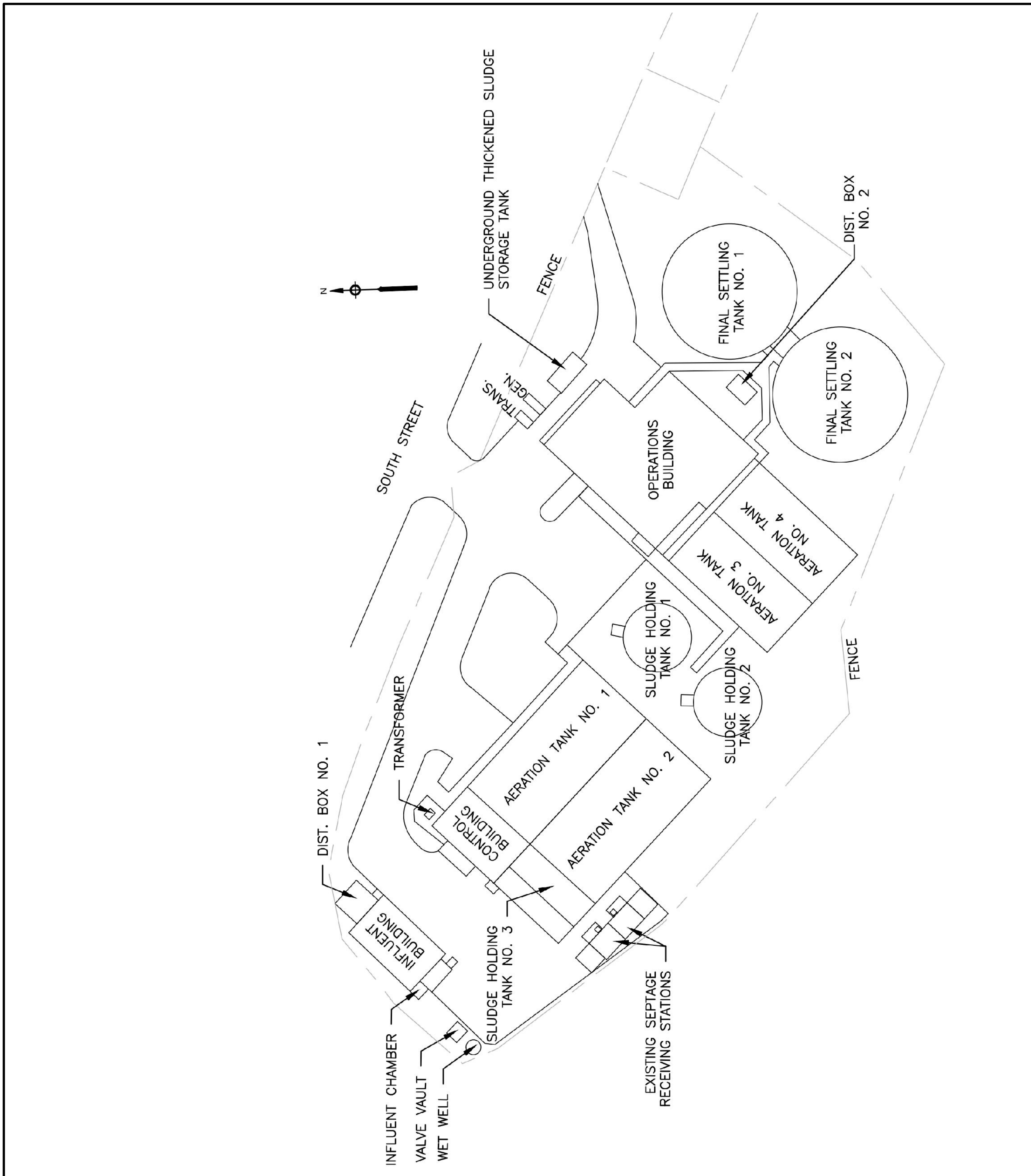
Phosphorus Removal. It is not anticipated that the existing unit processes at the WWTF would be able to meet the new total phosphorus effluent limit of 1.0 mg/l. By comparison the WWTF's current average effluent total phosphorus concentration is 5.3 mg/l. Potential modifications to meet the new total phosphorus limits include:

1. Single or multi-point chemical phosphorus removal (solids removal would occur in the existing settling tanks).
2. Biological phosphorus removal (this would require the construction of an activated sludge process).

SOUTH STREET WWTF CAPACITY EVALUATION

Background

The South Street WWTF which serves the needs of Sewer District 1, discharges its treated wastewater to the Great Swamp. Until the early 1970's, the South Street WWTF consisted of primary treatment followed by sand filtration in open beds. In 1973-74 the WWTF was upgraded to provide extended aeration with a design capacity of 0.72 mgd. The WWTF was subsequently upgraded and expanded in the early 1990's. This upgrade/expansion included the installation of a new influent headworks building, new aeration tanks to provide carbon oxidation as well as nitrification and denitrification, new final settling tanks, continuously backwashing sand filters, post aeration, ultraviolet disinfection, sludge storage, and sludge thickening/dewatering. The 1990's upgrade/expansion design capacity provided an average daily flow of 1.0 mgd and a peak hourly flow of 4.1 mgd. Figure 7-3 provides a layout of the existing South Street WWTF and Figure 7-4 presents a process flow schematic of the existing South Street WWTF.

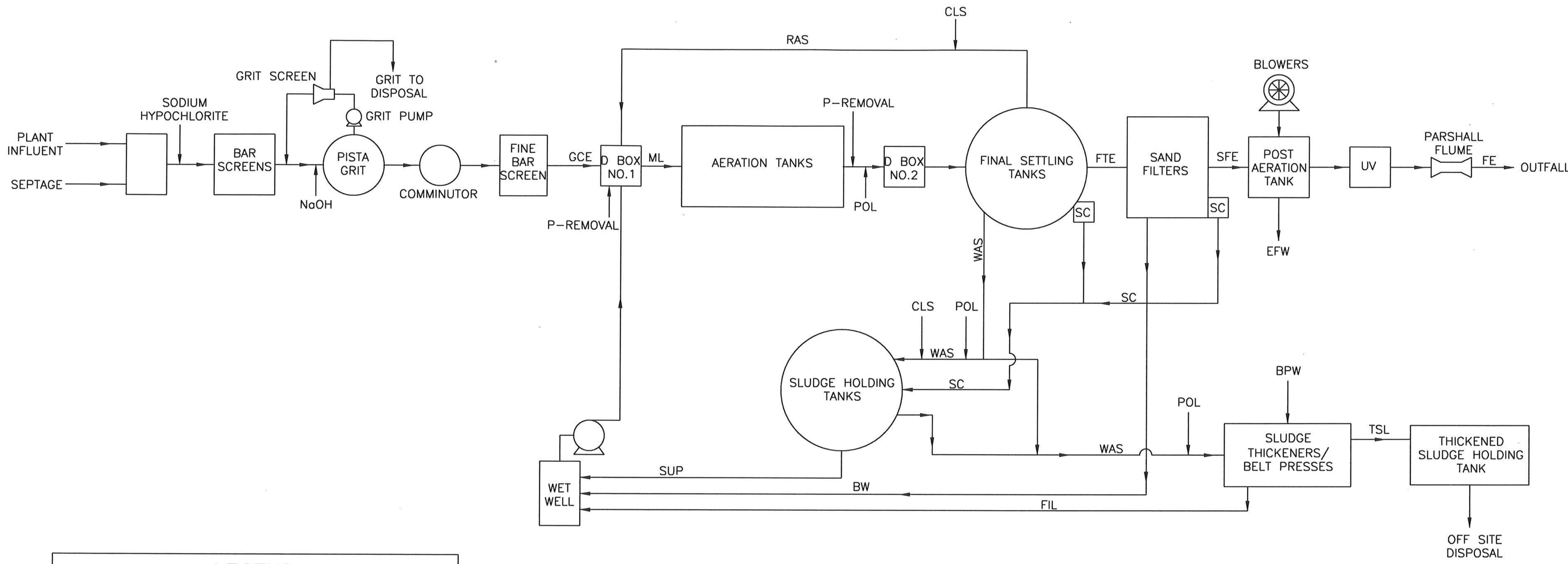


Drawn:	BC	4/24/2015
Approved:	AA	4/24/2015

FIGURE 7-3

SOUTH STREET WWTF
SITE PLAN

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT



LEGEND

FE	FINAL EFFLUENT
BW	BACKWASH
SC	SCUM
FIL	FILTRATE
POL	POLYMER
RAS	RETURN ACTIVATED SLUDGE
WAS	WASTE ACTIVATED SLUDGE
SUP	SUPERNATANT
GCE	GRIT CHAMBER EFFLUENT
ML	MIXED LIQUOR
FTE	FINAL SETTLING TANK EFFLUENT
SFE	SAND FILTER EFFLUENT
BPW	BELT PRESS WASH WATER
EFW	EFFLUENT FLUSHING WATER
CLS	CHLORINE SOLUTION
TSL	THICKENED SLUDGE

AECOM

Drawn: BC 4/14/2015

Approved: AA 4/14/2015

FIGURE 7-4

SOUTH STREET WWTF
PROCESS FLOW SCHEMATIC

PHASE 1 WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

South Street WWTF Hydraulic Capacity

The hydraulic capacity of the South Street WWTF was evaluated and an opinion of the current hydraulic capacity of each unit process in the facility was provided. The hydraulic capacity was evaluated with one of the redundant units out of service for each of the applicable processes (for example the old aeration tanks (No.1 and No. 2), one final settling tank, and one sand filter)) based on the requirements of "TR-16 Guides for the Design of Wastewater Treatment Works" published by the New England Interstate Water Pollution Control Commission in 2011.

For the purposes of this evaluation, unit processes or structures are considered to have limited hydraulic capacity if either of the following conditions occurs:

1. There is less than 1 foot of difference (freeboard) between the top of the structure (top of concrete) and the hydraulic grade line (water elevation).
2. There is less than 3 inches between a flow control weir in a structure and the water surface elevation downstream of the weir.

Based on the model runs performed, the hydraulic capacity of each of the unit process was evaluated. The summary of the hydraulic capacity of each of the unit process is summarized in Table 7-3.

Based on the evaluation there are hydraulic limitations in the plant influent chamber and upstream of the mechanically cleaned screen at peak flows and in the sand filter effluent channel (UV inlet channel) under current average daily flows.

South Street WWTF Loading Capacity

Each unit process was examined and the design capacity was reviewed against standards provided in "Guides for the Design of Wastewater Treatment Works (TR-16)" prepared by the New England Interstate Water Pollution Control Commission, 2011 edition and other industry standards including "Wastewater Engineering, Treatment and Reuse" 4^{th} edition (Metcalf and Eddy). The loading capacities for each unit process were evaluated based on the existing wastewater constituent data (see Chapter 3), the estimated or calculated constituent removal by unit process, unit process design data, WWTF mass balances, and a calibrated wastewater process model (BioWin). Table 7-4 presents the process capacity for each unit process.

Based on the evaluation the grit chamber has a peak hour flow capacity limitation of 4.1 mgd, the aeration tank capacity is limited by the current maximum month conditions under the operating condition of having two of the four aeration tanks online. The aerators under both the two and four aeration tanks online configurations have insufficient capacity at current loading conditions, and the sand filters peak hour capacity is limited to 5.3 mgd. Finally it is believed that the WWTF would not be able to meet the anticipated total phosphorus limit in the pending new WWTF NPDES permit or any new requirements for nitrogen or metals without WWTF modifications.

Opinion to Re-Rate the South Street WWTF

Based on results of the hydraulic and loading capacity analysis the potential to "re-rate" the South Street WWTF to a higher capacity was evaluated. Based on the evaluation it does not appear the South Street WWTF can be re-rated to a higher capacity.

TABLE 7-3. SOUTH STREET WWTF UNIT PROCESS HYDRAULIC CAPACITY WITH ONE UNIT OUT OF SERVICE

Treatment Unit	Unit Process Capacity	Comment
Plant Influent Chamber	Between 4.1 mgd and 4.50 mgd	
Influent Screen	Between 4.1 mgd and 4.50 mgd	
Grit Chamber	Between 6.0 mgd and 7.0 mgd	
Comminutor	Between 6.0 mgd and 7.0 mgd	
Fine Screen	Between 6.0 mgd and 7.0 mgd	
Distribution Box No. 1	Between 5.75 mgd and 6.0 mgd	
Distribution Box No.1 Effluent Chamber	Between 6.0 mgd and 7.0 mgd the TOC [1] Slightly less than 4.1 mgd for the weir	Weir impact not significant for flow control since only one flow path remains to ATs No. 3 and No.4
Aeration Tanks Influent Channel	Greater than 7.0 mgd	
Aeration Tanks	Greater than 7.0 mgd	
Aeration Tanks Effluent Channel	Greater than 7.0 mgd for TOC [1] Slightly less than 7.0 mgd for the weir [2]	Weir impact not significant for flow control since only one flow path remains out of ATs No. 3 and No.4
Distribution Box No. 2	Greater than 7.0 mgd	
Distribution Box No.2 Effluent Chamber	Greater than 7.0 mgd for TOC [1] Slightly less than 4.1 mgd for the weir [2]	Weir impact not significant for flow control since only one flow path remains to one FST
Final Settling Tanks	Greater than 7.0 mgd	
Final Settling Tank Launder	Greater than 7.0 mgd for TOC [1] Slightly less than 4.1 mgd for the weir [2]	At 7.0 mgd there is 0.14 ft between the weir and the downstream water surface
Final Settling Tank Effluent Box	Greater than 7.0 mgd	
Sand Filters	Greater than 7.0 mgd	
Sand Filter Effluent	0.85 mgd	Conservative UV system model parameter indicated less than 3 inches between weir and downstream water surface.
UV	Greater than 7.0 mgd	Manufacturer indicated system can pass 7.0 mgd but is not recommended
UV Effluent	Greater than 7.0 mgd	
Parshall Flume	Greater than 7.0 mgd	

1. TOC - Top of concrete.
2. Limitation not considered significant due to the fact that only one flow path remains.

TABLE 7-4. SOUTH STREET WWTF UNIT PROCESS LOADING CAPACITY

Treatment Unit	Unit Process Capacity	Limitation Comment
Grit Chamber	4.1 mgd	Based on vendor information. Grit capture reduced above 4.1 mgd
Aeration Tanks - Two Tanks in Service	Current Max Month Loading 1.83 mgd	Capacity at or in excess of maximum month conditions. All zones in ATs No.3 and No. 4 run in series
Aeration Tanks - Four Tanks in Service	2.7 – 2.9 mgd at current maximum month loading influent concentrations	All zones in ATs No. 3 and No. 4 run in series and all zones in ATs No. 1 and No. 2 run in series
Aerators - Two Tanks in Service	Insufficient aeration capacity in 1st aerobic zone under current average day conditions	All zones in ATs No. 3 and No. 4 run in series
Aerators - Four Tanks in Service	Insufficient aeration capacity in 1st aerobic zone under current maximum month conditions	All zones in ATs No. 3 and No. 4 run in series and all zones in ATs No. 1 and No. 2 run in series
Finial Settling Tanks	Ave Day - 2.35 mgd Peak Hour - 4.5 mgd	MLSS assumed to be 5,300 mg/l similar to current operation
Sand Filters	Ave Day – 1.5 mgd Peak Hour - 5.3 mgd	Based on vendor loading rates
UV Disinfection	6.2 mgd	Based on vendor information
Solids Handling - Thickening	3 x existing conditions 15% greater than design maximum month conditions	
Solids Handling - Dewatering	4 x existing conditions 45% greater than design maximum month conditions	

Options to Remove South Street WWTF Hydraulic Capacity Limitations

Based on the evaluation and as shown in Table 7-3 there are hydraulic limitations in the plant influent chamber and on the upstream side of the mechanical screen under peak flow conditions and there are hydraulic capacity limitations in the sand filter effluent channel (UV influent channel) under current average daily flows. The following is a summary of potential modifications to remove the hydraulic capacity limitations by unit process:

Plant Influent Chamber and Mechanical Screen. In order to provide the desired recommended one foot of freeboard in the influent chamber and on the upstream side of the mechanical screen at flows greater than 4.50 mgd, the following modifications could be considered:

1. Increasing the wall height of the chamber and on the upstream side of the mechanical screen.
2. Replacement of the downstream comminutor with a channel grinder if one with a lower head can be identified.
3. Replacement of the downstream comminutor and manually cleaned fine screen with a mechanically cleaned fine screen.

Sand Filters. The hydraulic capacity of the sand filters, based on providing three inches (0.25 feet) between the sand filter weirs and the downstream water surface, is less than 0.85 mgd. See Technical Memorandum No. 6, WWTF Capacity Evaluations included as Appendix D, for a more detailed description of this limitation. In order to provide the desired recommend 0.25 feet of freeboard on the downstream side of the weirs at higher flows, the following modifications could be considered:

1. Increasing the weir height of the sand filters. This would require modifications to the sand filter distribution and backwashing system components.
2. Providing a new UV system with reduced system headloss. Options to reduce the UV system headloss include:
    - Use of different UV system level controllers (ex. actuated weir gates).
    - Increased UV channel width.

Unit Process Treatment Loading Capacity Limitation Relief Modifications

Based on the evaluation and as shown in Table 7-4 there are loading capacity limitations in the grit chamber, aeration tanks, aerators, and sand filters. The following is a summary of potential modifications to remove the loading capacity limitations by unit process. In addition, modifications to the WWTF to meet potential future nutrient and metals NPDES permit limits are also summarized below.

Grit Chamber. Modifications to increase the peak hourly flow capacity of the grit chamber above 4.1 mgd include:

1. Replacing the grit chamber removal system with a system with higher capacity.
2. Providing a second grit removal system in series with the existing system to capture additional grit that is not captured in the existing system.
3. Investigate mechanical system modifications with the manufacturer to increase system capacity without structural change (may not be feasible).

Aeration Tanks. Under the current two Aeration Tank operating condition, the evaluation identified the unit process as having a capacity limitation at the current maximum month loading conditions. This limitation is based on the potential loss of nitrification and predicted increase in NO_2-N in the effluent. Under the four Aeration Tank operating condition there was sufficient capacity. Modifications to increase the capacity of aeration tanks include:

1. Operating the unit process in a four Aeration Tank configuration.
2. Modify Aeration Tanks No. 3 and No. 4 to increase the aeration tank biomass without requiring additional final settling tank capacity with a process such as:
    - Integrated Fixed Film Activated Sludge (IFAS) Process.
    - BioMag /BioActiflo Processes (Ballasted activated sludge).
    - Membrane Bioreactor (MBR)
    - Providing separate stage denitrification and run all zones aerobically.

Aerators. The existing aerators currently have insufficient capacity at current loading conditions with two or four aeration tanks on line operating in the current four zones in series configuration. Modifications to increase capacity include:

1. Provide larger surface aerators preferably with VFDs and DO monitoring to control aeration.
2. Provide fine bubble aeration and blowers with VFDs and DO monitoring to control aeration.
3. Provide Invent mixer style mixer/aerators with blowers, VFDs and DO monitoring to control aeration.
4. Providing an internal recycle stream to increase denitrification in the first stage and decrease the oxygen demand to the subsequent stages (other process impacts would need to be evaluated).

Sand Filter. Potential modifications to increase the peak hourly flow capacity of the sand filters above 5.3 mgd are noted below. However, consideration of increasing the capacity of the sand filters needs to be considered in conjunction with anticipated new effluent phosphorus limit which will require a tertiary solids removal system such as dual stage sand filters (see below for an additional discussion on phosphorus removal). Modifications to increase sand filter capacity include:

1. Construction of additional sand filters cells (it is not anticipated that the existing single stage sand filters will be able to achieve the new total phosphorus limits).
2. If the existing system is used in conjunction with another downstream solids separation process, then the filter loading rate could be increased (by either increasing the media size or using the same media size) with the recognition that treatment performance would be slightly reduced. Any loading rate changes should be discussed with the sand filter manufacturer.

Phosphorus Removal. It is not anticipated that the existing unit processes at the WWTF would be able to meet the new total phosphorus effluent limit of 0.06 mg/l (at the current 1.0 mgd average day design flow). By comparison the WWTF's current average effluent total phosphorus concentration is 0.21 mg/l. Potential modifications to meet the new total phosphorus limits include:

1. Biological Phosphorus Removal Options which would include/require:
    - Aeration Tank Modifications (which would impact current process and capacity)
    - Tertiary chemical phosphorus removal and solids separation processes.
    - Potential solids handling process modifications to prevent anaerobic conditions

2. Chemical Phosphorus Removal Options which would include/require:
    - Single or multi-point chemical addition.
    - Use of existing final settling tanks for partial phosphorus removal for multipoint chemical addition (tertiary treatment would still be required).
    - Membrane Bioreactor for secondary treatment with chemical addition (see the MBR option for increasing the aeration tank capacity above).
    - Use of the existing sand filters as noted in the sand filter section above followed by a second set of sand filters (Dynasand D2 Process) or another tertiary solids separation process.
    - Use of a tertiary solids separation process with or without the sand filters including:
        - Disc Filters
        - Membrane Filters
        - Ballasted flocculation systems (Comag, Actiflo)
        - BluePro

It should be noted that any additional chemical added for phosphorus removal will impact the aeration tank capacity. The impact of any additional solids generated from chemical phosphorus removal will need to be evaluated further as part of the Phase 2 Facilities Plan.

Nitrogen Removal. As discussed in Technical memorandum No. 6 (in Appendix D), there is the potential that the South Street WWTF may receive a more stringent total nitrogen limit when the Nitrogen General Permit is reissued. In addition, the portion of the Nitrogen General Limit program that allows for purchasing and selling of nitrogen credits may be modified or discontinued. The original Nitrogen General Permit was issued in 2002. Between 2002 and 2008 the WWTF was able to sell credits as the WWTF effluent nitrogen load was less than the permitted limit. However since 2009, the WWTF has been required to purchase credits as their effluent nitrogen load exceeded the permitted limit. These exceedances were due in part to the fact that the Nitrogen General Permit's required effluent nitrogen load decreased each year between 2002 and 2014 for the WWTF. There have also been indications that the CT DEEP Nitrogen General Limit program that allows for purchasing of nitrogen credits may be modified or discontinued. To improve the WWTFs nitrogen removal performance to meet the current and potential future total nitrogen limits, the following modifications could be considered:

1. The addition of a separate stage denitrification process (would increase the nitrification capacity of the existing aeration tanks but may require the use of supplemental carbon).
2. Modifications to the existing aeration tanks to improve nitrogen removal (which would impact the loading capacity and hydraulic capacity of the aeration tanks) including:
    - Providing internal recycle pumps for the current aeration tank to operate in a Modified Ludzack-Ettinger (MLE) configuration.
    - Modifications to provide a 4 stage Bardenpho process.

- Use of other processes (ex. MBRs, IFAS, etc).

Metals Removal. There is the potential for new or stricter metals limits to be included in the new South Street WWTF NPDES permit. It is anticipated that the South Street WWTF would not be able to meet new or stricter metals limits and as a result WWTF modifications would be required. The alternative to evaluate would be dependent upon the numerical limits and metals included in the permit. This evaluation should be performed in conjunction with the chemical phosphorus removal analysis as many of the technologies to improve phosphorus removal have the potential to increase metals removal. Of particular concern is the effluent limit on zinc in light of the past issues at the South Street WWTF with meeting the monthly average and daily maximum limits in the existing NPDES permit. See Technical Memorandum No.6 in Appendix D for additional information in the zinc limits and regulatory requirements related to zinc.

CHAPTER EIGHT
PROJECTED FUTURE FLOWS AND LOADS

To assess the future wastewater treatment needs for the Town, future wastewater flows and loads have been projected. A range of projected future flows and loads for the two treatment plants resulting from potential infilling and sewer needs areas of the collection systems over the next twenty years was prepared. Technical Memorandum No. 7, Projected Future Flows and Loads, contained in Appendix C, provides the details of this evaluation. A summary of this evaluation is provided below.

BACKGROUND

Projections of future flows and loads are prepared to determine the size of the treatment facilities needed to accommodate anticipated growth in the wastewater collection system over the next 20 years. The South Street (Main) WWTF serves Sewer District 1 and the Route 7 WWTF serves Sewer District 2.

Sewer District 1

The existing South Street WWTF was sized to accommodate flows in Sewer District 1 based on a report entitled "Report on Wastewater Treatment and Sewer System Rehabilitation Needs" prepared by Stearns & Wheler, Inc. dated November 1987. That report projected growth within the existing sewer district as well as identified areas of potential need for extension of sewer service to address health related septic system failures.

The report also identified 3 potential areas outside of the existing sewer district where extensions of the sewer system could be needed to address health or pollution resulting from failures of on-site septic systems. These areas were:

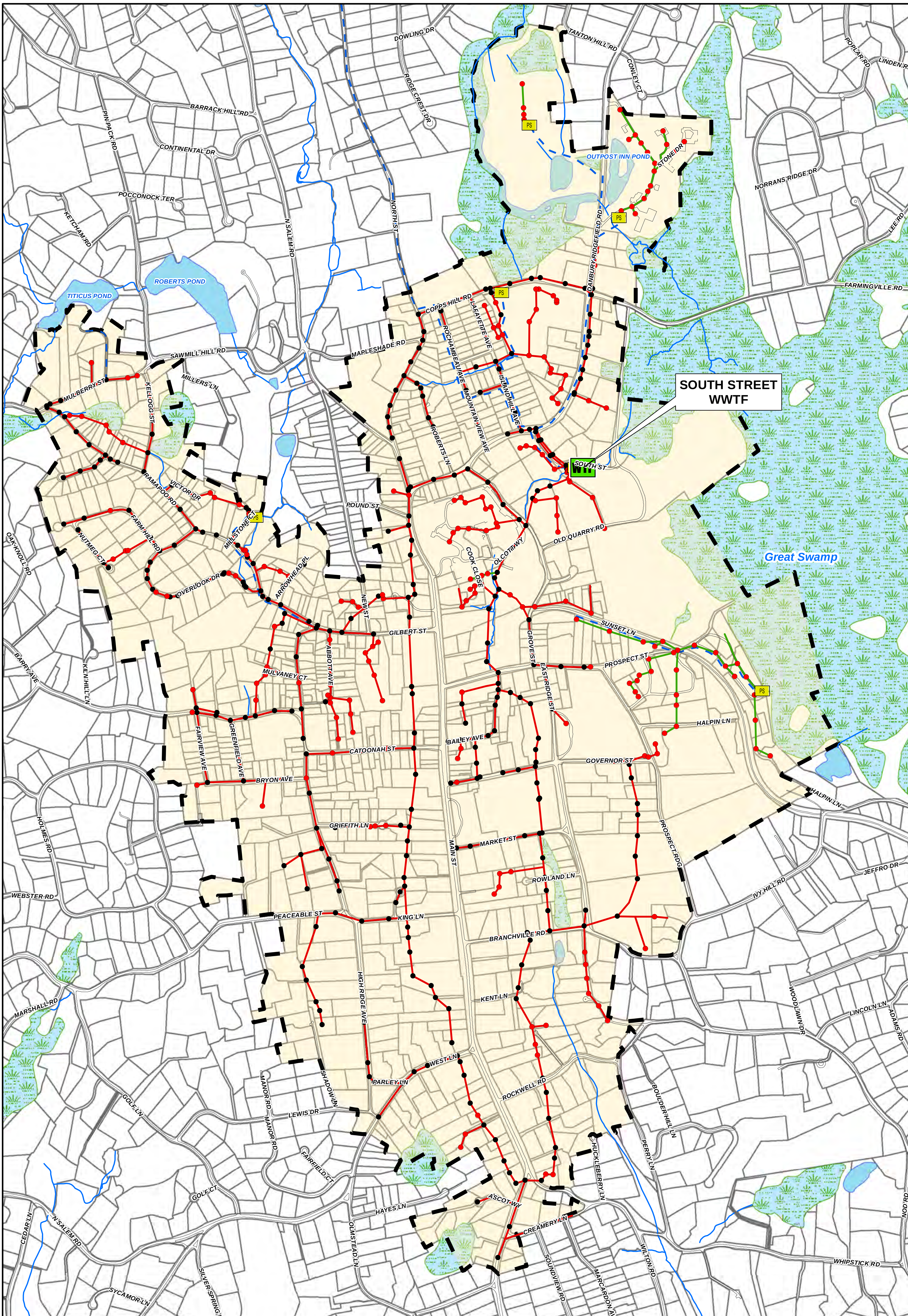
- The Ramapoo Road area
- The Soundview Road/Marcardon Avenue/Creamery Lane area
- The New Street area

Since the report was prepared, sewer service was extended to the Ramapoo Road area in 1999. Sewers have not been constructed to provide sewer service to the other needs areas. Figure 8-1 presents a plan showing the current limits of Sewer District 1.

Sewer District 2

In 1978 the Town was ordered to construct the Route 7 WWTF by the Connecticut DEEP to address documented water pollution problems from specific parcels in the vicinity of the intersection Route 7 and Route 35. To respond to the order, the Town prepared a report entitled "Town of Ridgefield Connecticut, Facilities Plan for Route7/Route 35 Area" prepared by Albertson, Sharp and Ewing dated April 1979 which outlined the sewer service area and projected flows, defined the details of the then proposed sewer collection system, as well as identified the size and treatment process for the proposed Route 7 WWTF.

To fund the construction of the sewer system and WWTF, all of the parcels to be served formed the basis for Sewer District 2, and each parcel was allocated a flow allowance. The owner of each parcel then purchased the allocated flow allowance which represented their share of the plant capacity. The Route 7 WWTF and collection system was then constructed by the Town. Nearly all of the parcels in Sewer District 2 have since connected to the sewer system, although many of the parcels have not been developed at the density of development permitted by current zoning of the District. As a result, all of the current Route 7 WWTF capacity has been allocated to the existing parcels in the district, with no capacity available for extension of the collection system. Figure 8-2 presents a plan of the current limits of Sewer District 2.

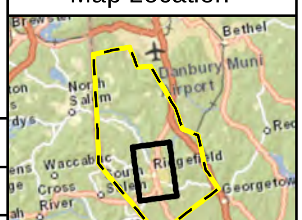


AECOM

Drawn: BC 4/8/2015

Approved: AA 4/8/2015

Map Location



Legend

- WWTF
- Pump Station
- Manhole
- Manhole - Not Field Located
- Municipal Sewer
- Private Sewer
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 500 1,000 2,000 Feet

FIGURE 8-1

EXISTING SEWER DISTRICT 1

PHASE I WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

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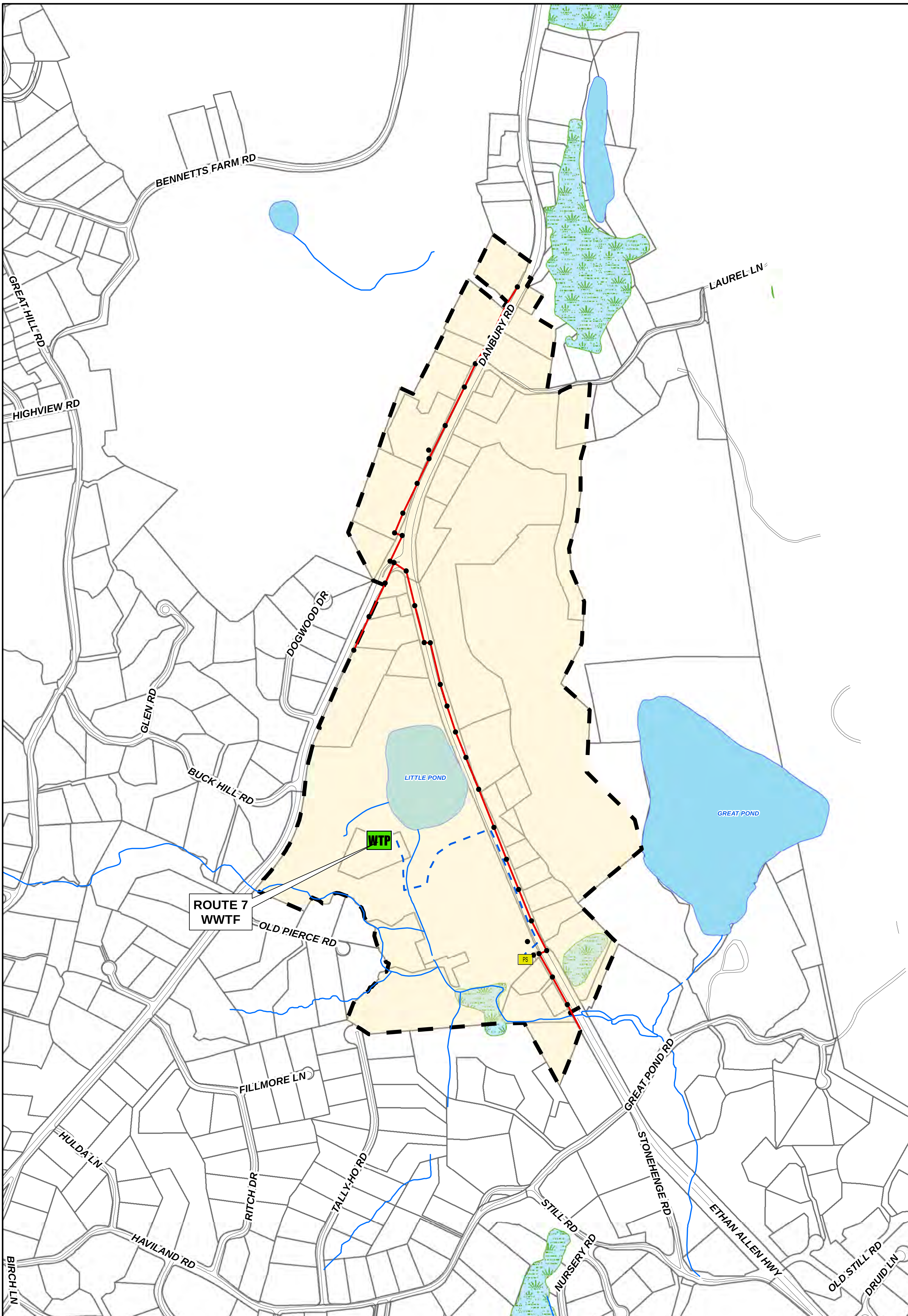

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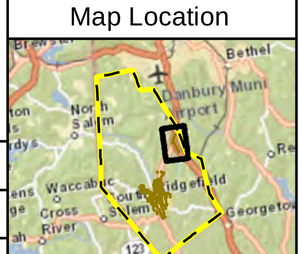
Drawn: BC 4/8/2015

Approved: AA 4/8/2015

Map Location



Legend

- WWTF
- Pump Station
- Manhole
- Municipal Sewer
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 300 600 1,200 Feet

FIGURE 8-2

EXISTING SEWER DISTRICT 2

PHASE I WASTEWATER FACILITIES PLAN
RIDGEFIELD, CT

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DESIGN PARAMETERS

In developing the projected future wastewater flows several parameters were established. These parameters included:

- Period of Design – The year 2035 was selected as the design year for projection of future flows.
- Domestic (Residential) Wastewater Flows – Using Ridgefield water use data, the per capita sewage production was estimated at 61 gallons per day (gpd) and the average household size in Sewer District 1 is approximately 2.7 people per household. These parameters were used for projecting residential flows.
- Non-Residential Commercial Flows – Using Ridgefield water use data, an allowance of 121 gallons/1000 square foot of floor area was used for projecting commercial and industrial flows.
- Infiltration/Inflow (I/I) – An allowance of 200 gallons per acre per day (gpad) has been used for projecting future infiltration in residential and non-residential commercial areas.
- Peaking Factor – Based on Ridgefield flow data, peaking factors of 2.8 and 3.0 have been used for projecting future flows for the South Street and the Route 7 WWTFs, respectively. Peaking factors of 1.81 and 1.79 have been used for projecting future infiltration flows for the South Street and the Route 7 WWTFs, respectively.

INFORMATION COLLECTION

The WPCA provided several sets of information for use in the projection of future flows. That information included:

- Existing Zoning
- Geographic Information System (GIS) Database
- Sewer User Listings
- Septic System Repair/Replacement Data
- Septage Hauler Discharge Data
- Aquarion Water Use Data
- WWTF Operating Data (both South St. and Route 7)

Throughout the development of the projected flows and loads a GIS system was used to spatially review collected data.

DEVELOPMENT OF PROPOSED FUTURE FLOWS – SEWER DISTRICT 1

Projected future flows for Sewer District 1 have been developed in steps as follows. First, flows resulting from new connections to the sewer system in the existing district were estimated. Next, flows resulting from potential redevelopment of existing sewered properties in Sewer District 1 based on current zoning designations were estimated. Lastly, data were reviewed to identify sewer needs areas where extension of the Sewer District 1 collection system to address pollution or health issues with the continued use of on-site septic system may be warranted in the future. Each of these steps are described below.

Infilling

Properties within the current Sewer District 1 boundaries that are not connected to the sewer system were considered “infilling” properties in the flow projections. These infilling properties were considered based on their existing development status as well as their potential for development. That is, properties currently not developed at the density allowed by their zoning classifications were evaluated at their “potential” development. Toward this end, a buildout analysis of Sewer District 1 was conducted by Planimetrics, Inc. of Simsbury, CT. The buildout report is included in Appendix A of Technical Memorandum No. 7 contained in Appendix C of this report. Table 8-1 presents a summary of projected infill flows from both existing and potential development conditions.

TABLE 8-1. SUMMARY OF PROJECTED FLOWS ASSOCIATED WITH INFILLING

Flow Component	Average Daily Flow (gpd)
Sanitary - Existing Development Condition	32,300
Sanitary - Potential Development condition	57,200
Total Sanitary Flow	89,500
Infiltration	22,600
Total Estimated Flow	112,100

Sewer Needs Areas

In addition to flows resulting from infilling within Sewer District 1, the other component of future wastewater flows would be extensions of the collection system to serve areas outside the current sewer district. The WPCA has directed that only sewer extensions to address documented public health or pollution issues from existing development be considered. The assessment of whether or not a property exhibited a need for the extension of municipal sewers was based on several criteria including:

- Consistency with Ridgefield Plan of Conservation and Development
- Consistency with Conservation and Development Policies Plan for Connecticut
- Previous Facilities Planning Data
- Review of Septic System Data
- Review of Septage Data
- Sewer Needs Public Input

Based on the assessment conducted, areas of potential sewer needs were identified. These areas are shown in Figure 8-3 and include the New Street Area and the Marcardon/Soundview Area.

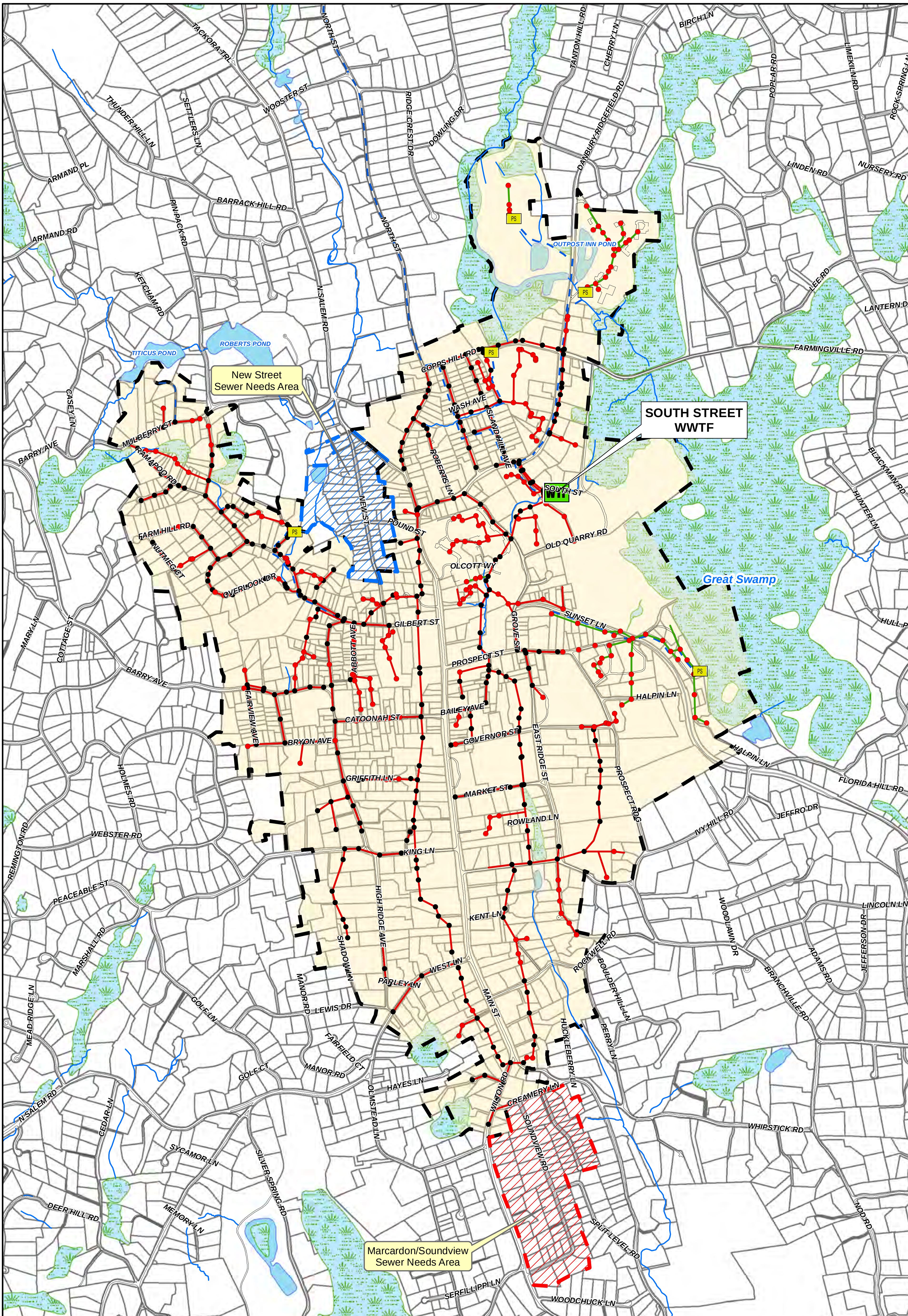
New Street Area. New Street is located to the north of the existing Sewer District 1. The unsewered portion of New Street, between Silver Birch Road and Saw Mill Road, consists of approximately 50 parcels, ranging in size between approximately 0.13 acre and approximately 6.5 acres. New Street is an established residential area. Public water supply is available for the entire length of the street.

Marcardon/Soundview Area. The Marcardon/Soundview area is located to the south of the existing Sewer District 1. It is made up of all of Marcardon Avenue, all of Soundview Road, a portion of Wilton Road West between St. Johns Road and Olmstead Lane, and Creamery Lane. It consists of approximately 76 parcels. It is an established residential area. All of Marcardon Avenue, Creamery Lane and Wilton Road West are served by public water. Only the properties on the northernmost half of Soundview Road have public water service. Lot sizes range from 0.18 to 2.8 acres.

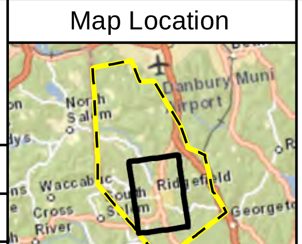
Table 8-2 presents a summary of projected flows from the identified Sewer Needs Areas in Sewer District 1.

TABLE 8-2. SUMMARY OF SEWER DISTRICT 1 PROJECTED FLOWS ASSOCIATED WITH SEWER NEEDS AREAS

Flow Component	Average Daily Flow (gpd)
Sanitary - New Street	9,000
Sanitary - Marcardon/Soundview	12,600
Total Sanitary Flow	21,600
Infiltration	16,700
Total Estimated Flow	38,300



AECOM	Map Location	Legend	FIGURE 8-3
Drawn: BC 4/8/2015			SEWER NEEDS AREAS
Approved: AA 4/8/2015			PHASE I WASTEWATER FACILITIES PLAN RIDGEFIELD, CT



Legend
- WTP WWTF
- PS Pump Station
- Manhole
- Manhole - Not Field Located
- Municipal Sewer
- Private Sewer
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 625 1,250 2,500 Feet

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Summary of Projected Flows – Sewer District 1

The additional projected 2035 average daily flow to the South Street WWTF from infilling and potential sewer needs areas is summarized in Table 8-3.

TABLE 8-3. SOUTH STREET WWTF PROJECTED AVERAGE DAILY FLOW IN GALLONS PER DAY (GPD)

Category	Infilling	Sewer Needs Areas		Total Estimated Average Daily Flow
		New Street Area	Marcardon/Soundview Area	
Residential	36,000	9,000	13,000	58,000
Non Residential	54,000	200	-	54,200
Subtotal				112,200
Infiltration	22,000	6,000	10,000	38,000
Total	112,000	15,200	23,000	150,200

To estimate a total peak flow rate for the year 2035, the projected peak flow from the potential sewer needs areas is added to the current peak flows. As described previously, the peaking factor for the sanitary wastewater component of the flow at the South Street WWTF is 2.8 and the peaking factor of the future infiltration component of the flow is 1.81. Based on projected flows in Table 8-3, the projected additional average sanitary wastewater flow at the South Street WWTF for 2035 is approximately 112,000 gpd with a peak of 314,000 gpd using a peaking factor of 2.8. The projected additional average infiltration flow for 2035 is approximately 39,000 gpd with a peak flow of 71,000 gpd based on a peaking factor of 1.81.

The current sanitary wastewater average daily flow at the South Street WWTF is 0.592 mgd (see Chapter 3). Multiplying this by a peaking factor of 2.8 for yields an existing sanitary wastewater peak flow of approximately 1.66 mgd. The current peak infiltration is approximately 363,000 gpd. The current peak inflow is estimated to be approximately 3,900,000 gpd. Summing the various components of the peak flow, the total year 2035 peak flow is projected to be approximately 6.3 mgd. Table 8-4 presents a summary of the components for the projected average and peak flow at the South Street WWTF.

TABLE 8-4. SOUTH STREET WWTF PROJECTED AVERAGE AND PEAK FLOW

Flow Component	Average Daily Flow (gpd)	Peaking Factor	Peak Flow (gpd)
Current Wastewater	592,000	2.8	1,658,000
Current Infiltration	201,000	1.81	363,000
Current Inflow	57,000	-	3,859,000
Projected Wastewater	111,100	2.8	311,000
Projected Infiltration	39,300	1.81	71,000
Total	1,000,400		6,262,000

DEVELOPMENT OF PROJECTED FUTURE FLOWS – SEWER DISTRICT 2

As noted previously, the capacity of the existing Route 7 WWTF has been fully allocated to the existing parcels that comprise the district, and each parcel owner has purchased their share of the plant capacity. The existing average daily flow at the Route 7 WWTF is approximately 0.054 mgd and the permitted design capacity for the WWTF is 0.12 mgd. There have been no public health or pollution issues identified by the Town from existing development in the area of Sewer District 2.

Since the capacity of the WWTF is fully allocated to the existing parcels in the District, the projected increase in the average daily flow to the Route 7 WWTF would be from the development of undeveloped or underdeveloped parcels within the existing service area. No allowance for sewer extensions to serve parcels outside the existing sewer district has been included. Consequently, the projected future average daily flow for Sewer District 2 is the current plant capacity of 0.12 mgd.

Similarly, the projected future peak flow for the Route 7 WWTF would be the current plant peak flow capacity. The current peak flow is approximately 0.36 mgd. As noted in Chapter 7, both the Route 7 Influent Pump Station and the Route 7 WWTF headworks have a maximum capacity of 0.72 mgd. Consequently, the projected future average peak flow for Sewer District 2 is 0.72 mgd.

PROJECTED FUTURE WASTEWATER LOADS – SEWER DISTRICTS 1 AND 2

The existing average influent pollutant concentrations at both the South Street and Route 7 WWTFs are presented in Chapter 3. Multiplying these concentrations by the projected 2035 average daily flows yields the projected 2035 average daily loads.

The projected average daily loads for the South Street and Route 7 WWTFs are summarized in Table 8-5.

TABLE 8-5. PROJECTED AVERAGE DAILY LOADS

WWTF	Existing Average Influent Concentration (mg/l)				Projected Average Daily Flow (mgd)	Projected Average Daily Load (lbs./day)			
	BOD_5	TSS	TKN	TP		BOD_5	TSS	TKN	TP
South St.	219	232	24.8	4.0	1.00	1,830	1,940	210	35
Route 7	280	226	33.0	6.0	0.12	280	230	33	6.0

CHAPTER NINE
ALTERNATIVES FOR ACCOMODATING FUTURE FLOWS AND LOADS

As noted in Chapter Eight, in the year 2035 sewer extensions and infilling within the existing sewer districts are projected to increase flow to the Town's two WWTFs under average and peak daily flow conditions. The projected year 2035 flows are presented in Table 9-1.

TABLE 9-1. PROJECTED YEAR 2035 AVERAGE AND PEAK FLOWS

Sewer District/WWTF	Year 2035 Projected Average Daily Flow (mgd)	Year 2035 Projected Peak Hour Flow (mgd)
Sewer District 1 - South Street WWTF	1.0	6.26
Sewer District 2- Route 7 WWTF	0.12	0.72

In Chapter Seven, the hydraulic and pollutant loading capacity of each WWTF was assessed and Table 9-2 summarizes the average and peak capacity for each WWTF.

TABLE 9-2. CURRENT WWTF AVERAGE DAY AND PEAK HOUR FLOW CAPACITY

Sewer District/WWTF	Current Average Daily Flow Capacity (mgd)	Current Peak Hour Flow Capacity (mgd)
Sewer District 1 - South Street WWTF	1.0	4.10
Sewer District 2- Route 7 WWTF	0.16	0.60 Upstream of EQ Tank 0.20 Downstream of EQ Tank

For the South Street WWTF, the existing WWTF can accommodate the projected average daily flow, but the projected peak hour flow significantly exceeds the WWTF peak capacity. The limitation on peak capacity is not a single unit process or element, but rather is consistent through a number of the WWTF processes. For the Route 7 WWTF, the existing plant can accommodate the projected average daily flow, but the projected peak hour flow exceeds the current plant capacity. The limitation on peak hour capacity is the UV disinfection system downstream of the equalization tank and the headworks upstream of the equalization tank. Based on this information, the Town will need to undertake actions to accommodate the projected future peak flows as part of addressing the long term wastewater needs for Route 7 WWTF. Since there is little I/I in Sewer District 2 the only viable alternative to address the peak hourly flow is to increase the WWTF capacity. However in Sewer District 1 I/I is a more significant issue. As a result alternatives to accommodate the projected future flows in Sewer District 1 are discussed below.

SEWER DISTRICT 1- SOUTH STREET WWTF

The existing and future flows for Sewer District 1 have a significant wet weather flow component that is larger than the dry weather wastewater flows. The alternatives for accommodating the future flows include implementation of an I/I reduction program, construction of modifications to the plant to increase the peak flow capacity, or construction of flow equalization facilities at the plant. Each of these alternatives is discussed below.

I/I Reduction Program

As noted in Chapter Three, significant I/I is present in the Sewer District 1 wastewater collection system and it is the largest factor influencing the peak flows at the WWTF. As noted in Chapter Eight, of the 6.26 mgd projected future peak flow approximately 0.7 mgd is the estimated peak infiltration and 3.9 mgd is the estimated peak inflow. As part of the preparation of this Phase 1 Facilities Plan, sewers in the entire sewer district were smoke tested and 45 direct and 33 indirect inflow sources which contribute inflow to the system were identified. Of the 45 direct sources identified, five sump pumps were identified, which are typically not found through a smoke testing program. The inflow sources identified through the smoke testing only account for a portion of the total inflow in the collection system. As a result it is clear that there are other sources of inflow, such as sump pumps, deteriorated manholes, and other defects that are contributing to the large peak inflow in the District 1 collection system. While the exact magnitude of the I/I flows contributed by each of these sources cannot be determined currently, these sources significantly contribute to the peak I/I experienced in the collection system. In addition, since manhole inspections have only been completed in one subarea, an effort to locate defective manholes which may contribute I/I in Sewer District 1 is recommended. The Town should continue to implement the ongoing program to locate additional sources of I/I.

The recommended remaining efforts to locate I/I sources for removal include:

1. Completing manhole inspections of all manholes in the Sewer District 1 system to locate leakage and defects.
2. Conducting house to house inspections to locate sump pumps and basement drains connected to the sewer system.
3. Follow-up investigations through dye water testing and dye water flooding of suspect I/I sources identified during the smoke testing program to confirm sources.

These efforts will provide identification of I/I sources that can be removed by subsequent efforts.

One other step that is recommended to further address I/I sources is selected television (TV) inspection. The latest TV inspection program of the sewers in Sewer District 1 by United Water was undertaken from 2005 to 2010. Most of these TV inspections were conducted in the late summer or early fall, when groundwater levels are typically lower than average. There has been previous discussion that some of the observed I/I in the collection system may be entering through some of the unusually long service connections present in Sewer District 1, particularly on both sides of Main Street. Due to topography, there are no sewers in Main Street. All of the Main Street properties are connected to sewers in low lying easements to the east and west of Main Street. To assess the potential for laterals to contribute significant I/I, it is recommended that a representative number of manhole to manhole segments located in low lying areas be inspected. It is recommended that 8-10 segments be TV inspected during the spring high groundwater season to observe leakage from both the mainline sewer and the service laterals. A lateral inspection camera can then be deployed to further investigate the service lateral connections that are observed to be leaking. The extent of potential service lateral inspection varies with the number of bends and condition of the service lateral piping. It would also be valuable to confirm whether the buildings served by apparent leaking laterals do not have a sump pump that could be contributing to the observed clean water flow. If significant leakage is observed, the lateral can either be lined or replaced to eliminate the leakage.

With the sources of I/I identified, a program to reduce I/I by addressing the sources is one approach to accommodating the future flows at the South Street WWTF. Reducing I/I not only will reduce the peak flow in the collection system and WWTF, but will reduce operational costs as the I/I flows are no longer pumped and treated. A secondary benefit of I/I removal is that the efficiency of the biological treatment process will be improved. This I/I reduction will decrease the large flow increases currently experienced at the WWTF during and after storm events. It should be noted that a disadvantage of this alternative to address peak flows is that it is difficult to predict the extent of I/I reduction that will be achieved.

WWTF Modifications to Increase Peak Flow Capacity

Rather than remove the I/I from the District 1 collection system, another alternative is to increase the capacity of the WWTF to treat the peak flow. In Technical Memorandum No. 6, the capacity of the WWTF was assessed both on pollutant loading and hydraulic loading. Based on the evaluation, the WWTF can treat a peak flow of approximately 4.1 mgd. It was concluded that the current limiting factor hydraulically was the peak flow capacity of the headworks. If this restriction were addressed, the final settling tanks and the sand filters would then be the limiting factors at 4.5 and 5.3 mgd respectively.

To significantly increase the plant capacity would require substantial additional facilities. The additional facilities to increase capacity would involve construction of additional units for each major unit process such as additional aeration tanks, final settling tank and sand filters. This approach would have significant capital costs as well as increased operational costs. Increasing the plant capacity to address peak flow may also present operational concerns, since the process is a biological system. It may not be practical to maintain the required biomass during dry weather that would be needed to treat the projected 6.3 mgd capacity.

Since the peak flows occur very infrequently, the additional treatment facilities would not be needed except during periods of high flows caused by elevated groundwater and storm conditions. Additional units for each major process would add to the level of operational complexity of the WWTF with the need to bring the additional facilities online when needed and remove them from service following periods of high flows.

Flow Equalization

A third alternative to accommodate the peak flow, in lieu of removing the I/I, is to construct flow equalization facilities at the WWTF. Providing flow equalization involves constructing a tank or tanks to receive diverted excess peak flows that exceed the WWTF's treatment capacity. The diverted flows are stored temporarily until the peak flows subside. Once there is capacity available at the WWTF the stored flow is returned to the WWTF influent. Typically, flow equalization systems consist of one or more open concrete tanks with floating aerators or diffused air. The stored flow is mixed and aerated to keep the stored wastewater aerobic. Following a storm event, the contents of the equalization tank would be pumped to the plant headworks for subsequent treatment. This is not an uncommon approach, as evidenced locally by the Danbury WWTF which has an equalization tank.

Similar to the alternative of modifying the WWTF to increase capacity, the flow equalization facilities would not normally be in use. The tanks would normally be empty to accept flow when needed. Once the flow equalization tanks are drained, washdown to remove any accumulated sediments and solids is required to prevent odors. The tank filling and decanting are typically automated and monitored.

Recommendations

Based on consideration of the potential alternatives, a phased approach to accommodating the future peak flow is recommended. In the first phase, the Town should continue to undertake the necessary investigations to locate sump pumps and other sources of I/I. This would involve the following:

1. Completion of manhole inspections for the balance of the Sewer District 1 collection system.
2. Conducting house to house inspections for sump pumps.
3. Conducting further investigations consisting of dye water flooding and testing to confirm suspect I/I sources.
4. Conducting TV inspection of selected sewer reaches under high groundwater conditions to assess lateral contributions of I/I.

In the second phase, the identified I/I sources should be addressed to remove the I/I flows from the sewer system. This will involve development and implementation of a program to redirect sump pump discharges out of the sanitary sewer and rehabilitate defective sewers and manholes.

After completion of the first two phases of the I/I reduction effort, the degree of success of the program in reducing I/I should be assessed by re-evaluating the plant flow data in conjunction with precipitation data. If the re-evaluation of the plant flow data indicates that the peak flows are still exceeding the plant capacity, the need for flow equalization at the plant should be investigated. It should be recognized that the I/I investigation and reduction efforts are not a one time event. The nature of sanitary sewer systems is such that as the system ages, deterioration occurs which can allow leakage. Additionally, over time, new sump pumps may be connected to the system.

Lastly, to address the long term needs, an evaluation of the plant as outlined in Chapter Ten should be conducted and the resulting recommendations implemented.

CHAPTER TEN
PHASE 2 FACILITIES PLAN SCOPE OF WORK

The Facilities Plan Scope of Work has been structured to address the planning issues for the Ridgefield collection systems and WWTFs in a two phased approach. As part of the Phase 1 Facilities Plan the following efforts were conducted:

1. The existing flows and loads to the WWTFs were evaluated including an estimate of the wastewater component and infiltration and inflow components (I/I).
2. An in-plant sampling program was conducted to better understand the current influent loading conditions and unit process removal performance at the WWTFs.
3. The sewer system area maps were updated.
4. Smoke testing in Sewer District 1 was conducted to identify sources of inflow.
5. Internal manhole inspections were conducted in a portion of Sewer District 1.
6. The 2003 Route 7 and Quail Ridge Pump Stations Evaluations were updated including an investigation of relocating the Quail Ridge Pump Station.
7. Future flows and loadings for Sewer District 1 and Sewer District 2 for the next 20 years were projected.
8. An evaluation of the collection system bottlenecks in Sewer District 1 was conducted.
9. The capacities of the two WWTFs were evaluated.
10. The feasibility of land applying treated effluent at the South Street WWTF was assessed.

Phase 2 of the facilities planning effort will complete the necessary evaluations to develop a recommended plan to accommodate the future flows and loads, address inflow in Sewer District 1 and peak flows at the South Street WWTF, address the physical needs of the existing facilities, improve energy efficiency of the existing facilities, and to meet projected future effluent limits. Based on the evaluations conducted as part of Phase 1, as summarized in Chapters One through Nine of this report, the following items are recommended to be considered as part of the Phase 2 facilities planning effort.

SOUTH STREET WWTF PEAK FLOW MANAGEMENT

Under current conditions the South Street WWTF experiences high peak flows under wet weather conditions. There are a number of unit processes at the WWTF where the hydraulic and pollutant loading capacities are in excess of these peak flows which would require upgrades to the WWTF. In order to eliminate or minimize the WWTF upgrade requirements to manage these peak flows, alternatives will be evaluated including collection system inflow reduction efforts and peak flow equalization at the WWTF. A description of these evaluations is described below.

Collection System Inflow Reduction Efforts

As part of Phase 2, efforts to evaluate the potential reduction of collection system inflow sources will be conducted. Inflow increases the influent flow to the South Street WWTF with extraneous non-sanitary flows. This reduces the capacity of the WWTF to treat sanitary flows. Inflow generally refers to stormwater that is discharged into the sanitary sewer system. Sources of inflow can be from defective or low lying manholes, sump pumps connected to the sewer system, and stormwater connections to the sewer system (rain leaders, area drains, etc.).

Manhole Inspections. As noted in Chapter Four, manhole inspections were completed only in Subarea 1 of Sewer District 1 as part of the Phase 1 Facilities Plan. These inspections identified a number of defects that contribute both inflow and infiltration to the South Street WWTF. Accordingly, manhole inspections are recommended to be performed in the remaining subareas of Sewer District 1. For each manhole an inspection would be conducted and defects noted on an inspection form. A summary of the manhole inspections will be provided to the Town and United Water along with inspection sheets and recommendations for addressing the identified defects in a Technical Memorandum.

Public Education Program. As discussed in Chapter Four, more than three quarters of the inflow sources in Sewer District 1 identified in the 2013 smoke testing program were located on private property. To address these sources successfully, the Town will need to educate the public on the impacts of inflow on the collection and treatment system, and on the operating costs for the system, to build support for removing these inflow sources. As part of the Phase 2 Facilities Plan, a public education program is recommended.

The purpose of the program is to both educate the public on the impacts of inflow on the Town's infrastructure and the operating costs for wastewater collection and treatment, and to implement the policy that the WPCA will develop regarding the procedure to eliminate and redirect inflow sources located on private property. The public education program is envisioned to include the following elements:

- Sewer bill stuffer notices and flyers
- Newspaper articles
- Notices and information on the Town website
- Press releases
- Interviews with Town staff on government cable TV access channel 24

The public education program should be initiated early in the Phase 2 facilities planning process to build public understanding and support for the need to conduct the house to house inspections for sump pumps, as well as the planned dye water testing program to confirm suspect inflow sources.. The program can then shift focus to communicating the procedure to address private inflow sources once it is developed.

House to House Sump Pump Inspections. In order to identify sump pump connections to the sanitary sewer system, house to house inspections will be performed. An inspection sheet will be developed and completed for each residence inspected. A summary of the house to house inspections will be provided to the Town along with inspection sheets in a Technical Memorandum.

Inflow Identification Follow Up Efforts. In order to investigate suspect stormwater connections to the collection system, dye water testing and flooding will be performed to confirm suspect sources of inflow that were identified during the Phase 1 smoke testing. Dye water testing will be conducted through the injection of water, colored by a non-toxic dye, into a designated storm water structure such as a roof leader, driveway drain or area drain without plugging. Observation of the flows in the downstream sanitary sewer will verify whether the structure is connected to the sanitary sewer. Dye water flooding will be conducted through the injection of a large volume of water, colored by a non-toxic dye, into a designated catch basin or storm sewer section which crosses or is in close proximity to a sanitary sewer. The storm sewer will normally be plugged to simulate surcharged conditions. The findings of the dye and flood tests will be documented in a Technical Memorandum and submitted to the Town and United Water.

Selected Television Inspection. The latest television inspection program by United Water was undertaken from 2005 to 2009 , and most of the TV inspections were conducted in the late summer or early fall, when groundwater levels are typically lower than average. There has been previous discussion that some of the observed I/I in the collection system may be entering through some of the unusually long service connections present in Sewer District 1, particularly on both sides of Main Street. To assess the potential for laterals to contribute significant I/I, it is recommended that a representative number of manhole to manhole segments, for example 8-10 segments, be television inspected during the spring high groundwater season to observe leakage from both the mainline sewer and the service laterals. A lateral inspection camera can then be deployed to further observe leakage within the service lateral connections that may be observe to be leaking. It would also be valuable to confirm whether the buildings served by apparent leaking laterals do not have a sump pump that could be contributing the observed clean water flow. The extent of potential service lateral inspection varies with the number of bends and condition of the service lateral piping. The results of the TV inspection work will be recorded on DVDs and logs of the inspections prepared. The findings and recommendations from the TV inspection effort will be summarized in a Technical Memorandum and submitted to the Town and United Water.

Peak Flow Equalization Evaluation

The use of peak flow equalization at the South Street WWTF will be evaluated as an option to reduce peak flows to be treated at the WWTF. This evaluation will include conceptual development of design requirements of an equalization tank or tanks at the WWTF. This includes conceptual sizing, site location, equipment needs identification and estimated costs for implementation. A summary of the equalization concept layouts and cost will be summarized in a Technical Memorandum and submitted to the Town and United Water.

WWTF CONDITION ASSESSMENT

The South Street WWTF was last upgraded in the early 1990s while the Route 7 WWTF has had only minor upgrades since it was originally constructed in the mid 1980s. As a result a significant amount of equipment at both WWTFs has reached or has exceeded its anticipated service life. Also, some of the facility equipment is in poor condition due to the difficult service conditions (operating in corrosive environments, processing abrasive materials, etc.). As a result of the age and condition of the equipment, reliability has and will become more of an issue as the equipment continues to age. This will result in the need for additional maintenance which will become more problematic as it will be more difficult to find replacement parts. It is anticipated that a large portion of the equipment at the two WWTFs will not be able to provide reliable service for the next 20 years.

The physical condition of the existing plant structures, equipment, and processes will be assessed so that recommendations for upgrades necessary to provide reliable wastewater treatment throughout the design period can be developed. An assessment of condition and expected remaining life of all of the structures and equipment at the plant will be prepared. United Water's plant staff's experience with the existing facilities has identified some areas where improvements in performance will be considered. In addition, since the last upgrades, improvements in available process equipment which provide more efficient operation may be available for some processes and will be considered. Based on the characteristics of the existing facilities and on discussions with United Water staff, the following areas have been identified for assessment:

South Street WWTF

1. The influent distribution box often collects material/debris that makes it difficult to convey high flows to the Headworks Building. Increasing the size/configuration of this box will be considered to alleviate this conveyance limitation.

2. The septage receiving tanks are deteriorating and need to be addressed structurally. In addition, Improvements to the septage tank configuration to reduce the collection of solids in only one of the two tanks will be considered.

3. The influent screening configuration allows a significant amount of material to pass to the downstream processes which results in the ragging of the return activated sludge pumps. This ragging has resulted in the need for the operators to disassemble and de-rag the pumps during high flow conditions up to five times per day. Improving the influent screening (finer mechanically cleaned screens) as well as providing chopper pumps to reduce the ragging issue will be considered.

4. The grit removal system equipment is old and in need of replacement.

5. The aeration tank dissolved oxygen (DO) is currently measured by hand and there is limited process control of the aerators. Improvement to be considered will include providing in tank DO monitoring, replacement of the surface aerators with new surface aerators or other types of aeration technologies (ex. fine bubble), and aeration supply control (VFDs). These improvements would help to improve process performance and reduce energy costs at the WWTF.

6. The original aeration tanks are in an unusable condition. Consideration to upgrading these tanks to make then operable will be considered.

7. The final settling tanks sludge collection equipment is reaching the end of their service life and will be evaluated for replacement.

8. The polymer system and aluminum sulfate system are old and are in need of replacement.

9. Sand filters need to be rehabilitated, specifically the air lift pumps. Consideration will be given to looking at other solids removal alternatives.

10. The ultraviolet disinfection system is 20 years old, has no means of automatic cleaning, and has a single power supply and single channel which has no redundancy. Its replacement will be considered.

11. The solids processing at the WWTF is often a 5 day per week operation that sometimes cannot keep up with the generated solids. Upgrades to the belt filter press/thickener and the solids pumping systems to handle thicker solids will be considered. The addition of a second redundant belt filter press/thickener to increase solids throughput and system reliability will also be considered.

12. The standby power generator is not large enough to power any of the aeration systems. Consideration will be given to providing a generator which can handle some of the aeration equipment which would help maintain the biological process during utility power loss.

13. The WWTFs electrical systems are in need of an upgrade to provide reliable service including the implementation of surge protection for protection of sensitive electrical and instrumentation components.

14. The phone system has not been functional for many years. A replacement is needed and will be considered.

15. Operations and administration spaces are in need of various improvements including roof replacements, HVAC upgrades, storage improvements, vehicle storage and maintenance space improvements, and communication improvements. Consideration will be given to relocating the plant administrative office to a location that is handicapped accessible.

16. Plant control and alarm systems are outdated. A SCADA system that incorporates the WWTFs and the remote pump stations will be considered. In addition improvements to the WWTF security will be considered (automatic gate, building intrusion alarms, etc.)

17. Consideration will be given to providing odor control/mitigation measures for open tanks and areas with potential odors (ex. Distribution Box No. 1, septage receiving tanks, etc.).

Route 7 WWTF

The Route 7 WWTF has had only minor upgrades since it was originally constructed in the mid 1980s. All of the mechanical and electrical systems with the exception of the emergency generator and RBCs are well past their service life and are in need of replacement and upgrade. These systems will be assessed as part of the Phase 2 efforts as well as the condition of the structural elements at the WWTF.

ROUTE 7 WWTF DECOMMISSIONING EVALUATION

Due to the size, age and condition of the Route 7 WWTF, consideration should be given to decommissioning the Route 7 WWTF and upgrading the Route 7 Pump Station to pump flow from Sewer District 2 to the South Street WWTF for treatment. An evaluation will be performed to compare the costs (capital and life cycle costs) to upgrade the Route 7 WWTF versus conveying this flow to the South Street WWTF and the additional upgrade requirements at the South Street WWTF to accommodate these flows. To minimize the impact of conveying/treating the Sewer District 2 peak flows to/at the South Street WWTF, consideration should be given to providing a peak flow storage facility at the Route 7 Pump Station site.

FUTURE EFFLUENT LIMITS

As noted in Chapter Two, in October 2014 DEEP issued a new NPDES permit for the Route 7 WWTF that includes an effluent phosphorus limit that the existing treatment facility cannot meet without modifications. The new permit for the Route 7 WWTF also includes a change in the indicator organism used to monitor disinfection performance from fecal coliform to Escherichia coli. It is anticipated that the South Street WWTF permit, once issued, will also contain a more stringent limit on effluent phosphorus, a change in the indicator organism used to monitor disinfection performance from fecal coliform to Escherichia coli, as well as a compliance schedule to meet the new limits. In addition, DEEP has noted in the past that there is the potential for new metals limits to be imposed on the South Street WWTF. It is anticipated that the existing South Street WWTF will not be able to meet its future permit limits without some modifications.

In addition, the DEEP Nitrogen General Permit that imposes limits on effluent total nitrogen from the South Street WWTF expires at the end of 2015. DEEP has indicated that the permit will be re-issued, and is considering modifications to the permit requirements as part of the renewal of the permit. As noted in Chapter 7, the original Nitrogen General Permit allowed for purchasing and selling of nitrogen credits as one approach to meeting the effluent total nitrogen limit. Between 2002 and 2008 the WWTF was able to sell credits as the WWTF effluent nitrogen load was less than the permitted limit. However since 2009, the WWTF has been required to purchase credits since the effluent nitrogen load exceeded their permitted limit. As part of the Phase 2 Facilities Plan an evaluation of alternatives to improve nitrogen removal at the South Street WWTF will be performed based on direction from the DEEP on the anticipated changes to the Nitrogen General Permit. The Route 7 WWTF is currently exempt from the nitrogen limits under the existing Nitrogen General Permit.

With the definition of the projected future flow, discussions with the DEEP should be conducted as an initial step in the Phase 2 Facilities Plan to identify any other changes in effluent limits that may be imposed on the South Street WWTF or the Route 7 WWTF including if the Sewer District 2 flows were conveyed there for treatment. If the Route 7 WWTF was eliminated and the South Street WWTF capacity was increased to 1.12 mgd, this may trigger anti-degradation concerns by the DEEP. This could in turn result in more stringent effluent limits on parameters to maintain the same effluent mass loading to the Great Swamp if the two WWTFs are combined. This will need to be considered in consultation with the DEEP. Discussions with DEEP on all of the permit requirements will allow any needed process changes or upgrades to meet those limits to be identified and incorporated into the planning process. Alternatives to address the new permit limits will be identified and evaluated.

In addition consideration should be given to performing pilot testing at the South Street WWTF to assess phosphorus removal technologies and their impact on improving zinc removal. These pilot tests could be performed at the end of the Phase 2 Facilities Plan in advance of preliminary design. The need for pilot testing will be dependent upon input from DEEP, the potential for collection system inflow reduction, and the recommendation to keep or decommission the Route 7 WWTF.

ENERGY USE

Due to the age of the existing equipment at the WWTFs it is believed that improvements could be made at the WWTFs to improve energy usage. Some areas that have the potential to improve energy use at the WWTFs include the use of premium efficiency motors, VFDs, high efficiency heating and cooling equipment, solar panels, plant effluent for building heating through the use of heat pump systems, and high speed turbo blowers. When United Water first began operating the WWTFs, an independent Energy Audit was conducted by Process Energy Services that identified a number of energy savings measures. Measures should be reviewed as part of the Facilities Plan and those that are cost effective should be implemented.

In addition an updated energy review by Process Energy Services will be performed on the upgrade alternative evaluated to check that the upgrades are being performed with consideration to the latest in good energy practice. The potential for energy rebates from the power company for implementing the energy efficiency upgrades will also reviewed.

RECOMMENDED PLAN

Following the assessment of the physical condition of the WWTFs structures and equipment, and the investigation of future effluent limits that may be imposed on the facilities, a recommended plan to address the physical needs of the WWTFs, accommodate the future flows and loads, and meet the future effluent limits, will be developed including whether to decommission the Route 7 WWTF. The recommended plan would include estimated project costs and a projected schedule for implementation of the recommendations.

Respectfully Submitted,

AECOM, INC.

Approved:

Jon R. Pearson
Project Manager
Registered Professional Engineer
Connecticut License No. 16082

Donald J. Chelton
Vice President
Registered Professional Engineer
Connecticut License No. 15069

APPENDIX A:

SOUTH STREET WWTF NPDES PERMIT

Tom Gagnon

STATE OF CONNECTICUT
DEPARTMENT OF ENVIRONMENTAL PROTECTION

MUNICIPAL NPDES PERMIT

RECEIVED

issued to

OCT 04 2004

CT STATE PROGRAM UNIT

Permittee:
Town of Ridgefield, Town Hall
400 Main Street
Ridgefield, Connecticut 06877

Location Address:
Town of Ridgefield WPCF (Main)
22 South Street
Ridgefield, Connecticut 06877

Facility ID: 118-001 **Permit ID:** CT0100854 **Permit Expires:** September 29, 2009

Receiving Stream: Great Swamp **Design Flow Rate:** 1.0 MGD

SECTION 1: GENERAL PROVISIONS

(A) This permit is issued in accordance with Section 22a-430 of Chapter 446k, Connecticut General Statutes ("CGS"), and Regulations of Connecticut State Agencies ("RCSA") adopted thereunder, as amended, and Section 402(b) of the Clean Water Act, as amended, 33 USC 1251, et. seq., and pursuant to an approval dated September 26, 1973, by the Administrator of the United States Environmental Protection Agency for the State of Connecticut to administer a N.P.D.E.S. permit program.

(B) Town of Ridgefield, ("permittee"), shall comply with all conditions of this permit including the following sections of the RCSA which have been adopted pursuant to Section 22a-430 of the CGS and are hereby incorporated into this permit. **Your attention is especially drawn to the notification requirements of subsection (i)(2), (i)(3), (j)(1), (j)(6), (j)(8), (j)(9)(C), (j)(10)(C), (j)(11)(C), (D), (E), and (F), (k)(3) and (4) and (l)(2) of Section 22a-430-3.** To the extent this permit imposes conditions more stringent than those found in the regulations, this permit shall apply.

Section 22a-430-3 General Conditions

- **(a)** Definitions
- **(b)** General
- **(c)** Inspection and Entry
- **(d)** Effect of a Permit
- **(e)** Duty to Comply
- **(f)** Proper Operation and Maintenance
- **(g)** Sludge Disposal
- **(h)** Duty to Mitigate
- **(i)** Facility Modifications; Notification
- **(j)** Monitoring, Records and Reporting Requirements
- **(k)** Bypass
- **(l)** Conditions Applicable to POTWs
- **(m)** Effluent Limitation Violations
- **(n)** Enforcement
- **(o)** Resource Conservation
- **(p)** Spill Prevention and Control
- **(q)** Instrumentation, Alarms, Flow Recorders
- **(r)** Equalization

Section 22a-430-4 Procedures and Criteria

- **(a)** Duty to Apply
- **(b)** Duty to Reapply
- **(c)** Application Requirements
- **(d)** Preliminary Review
- **(e)** Tentative Determination
- **(f)** Draft Permits, Fact Sheets

(g) Public Notice, Notice of Hearing
(h) Public Comments
(i) Final Determination
(j) Public Hearings
(k) Submission of Plans and Specifications. Approval.
(l) Establishing Effluent Limitations and Conditions
(m) Case-by-Case Determinations
(n) Permit Issuance or Renewal
(o) Permit or Application Transfer
(p) Permit Revocation, Denial or Modification
(q) Variances
(r) Secondary Treatment Requirements
(s) Treatment Requirements
(t) Discharges to POTWs - Prohibitions

(C) Violations of any of the terms, conditions, or limitations contained in this permit may subject the permittee to enforcement action including, but not limited to, seeking penalties, injunctions and/or forfeitures pursuant to applicable sections of the CGS and RCSA.

(D) Any false statement in any information submitted pursuant to this Section of the permit may be punishable as a criminal offense under Section 22a-438 or 22a-131a of the CGS or in accordance with Section 22a-6, under Section 53a-157b of the CGS.

(E) The permittee shall comply with Section 22a-416-1 through Section 22a-416-10 of the RCSA concerning operator certification.

(F) No provision of this permit and no action or inaction by the Commissioner shall be construed to constitute an assurance by the Commissioner that the actions taken by the permittee pursuant to this permit will result in compliance or prevent or abate pollution.

(G) Nothing in this permit shall relieve the permittee of other obligations under applicable federal, state and local law.

(H) An annual fee shall be paid for each year this permit is in effect as set forth in Section 22a-430-7 of the RCSA. As of August 20, 2003 the annual fee is $ 2,242.50.

SECTION 2: DEFINITIONS

(A) The definitions of the terms used in this permit shall be the same as the definitions contained in Section 22a-423 of the CGS and Section 22a-430-3(a) and 22a-430-6 of the RCSA, except for "Composite", "No Observable Acute Effect Level (NOAEL) " and "Grab Sample Average" which are redefined below.

(B) In addition to the above, the following definitions shall apply to this permit:

"-----" in the limits column on the monitoring tables in Attachment 1 means a limit is not specified but a value must be reported on the DMR, MOR, NAR, and/or the ATMR.

"Annual" in the context of any sampling frequency, shall mean the sample must be collected in the month of June.

"Average Monthly Limit" means the maximum allowable "Average Monthly Concentration" as defined in Section 22a-430-3(a) of the RCSA when expressed as a concentration (e.g. mg/l); otherwise, it means "Average Monthly Discharge Limitation" as defined in Section 22a-430-3(a) of the RCSA.

"Bi-Weekly" in the context of any sampling frequency, shall mean once every two weeks.

"Critical Test Concentration" or **"(CTC)"** means the specified effluent dilution at which the permittee is to conduct a single-concentration Aquatic Toxicity Test.

"Daily Composite" or **"(DC)"** means a composite sample taken over a full operating day consisting of grab samples collected at equal intervals of no more than sixty (60) minutes and combined proportionally to flow; or, a composite sample continuously collected over a full operating day proportionally to flow.

"Daily Concentration" means the concentration of a substance as measured in a daily composite sample, or, arithmetic average of all grab sample results defining a grab sample average.

"Daily Quantity" means the quantity of waste discharged during an operating day.

"Geometric Mean" is the "n"th root of the product of "n" observations.

"Infiltration" means water other than wastewater that enters a sewer system (including sewer system and foundation drains) from the ground through such means as defective pipes, pipe joints, connections, or manholes. Infiltration does not include, and is distinguished from, inflow.

"Inflow" means water other than wastewater that enters a sewer system (including sewer service connections) from sources such as, but not limited to, roof leaders, cellar drains, yard drains, area drains, drains from springs and swampy areas, cross connections between storm sewers and sanitary sewers, catch basins, cooling towers, storm waters, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration.

"In-stream Waste Concentration" or **"(IWC)"** means the concentration of a discharge in the receiving water after mixing has occurred in the allocated zone of influence.

"Maximum Daily Limit" means the maximum allowable "Daily Concentration" (defined above) when expressed as a concentration (e.g. mg/l), otherwise, it means the maximum allowable "Daily Quantity" as defined above, unless it is expressed as a flow quantity. If expressed as a flow quantity it means "Maximum Daily Flow" as defined in Section 22a-430-3(a) of the RCSA.

"Maximum Daily Limit" means the maximum allowable "Daily Concentration" (defined above) when expressed as a concentration (e.g. mg/l), otherwise, it means the maximum allowable "Daily Quantity" as defined above, unless it is expressed as a flow quantity. If expressed as a flow quantity it means "Maximum Daily Flow" as defined in Section 22a-430-3(a) of the RCSA.

"Monthly Minimum Removal Efficiency" means the minimum reduction in the pollutant parameter specified when the effluent average monthly concentration for that parameter is compared to the influent average monthly concentration.

"NA" as a Monitoring Table abbreviation means "not applicable".

"NR" as a Monitoring Table abbreviation means "not required".

"No Observable Acute Effect Level" or **"(NOAEL)"** means any concentration equal to or less than the critical test concentration in a single concentration (pass/fail) toxicity test, conducted pursuant to Section 22a-430-3(j)(7)(A)(i) of the RCSA, demonstrating greater than 50% survival of test organisms in 100% (undiluted) effluent and 90% or greater survival of test organisms at the CTC.

"Quarterly" in the context of any sampling frequency, shall mean sampling is required in the months of March, June, September, and December.

"MGD" means million gallons per day.

"Sanitary Sewage" means wastewaters from residential, commercial and industrial sources introduced by direct connection to the sewerage collection system tributary to the treatment works including non-excessive inflow/infiltration sources.

"Semi-Annual" in the context of any sampling frequency, shall mean the sample must be collected in the months of June and December.

"ug/l" means micrograms per liter

"Work Day" in the context of a sampling frequency means, Monday through Friday excluding holidays.

SECTION 3: COMMISSIONER'S DECISION

(A) The Commissioner of Environmental Protection ("Commissioner") has issued a final decision and found continuance of the existing system to treat the discharge will protect the waters of the state from pollution. The Commissioner's decision is based on application #200400717 for permit reissuance received on March 5, 2004 and the administrative record established in the processing of that application.

(B) The Commissioner hereby authorizes the Permittee to discharge in accordance with the provisions of this permit, the above referenced application, and all approvals issued by the Commissioner or his authorized agent for the discharges and/or activities authorized by, or associated with, this permit.

(C) The Commissioner reserves the right to make appropriate revisions to the permit, if required after Public Notice, in order to establish any appropriate effluent limitations, schedules of compliance, or other provisions which may be authorized under the Federal Clean Water Act or the CGS or regulations adopted thereunder, as amended. The permit as modified or renewed under this paragraph may also contain any other requirements of the Federal Clean Water Act or CGS or regulations adopted thereunder which are then applicable.

SECTION 4: GENERAL LIMITATIONS AND OTHER CONDITIONS

(A) The Permittee shall not accept any new sources of non-domestic wastewater conveyed to its POTW through its sanitary sewerage system or by any means other than its sanitary sewage system unless the generator of such wastewater; (a) is authorized by a permit issued by the Commissioner under Section 22a-430 CGS (individual permit), or, (b) is authorized under Section 22a-430b (general permit), or, (c) has been issued an emergency or temporary authorization by the Commissioner under Section 22a-6k. All such non-domestic wastewaters shall be processed by the POTW via receiving facilities at a location and in a manner prescribed by the permittee which are designed to contain and control any unplanned releases.

(B) No new discharge of domestic sewage from a single source to the POTW in excess of 50,000 gallons per day may be authorized by the permittee until the discharger has registered the discharge under the "General Permit for Domestic Sewage" reissued by the Commissioner on June 12, 2002 pursuant to Section 22a-430b of the CGS.

(C) The permittee shall maintain a system of user charges based on actual use sufficient to operate and maintain the POTW (including the collection system) and replace critical components.

(D) The permittee shall maintain a sewer use ordinance that is consistent with the Model Sewer Ordinance for Connecticut Municipalities prepared by the Department of Environmental Protection. The Commissioner of Environmental Protection alone may authorize certain discharges which may not conform to the Model Sewer Ordinance.

(E) No discharge shall contain, or cause in the receiving stream, a visible oil sheen or floating solids; or cause visible discoloration or foaming in the receiving stream.

(F) No discharge shall cause acute or chronic toxicity in the receiving water body beyond any Zone Of Influence (ZOI) specifically allocated to that discharge in this permit.

(G) The permittee shall maintain an alternate power source adequate to provide full operation of all pump stations in the sewerage collection system and to provide a minimum of primary treatment and disinfection at the water pollution control facility to insure that no discharge of untreated wastewater will occur during a failure of a primary power source.

(H) The average monthly effluent concentration shall not exceed 15% of the average monthly influent concentration for BOD_5, and Total Suspended Solids, for all daily composite samples taken in any calendar month.

(I) Any new or increased amount of sanitary sewage discharge to the sewer system is prohibited where it will cause a dry weather overflow or exacerbate an existing dry weather overflow.

(J) Sludge Conditions

(1) The permittee shall comply with all existing federal and state laws and regulations that apply to sewage sludge use and disposal practices, including but not limited to 40 CFR Part 503.

(2) If an applicable management practice or numerical limitation for pollutants in sewage sludge more stringent than existing federal and state regulations is promulgated under Section 405(d) of the Clean Water Act (CWA), this permit shall be modified or revoked and reissued to conform to the promulgated regulations.

(3) The permittee shall give prior notice to the Commissioner of any change(s) planned in the permittees' sludge use or disposal practice. A change in the permittees' sludge use or disposal practice may be a cause for modification of the permit.

(K) The limits imposed on the discharges listed in this permit take effect on the issuance date of this permit, hence any sample taken after this date which, upon analysis, shows an exceedence of permit limits will be considered non-compliance.

(L) When the arithmetic mean of the average daily flow from the POTW for the previous 180 days exceeds 90% of the design flow rate, the permittee shall develop and submit for the review of the Commissioner within one year, a plan to accommodate future increases in flow to the plant. This plan shall include a schedule for completing any recommended improvements and a plan for financing the improvements.

(M) When the arithmetic mean of the average daily BOD_5, or TSS loading into the POTW for the previous 180 days exceeds 90% of the design load rate, the permittee shall develop and submit for the review of the Commissioner within one year, a plan to accommodate future increases in load to the plant. This plan shall include a schedule for completing any recommended improvements and a plan for financing the improvements.

(N) On or before July 31st of each calendar year the main flow meter shall be calibrated in accordance with the manufacturers' specifications. The actual record of the calibration shall be retained onsite and, upon request, the permittee shall submit to the Commissioner a copy of that record.

(O) The permittee shall operate and maintain all processes as installed in accordance with the approved plans and specifications and as outlined in the associated operation and maintenance manual. This includes but is not limited to all recycle pumping systems, aeration equipment, aeration tank cycling, mixing equipment, anoxic basin, chemical feed systems, effluent filters or any other process equipment necessary for the optimal removal of pollutants. The permittee shall not bypass or fail to operate any of the approved equipment or processes without the written approval of the Commissioner.

(P) The permittee is hereby authorized to accept septage at the treatment facility; or other locations as approved by the Commissioner.

(Q) The temperature of any discharge shall not increase the temperature of the receiving stream above 85°F, or, in any case, raise the normal temperature of the receiving stream more than 4°F.

SECTION 5: SPECIFIC EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

(A) The discharge(s) shall not exceed and shall otherwise conform to the specific terms and conditions listed in this permit. The discharge is restricted by, and shall be monitored in accordance with Tables A through D incorporated in this permit as Attachment 1.

(B) The Permittee shall monitor the performance of the treatment process in accordance with the Monthly Operating Report (MOR) and the Nutrient Analysis Report (NAR) incorporated in this permit as Attachment 2, Tables A and B, respectively.

SECTION 6: SAMPLE COLLECTION, HANDLING and ANALYTICAL TECHNIQUES

(A) Chemical Analysis

(1) Chemical analyses to determine compliance with effluent limits and conditions established in this permit, shall be performed using the methods approved pursuant to the Code of Federal Regulations, Part 136 of title 40 (40 CFR

136) unless an alternative method has been approved in writing pursuant to 40 CFR 136.4 or as provided in Section 22a-430-3-(j)(7) of the RCSA. Chemicals which do not have methods of analysis defined in 40 CFR 136 or the RCSA shall be analyzed in accordance with methods specified in this permit.

(2) All metals analyses identified in this permit shall refer to analyses for Total Recoverable Metal, as defined in 40 CFR 136 unless otherwise specified.

(3) Grab samples shall be taken during the period of the day when the peak hourly flow is normally experienced.

(4) Samples collected for bacteriological examination shall be collected between the hours of 11 a.m. and 3 p.m. or at that time of day when the peak hourly flow is normally experienced.

(5) The Minimum Levels specified below represent the concentrations at which quantification must be achieved and verified during the chemical analyses for the parameters identified in Attachment 1, Tables A and B. Analyses for these parameters must include check standards within ten percent of the specified Minimum Level or calibration points equal to or less than the specified Minimum Level.

Parameter	Minimum Level
Arsenic, Total	0.005 mg/l
Beryllium, Total	0.001 mg/l
Cadmium, Total	0.0005 mg/l
Cyanide, Total	0.010 mg/l
Lead, Total	0.005 mg/l
Mercury, Total	0.0002 mg/l
Selenium, Total	0.005 mg/l
Silver, Total	0.002 mg/l
Thallium, Total	0.010 mg/l
Zinc, Total	0.020 mg/l

(6) The value of each parameter for which monitoring is required under this permit shall be reported to the maximum level of accuracy and precision possible consistent with the requirements of this Section of the permit.

(7) Effluent analyses for which quantification was verified during the analysis at or below the minimum levels specified in this Section and which indicate that a parameter was not detected shall be reported as "less than x" where 'x' is the numerical value equivalent to the analytical method detection limit for that analysis.

(8) Results of effluent analyses which indicate that a parameter was not present at a concentration greater than or equal to the Minimum Level specified for that analysis shall be considered equivalent to zero (0.0) for purposes of determining compliance with effluent limitations or conditions specified in this permit.

(B) Acute Aquatic Toxicity Test

(1) Samples for monitoring of Aquatic Toxicity shall be collected and handled as prescribed in "Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms" (EPA/600/4-90/027F).

(a) Composite samples shall be chilled as they are collected. Grab samples shall be chilled immediately following collection. Samples shall be held at 0 - 6ºC until Aquatic Toxicity testing is initiated.

(b) Samples shall be taken at the final effluent for Aquatic Toxicity unless otherwise approved in writing by the Commissioner for monitoring at this facility.

(c) Chemical analyses of the parameters identified in Attachment 1, Table B shall be conducted on an aliquot of the same sample tested for Aquatic Toxicity.

(i) At a minimum, pH, specific conductance, total alkalinity, total hardness, and total residual chlorine shall be measured in the effluent sample and, during Aquatic Toxicity tests, in the highest concentration of the test and in the dilution (control) water at the beginning of the test and at test termination. If total residual chlorine is not detected at test initiation, it does not need to be measured at test termination.

Dissolved oxygen, pH, and temperature shall be measured in the control and all test concentrations at the beginning of the test, daily thereafter, and at test termination.

(d) Tests for Aquatic Toxicity shall be initiated within 36 hours of sample collection.

(2) Monitoring for Aquatic Toxicity to determine compliance with the permit limit on Aquatic Toxicity (invertebrate) shall be conducted for 48 hours utilizing neonatal (less than 24 hours old) *Daphnia pulex*.

(3) Monitoring for Aquatic Toxicity to determine compliance with the permit limit on Aquatic Toxicity (vertebrate) shall be conducted for 48 hours utilizing larval (1 to 14-day old with no more than 24 hours range in age) *Pimephales promelas*.

(4) Tests for Aquatic Toxicity shall be conducted as prescribed for static non-renewal acute tests in "Methods for measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms" (EPA/600/4-90/027F), except as specified below.

(a) For Aquatic Toxicity limits, and for monitoring only conditions, expressed as a NOAEL value, Pass/Fail (single concentration) tests shall be conducted at a specified Critical Test Concentration (CTC) equal to the Aquatic Toxicity limit, (100% in the case of monitoring only conditions), as prescribed in Section 22a-430-3(j)(7)(A)(i) of the RCSA.

(b) Organisms shall not be fed during the tests.

(c) Synthetic freshwater prepared with deionized water adjusted to a hardness of 50+/-5 mg/L as $CaCO_3$ shall be used as dilution water in the tests.

(d) Copper nitrate shall be used as the reference toxicant.

(5) For limits expressed as NOAEL = 100%, compliance shall be demonstrated when the results of a valid pass/fail Aquatic Toxicity Test indicate 90% or greater survival in the effluent sample at the CTC (100%).

(C) Chronic Aquatic Toxicity Test

(1) Chronic toxicity testing of the discharge shall be conducted annually during July, August, or September of each year.

(2) Chronic toxicity testing shall be performed on the discharge in accordance with the test methodology established in "Short-Term Methods for Estimating The Chronic Toxicity of Effluents and Receiving Water to Freshwater Organisms" (EPA-600-4-91-002) as referenced in 40 CFR 136 for *Ceriodaphnia* survival and reproduction and Fathead minnow larval survival and growth.

(a) Chronic toxicity tests shall utilize a minimum of five effluent dilutions prepared using a dilution factor of 0.5 (100% effluent, 50% effluent, 25% effluent, 12.5% effluent, 6.25% effluent).

(b) Synthetic freshwater prepared in accordance with EPA-600-4-91-002 at a hardness of 50+/-5 mg/l shall be used as control (0% effluent) and dilution water in the toxicity tests.

(c) Daily composite samples of the discharge (final effluent following disinfection) shall be collected on day 0, day 2, and day 4 of the test. Samples shall not be pH or hardness adjusted, or chemically altered in any way.

(d) Test solutions shall be renewed on day 1 (test initiation), day 3, and day 5 of the test.

(3) All samples of the discharge shall, at a minimum, be analyzed and results reported in accordance with the provisions listed in Section 6(A) of this permit for the following parameters:

pH
Hardness
Alkalinity

Conductivity
Nitrogen, ammonia (total as N)
Solids, Total Suspended
Copper (total recoverable and dissolved)
Zinc (total recoverable and dissolved)

SECTION 7: RECORDING AND REPORTING REQUIREMENTS

(A) The results of chemical analyses and any aquatic toxicity test required above in Section 5 and the referenced Attachment 1 shall be entered on the Discharge Monitoring Report (DMR) and reported to the Bureau of Water Management. The report shall also include a detailed explanation of any violations of the limitations specified. The DMR must be received at the following address by the 15th day of the month following the month in which samples are collected.

ATTN: Municipal Wastewater Monitoring Coordinator
Connecticut Department of Environmental Protection
Bureau of Water Management, Planning and Standards Division
79 Elm Street
Hartford, Connecticut 06106-5127

(1) For composite samples, from other than automatic samplers, the instantaneous flow and the time of each aliquot sample collection shall be recorded and maintained at the POTW.

(B) Complete and accurate test data, including percent survival of test organisms in each replicate test chamber, LC_{50} values and 95% confidence intervals for definitive test protocols, and all supporting chemical/physical measurements performed in association with any aquatic toxicity test, shall be entered on the Aquatic Toxicity Monitoring Report form (ATMR) and sent to the Bureau of Water Management at the address specified above in Section 7 (A) of this permit by the 15th day of the month following the month in which samples are collected.

(C) The results of the process monitoring required above in Section 5 shall be entered on the Monthly Operating Report (MOR) and Nutrient Analysis Report (NAR) forms, included herein as Attachment 2, Tables A and B, respectively, and reported to the Bureau of Water Management. The MOR report shall also be accompanied by a detailed explanation of any violations of the limitations specified. The MOR and NAR must be received at the address specified above in Section 7 (A) of this permit by the 15th day of the month following the month in which the data and samples are collected.

(D) A complete and thorough report of the results of the chronic toxicity monitoring outlined in Section 6(C) shall be prepared as outlined in Section 10 of EPA-600-4-91-002 and submitted to the Department for review on or before December 31 of each calendar year to the address specified above in Section 7 (A) of this permit.

SECTION 8: RECORDING AND REPORTING OF VIOLATIONS, ADDITIONAL TESTING REQUIREMENTS, BYPASSES, MECHANICAL FAILURES, AND MONITORING EQUIPMENT FAILURES

(A) If any acute toxicity sample analysis indicates that an Aquatic toxicity effluent limitation has been exceeded, or that the test was invalid, a second sample of the effluent shall be collected and tested for Acute Aquatic Toxicity and associated chemical parameters, as described above in Section 5 and Section 6, and the results reported to the Bureau of Water Management (Attn: Aquatic Toxicity) via the ATMR form (see Section 7 (B)) within 30 days of the previous test. These test results shall also be reported on the next month's DMR report pursuant to Section 7 (A). The results of all toxicity tests and associated chemical parameters, valid and invalid, shall be reported.

(B) If any two consecutive test results or any three test results in a twelve month period indicates that the aquatic toxicity limit has been exceeded, the permittee shall immediately take all reasonable steps to eliminate toxicity wherever possible and shall submit a report, to the Bureau of Water Management (Attn: Aquatic Toxicity), for the review and written approval of the Commissioner in accordance with Section 22a-430-3(j)(10)(c) of the RCSA describing proposed steps to eliminate the toxic impact of the discharge on the receiving water body. Such a report shall include a proposed time schedule to accomplish toxicity reduction and the permittee shall comply with any schedule approved by the Commissioner.

(C) Section 22a-430-3(k) of the RCSA shall apply in all instances of bypass including a bypass of the treatment plant or a component of the sewage collection system planned during required maintenance. The Department of Environmental Protection, Bureau of Water Management, Planning and Standards Division (860) 424-3704, the Department of Public

Health, Water Supply Section (860) 509-7333 and Recreation Section (860) 509-7297, and the local Director of Health shall be notified within 2 hours of learning of the event by telephone during normal business hours. If the discharge or bypass occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday), notification shall be made within 2 hours of learning of the event to the Emergency Response Unit at (860) 424-3338 and the Department of Public Health at (860) 509-8000. A written report shall be submitted to the Department of Environmental Protection, Bureau of Water Management, Planning and Standards Division, Municipal Facilities Section within five days of each occurrence, or potential occurrence, of a discharge or bypass of untreated or partially treated sewage.

The written report shall contain:

(a) The nature and cause of the bypass, permit violation, treatment component failure, and/or equipment failure,

(b) the time the incident occurred and the anticipated time which it is expected to continue or, if the condition has been corrected, the duration,

(c) the estimated volume of the bypass or discharge of partially treated or raw sewage,

(d) the steps being taken to reduce or minimize the effect on the receiving waters, and

(e) the steps that will be taken to prevent reoccurrence of the condition in the future.

(D) Section 22a-430-3(j) of the RCSA shall apply in the event of any noncompliance with a maximum daily limit and/or any noncompliance that is greater than two times any permit limit. The permittee shall notify in the same manner as in paragraph C of this Section, the Department of Environmental Protection, Bureau of Water Management, Planning and Standards Division except, if the failure occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday) the permittee may wait to make the verbal report until 10:30 am of the next business day.

(E) Section 22a-430-3(j) of the RCSA shall apply in all instances of monitoring equipment failures. In the event of any failure of the monitoring equipment including, but not limited to, loss of refrigeration or loss of flow proportion sampling ability, the permittee shall notify in the same manner as in paragraph C of this Section, the Department of Environmental Protection, Bureau of Water Management, Planning and Standards Division except, if the failure occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday) the permittee may wait to make the verbal report until 10:30 am of the next business day.

(F) In addition to the reporting requirements contained in Section 22a-430-3(i), (j), and (k) of the Regulations of Connecticut State Agencies, the permittee shall notify in the same manner as in paragraph C of this Section, the Department of Environmental Protection, Bureau of Water Management, Planning and Standards Division, Municipal Facilities Section (860) 424-3704 concerning the failure of any major component of the treatment facilities which the permittee may have reason to believe would result in an effluent violation. If the failure occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday), notification shall be made within 2 hours of learning of the event to the Emergency Response Unit at (860) 424-3338 and the Department of Public Health at (860) 509-8000.

This permit is hereby issued on the 29th day of September, 2004.

Arthur J. Rocque, Jr.
Commissioner

ATTACHMENT 1

Tables A through E

TABLE A

Discharge Serial Number (DSN): **001-1**						Monitoring Location: **1**				
Wastewater Description: **Sanitary Sewage**										
Monitoring Location Description: **Final Effluent**										
Allocated Zone of Influence (ZOI): **0.50cfs**					In-stream Waste Concentration (IWC): **75.59%**					
PARAMETER	**Units**	**FLOW/TIME BASED MONITORING**				**INSTANTANEOUS MONITORING**			**REPORT FORM**	**Minimum Level Analysis See Section 6**
		Average Monthly Limit	**Maximum Daily Limit**	**Sample Freq.**	**Sample type**	**Instantaneous Limit or Required Range**[3]	**Sample Freq.**	**Sample Type**		
Alkalinity	mg/l	NA	NA	NR	NA	------	Monthly	Grab	MOR	
Biochemical Oxygen Demand (5 day) (November 1st through March 31st)	mg/l	20 mg/l and 15 % of Influent [1]	40 mg/l	Weekly	Daily Composite	NA	NR	NA	DMR/MOR	
Biochemical Oxygen Demand (5day) (April 1st through October 31st)	mg/l	10 mg/l and 15% of Influent[1]	20mg/l	Weekly	Daily Composite	NA	NR	NA	DMR/MOR	
Fecal Coliform (May 1st through September 30th)	per100 ml	NA	NA	NR	NA	see remarks (A) and (B) below	Weekly	Grab	DMR/MOR	
Flow, Average Daily	MGD	1.0	------	Continuous [2]	Daily flow	NA	NR	NA	DMR/MOR	
Nitrogen, Ammonia (total as N) April	mg/l	7.3mg/l	------	2/Week	Daily Composite	NA	NR	NA	DMR/MOR/NAR	*
May		4.9mg/l	------	2/Week	Daily Composite	NA	NR	NA	DMR/MOR/NAR	*
June		2.3mg/l	------	2/Week	Daily Composite	NA	NR	NA	DMR/MOR/NAR	*
July through September		1.6mg/l	------	2/Week	Daily Composite	NA	NR	NA	DMR/MOR/NAR	*
October		2.7mg/l	------	2/Week	Daily Composite	NA	NR	NA	DMR/MOR/NAR	*
November through March		------	------	Monthly	Daily Composite	NA	NR	NA	MOR/NAR	
Nitrogen, Ammonia (total as N) ()	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	NAR	
Nitrogen, Nitrate (total as N)	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	NAR	
Nitrogen, Nitrite (total as N)	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	NAR	
Nitrogen, Total Kjeldahl	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	NAR	
Nitrogen, Total	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	NAR	
Oxygen Dissolved (May 1st through September 30th)	mg/l	NA	NA	NR	NA	> 6.0	Work Day	Grab	MOR	

Oxygen Dissolved (October 1st through April 30th)	mg/l	NA	NA	NR	NA	------	Work Day	Grab	MOR	
pH	S.U.	NA	NA	NR	NA	6 - 9	Work Day	Grab	DMR/MOR	
Phosphate, Ortho	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	NAR	
Phosphorous, Total (May 1st through September 30th)	mg/l	1.0 mg/l	2.0 mg/l	Weekly	Daily Composite	NA	NR	NA	DMR/NAR	
Phosphorous, Total (October 1st through April 30th)	mg/l	------	------	Weekly	Daily Composite	NA	NR	NA	NAR	
Solids, Settleable	mg/l	NA	NA	NA	NA	------	Work Day	Grab	MOR	
Solids, Total Suspended (November 1st through March 31st)	mg/l	20 mg/l and 15 % of Influent [1]	40 mg/l	Weekly	Daily Composite	NA	NA	NA	DMR/MOR	
Solids, Total Suspended (April 1st through October 31st)	mg/l	10 mg/l and 15 % of Influent [1]	20 mg/l	Weekly	Daily Composite	NA	NA	NA	DMR/MOR	
Turbidity	NTU	NA	NA	NA	NA	------	Work Day	Grab	MOR	
Temperature	°F	NA	NA	NR	NA	------	Work Day	Grab	MOR	
UV Intensity (May 1st through September 30th)	mW/cm^2	NA	NA	NA	NA	------	4/Work Day	Grab	MOR	
Zinc, Total	kg/d	0.195 kg/d	0.326 kg/d	Weekly	Daily Composite	NA	NA	NA	DMR/MOR	*

TABLE A – FOOTNOTES AND REMARKS

Footnotes:

[1] The discharge shall meet the more stringent of 10 mg/l (April 1 through October 31) and 20 mg/l (November 1 through March 31) or 15% of the average monthly influent BOD_5 and suspended solids (Table D, Monitoring Location G).

[2] The permittee shall record and report on the monthly operating report the minimum, maximum and total flow for each day of discharge and the average daily flow for each sampling month. The permittee shall report, on the discharge monitoring report, the average daily flow for each sampling month.

[3] The instantaneous limits in this column are maximum limits except for Dissolved Oxygen and UV Dose which are minimum limits.

Remarks:

(A) The geometric mean of the fecal coliform bacteria values for the effluent samples collected in a period of thirty (30) consecutive days during the period from May 1st through September 30th shall not exceed 200 per 100 milliliters.

(B) The geometric mean of the fecal coliform bacteria values for the effluent samples collected in a period of seven (7) consecutive days during the period from May 1st through September 30th shall not exceed 400 per 100 milliliters.

TABLE B

Discharge Serial Number (DSN): **001-1**				Monitoring Location: **T**		
Wastewater Description: **Sanitary Sewage**						
Monitoring Location Description: **Final Effluent**						
Allocated Zone of Influence (ZOI): **0.50cfs**			In-stream Waste Concentration (IWC): **75.59 %**			
PARAMETER	**Units**	**Maximum Daily Limit**	**Sampling Frequency**	**Sample Type**	**Reporting form**	**Minimum Level Analysis See Section 6**
Antimony, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Aquatic Toxicity, *Daphnia pulex* [1]	%	NOAEL=100%	Quarterly	Daily Composite	ATMR/DMR	
Aquatic Toxicity, *Pimephales promelas* [1]	%	NOAEL=100%	Quarterly	Daily Composite	ATMR/DMR	
Arsenic, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Beryllium, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
BOD5	mg/l	-----	Quarterly	Daily Composite	ATMR	
Cadmium, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Chromium, Hexavalent	mg/l	-----	Quarterly	Daily Composite	ATMR	
Chromium, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Chlorine, Total Residual	mg/l	-----	Quarterly	Daily Composite	ATMR	
Copper, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Cyanide, Amenable	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Cyanide, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Lead, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Mercury, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Nickel, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Nitrogen, Ammonia (total as N)	mg/l	-----	Quarterly	Daily Composite	ATMR	
Nitrogen, Nitrate, (total as N)	mg/l	-----	Quarterly	Daily Composite	ATMR	
Nitrogen, Nitrite, (total as N)	mg/l	-----	Quarterly	Daily Composite	ATMR	
Phenols, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Selenium, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Silver, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Suspended Solids, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	
Thallium, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Zinc, Total	mg/l	-----	Quarterly	Daily Composite	ATMR	*
Remarks: [1]The results of the Toxicity Tests are recorded in % survival, however, the permittee shall report pass/fail on the DMR based on criteria in Section 6(B) of this permit.						

TABLE C

Discharge Serial Number: **001-1**	Monitoring Location: N			
Wastewater Description: **Activated Sludge**				
Monitoring Location Description: **Each Aeration Unit**				
PARAMETER	**REPORTING FORMAT**	**INSTANTANEOUS MONITORING**		**REPORTING FORM**
		Sample Frequency	Sample Type	
Oxygen, Dissolved	High & low for each WorkDay	4/WorkDay	Grab	MOR
Sludge Volume Index	WorkDay	WorkDay	Grab	MOR
Mixed Liquor Suspended Solids	WorkDay	WorkDay	Grab	MOR

TABLE D

Discharge Serial Number: **001-1**				Monitoring Location: **G**			
Wastewater Description: **Sanitary Sewage**							
Monitoring Location Description: **Influent**							
PARAMETER	**Units**	**DMR REPORTING FORMAT**	**FLOW/TIME BASED MONITORING**		**INSTANTANEOUS MONITORING**		**REPORTING FORM**
			Sample Frequency	**Sample Type**	**Sample Frequency**	**Sample Type**	
Alkalinity, Total	mg/l		NA	NA	Monthly	Grab	MOR
Biochemical Oxygen Demand (5 day)	mg/l	Monthly Average	Weekly	Daily Composite	NA	NA	DMR/MOR
Nitrogen, Ammonia (total as N)	mg/l		Monthly	Daily Composite	NA	NA	NAR
Nitrogen, Nitrate (total as N)	mg/l		Monthly	Daily Composite	NA	NA	NAR
Nitrogen, Nitrite (total as N)	mg/l		Monthly	Daily Composite	NA	NA	NAR
Nitrogen, Tkn	mg/l		Monthly	Daily Composite	NA	NA	NAR
Nitrogen, Total	mg/l		Monthly	Daily Composite	NA	NA	NAR
Phosphorus, Total	mg/l		Monthly	Daily Composite	NA	NA	MOR
pH	S.U.		NA	NA	Work Day	Grab	MOR
Solids, Total Suspended	mg/l	Monthly Average	Weekly	Daily Composite	NA	NA	DMR/MOR
Temperature	°F		NA	NA	Work Day	Grab	MOR

TABLE E

Discharge Serial Number: **001-1**	Monitoring Location: S		
Wastewater Description: **dewatered sludge**			
Monitoring Location Description: **dewatered sludge**			
PARAMETER	**INSTANTANEOUS MONITORING**		**REPORTING FORM**
	Units	**Grab Sample Freq.**	
Arsenic, Total	mg/kg	Semi-annually	DMR
Beryllium, Total	mg/kg	Semi-annually	DMR
Cadmium, Total	mg/kg	Semi-annually	DMR
Chromium, Total	mg/kg	Semi-annually	DMR
Copper, Total	mg/kg	Semi-annually	DMR
Lead, Total	mg/kg	Semi-annually	DMR
Mercury, Total	mg/kg	Semi-annually	DMR
Nickel, Total	mg/kg	Semi-annually	DMR
Nitrogen, Ammonia *	mg/kg	Semi-annually	DMR*
Nitrogen, Nitrate (total as N) *	mg/kg	Semi-annually	DMR*
Nitrogen, Organic *	mg/kg	Semi-annually	DMR*
Nitrogen, Nitrite (total as N) *	mg/kg	Semi-annually	DMR*
Nitrogen, Total *	mg/kg	Semi-annually	DMR*
pH *	S.U.	Semi-annually	DMR*
Polychlorinated Biphenyls	mg/kg	Semi-annually	DMR
Solids, Fixed	%	Semi-annually	DMR
Solids, Total	%	Semi-annually	DMR
Solids, Volatile	%	Semi-annually	DMR
Zinc, Total	mg/kg	Semi-annually	DMR
(*) required for composting or land application only			

ATTACHMENT 2

MONTHLY OPERATING REPORT FORM
AND
NUTRIENT ANALYSIS REPORT

Monthly Operating Report

Facility Name: South Street
Town: Ridgefield
Month:
Permit # CT0100854

Please return form to:
Municipal Facilities Section
Bureau of Water Management
79 Elm Street
Hartford, CT 06106-5127

Flow-		
Max. Day	Mgd	daily
Min. Day	Mgd	daily
Total Day	Mgd	daily
Primary Sludge		
Volume	gals	NA
% Solids	%	NA
Weight	lbs.	NA
Aeration Tanks		
MLSS	mg/l	NR
SVI	ml/g	NR
Dissolved Oxygen	mg/l	NR
Trickling Filter Recirc.	%	NA
Return Sludge		
% Flow	%	Daily
% Solids	%	NR
Waste Sludge	lbs.	Daily
Sludge Disp. Loc.	PASAIC VALLEY SEW[illegible] COMMIS[illegible]	
Day Removed (gal)		
Anaerobic Digester		
pH, Pri.	S.U.	NA
pH, Sec	S.U.	NA
Alkalinity, Pri.	mg/l	NA
Alkalinity, Sec.	mg/l	NA
Temp., Pri	F	NA
Temp., Sec.	F	NA
Volatile Acids, Pri	mg/l	NA
Volatile Acids, Sec	mg/l	NA
Aerobic Digester		
Solids, Tot.	lbs.	NA
Solids, Volatile	lbs.	NA
Waste Accepted		
Septic	gals	Daily
BOD(CBOD)(5-day)		
Inf.	mg/l	weekly
Prim.	mg/l	NA
Eff.	mg/l	weekly
Fecal Coliform	per	weekly
Chlorine Dose	lbs.	NA
	mg/l	NA
Chlorine Res. - hi	mg/l	NA
- low	mg/l	NA

Nitrogen		
Ammonia, Inf	mg/l	weekly
Ammonia, Prim. Eff.	mg/l	NA
Ammonia, Eff	mg/l	weekly
Nitrate, Inf	mg/l	weekly
Nitrate, Prim. Eff	mg/l	NA
Nitrate Eff.	mg/l	weekly
Nitrite, Inf	mg/l	weekly
Nitrite, Prim. Eff	mg/l	NA
Nitrite Eff.	mg/l	weekly
Total Nitrogen, Inf	mg/l	weekly
Total Nitrogen, Prim. Eff	mg/l	NA
Total Nitrogen, Eff.	mg/l	weekly
TKN, Inf.	mg/l	weekly
TKN, Prim. Eff.	mg/l	NA
TKN, Eff.	mg/l	weekly
O_2 Dissolved, Eff	mg/l	WD
pH		
Inf	S.U.	WD
Eff	S.U.	WD
Phosphorus		
Inf	mg/l	NA
Eff	mg/l	NA
Total Phosphorus, Eff	mg/l	weekly
Orthophosphate, Eff	mg/l	weekly
Solids Settleable		
Eff	ml/l	WD
Solids Susp.		
Inf	mg/l	weekly
Prim	mg/l	NA
Eff	mg/l	weekly
Temperature		
Inf	t	NR
Eff	t	NR
Turbidity, Eff	N.T.U.	WD
Metals		
Copper	KG/D	weekly
Zinc	KG/D	weekly
Others		

Comments

Statement of Acknowledgment: I certify under penalty of Law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is to the best of my knowledge and belief, true accurate and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

Authorized Official (please type or print) ____________ Title ____________

Signature ____________ Date ________ Phone ____________

TABLE B

Nutrient Analysis Report

for compliance with NPDES permit

Town of Ridgefield (Main) Permit # CT0100854 Flow Rate ______ mgd Sampling Date __/__/__

Parameter	Raw Influent		Primary Effluent		Final Effluent		Plant Efficiency
	mg/l	lbs/day	mg/l	lbs/day	mg/l	lbs/day	%
Ammonia							
Nitrite							
Nitrate							
TKN							
Total Nitrogen = TKN + nitrite + nitrate							
Orthophosphates							
Total Phosphorus							

Notes: lbs/day = 8.34 x flow (mgd) x mg/l of pollutant
Flow = Total daily flow on sampling date (mgd)
Plant Efficiency = 100% x (raw influent – final effluent) / raw influent

DATA TRACKING AND TECHNICAL FACT SHEET

Permittee: Town of Ridgefield **PAMS Company ID:** 92440

PERMIT, ADDRESS, AND FACILITY DATA

PERMIT #: CT0100854 **APPLICATION #:** 200400717 **FACILITY ID.** 118-001

Mailing Address:	Location Address:
Street: 400 Main Street	Street: 22 South Street
City: Ridgefield ST: CT Zip: 06877	City: Riddgefield ST: CT Zip: 06877
Contact Name: Joseph Lauria	Contact Name: Joseph Lauria
Phone No.: 203-431-2734	Phone No.: 203-431-2734

PERMIT INFORMATION

DURATION 5 YEAR X 10 YEAR __ 30 YEAR

TYPE New _ Reissuance X Modification

CATEGORIZATION POINT (X) NON-POINT ( ) GIS # 6006

NPDES (X) PRETREAT () GROUND WATER(UIC) ( ) GROUND WATER (OTHER) ( )

NPDES MAJOR(**MA**) X
NPDES SIGNIFICANT MINOR or PRETREAT SIU **(SI)**
NPDES or PRETREATMENT MINOR **(MI)**

COMPLIANCE SCHEDULE YES__ NO X
POLLUTION PREVENTION __ TREATMENT REQUIREMENT__
WATER QUALITY REQUIREMENT __ OTHER

OWNERSHIP CODE
Private __ Federal __ State __ Municipal (town only) X Other public

DEP STAFF ENGINEER Stela Marusin

PERMIT FEES

Discharge Code	DSN Number	Annual Fee
111000d	001	$2,242.50

FOR NPDES DISCHARGES
Drainage basin Code: 4952 Present/Future Water Quality Standard: B/B

NATURE OF BUSINESS GENERATING DISCHARGE
Domestic Sewage

PROCESS AND TREATMENT DESCRIPTION (by DSN)
Advanced wastewater treatment, phoshorus removal, sand filtration and ultraviolet seasonal disinfection

RESOURCES USED TO DRAFT PERMIT

X Federal Effluent Limitation Guideline 40CFR 133
Secondary Treatment Category

__ Performance Standards

__ Federal Development Document
name of category

X Department File Information

X Connecticut Water Quality Standards

__ Anti-degradation Policy

__ Coastal Management Consistency Review Form

__ Other - Explain

BASIS FOR LIMITATIONS, STANDARDS OR CONDITIONS

X Secondary Treatment

__ Case by Case Determination (See Other Comments)

X Section 22a-430-4(r) of the Regulations of Connecticut State Agencies

X In order to meet in-stream water quality (See General Comments)

__ Anti-degradation policy

GENERAL COMMENTS
The need for inclusion of water quality based discharge limitations in this permit was evaluated consistent with Connecticut Water Quality Standards and criteria, pursuant to 40 CFR 122.44(d). Each parameter was evaluated for consistency with the available aquatic life criteria (acute and chronic) and human health (fish consumption only) criteria, considering the zone of influence allocated to the facility where appropriate. The effluent limits contained in this permit were established on a Waste Load Allocation developed for the upper Norwalk River in 1988 and first adopted into Ridgefield's discharge permit in 1989. The statistical procedures outlined in the EPA Technical Support Document for Water Quality-based Toxics Control (EPA/505/2-90-001) were employed to calculate the need for such limits. Comparison of monitoring data and its inherent variability with the calculated water quality based limits indicates a low statistical probability of exceeding such limits. Therefore, water quality based limits for ammonia, and zinc were included in the permit at this time.

OTHER COMMENTS
There are no water quality based limits for copper included in this permit since there appears to be no reasonable potential for exceeding the limit.

APPENDIX B

ROUTE 7 WWTF NPDES PERMIT

Connecticut Department of
ENERGY &
ENVIRONMENTAL
PROTECTION

79 Elm Street • Hartford, CT 06106-5127 www.ct.gov/deep Affirmative Action/Equal Opportunity

MUNICIPAL NPDES PERMIT

issued to

Permittee:
Town of Ridgefield
400 Main Street, Ridgefield,
Ridgefield, Connecticut 06877

Location Address:
Town of Ridgefield
9101 Ethan Allen Highway (Rte. 7)
Ridgefield, Connecticut 06877

Facility ID: 118-002 **Permit ID:** CT0101451 **Permit Expires:** September 17, 2019

Receiving Stream: Norwalk River **Design Flow Rate:** 0.12 MGD

SECTION 1: GENERAL PROVISIONS

(A) This permit is reissued in accordance with Section 22a-430 of Chapter 446k, Connecticut General Statutes ("CGS"), and Regulations of Connecticut State Agencies ("RCSA") adopted thereunder, as amended, and Section 402(b) of the Clean Water Act, as amended, 33 USC 1251, et. seq., and pursuant to an approval dated September 26, 1973, by the Administrator of the United States Environmental Protection Agency for the State of Connecticut to administer a N.P.D.E.S. permit program.

(B) The Town of Ridgefield ("permittee"), shall comply with all conditions of this permit including the following sections of the RCSA which have been adopted pursuant to Section 22a-430 of the CGS and are hereby incorporated into this permit. **Your attention is especially drawn to the notification requirements of subsection (i)(2), (i)(3), (j)(1), (j)(6), (j)(8), (j)(9)(C), (j)(10)(C), (j)(11)(C), (D), (E), and (F), (k)(3) and (4) and (l)(2) of Section 22a-430-3.** To the extent this permit imposes conditions more stringent than those found in the regulations, this permit shall apply.

Section 22a-430-3 General Conditions

- **(a)** Definitions
- **(b)** General
- **(c)** Inspection and Entry
- **(d)** Effect of a Permit
- **(e)** Duty to Comply
- **(f)** Proper Operation and Maintenance
- **(g)** Sludge Disposal
- **(h)** Duty to Mitigate
- **(i)** Facility Modifications; Notification
- **(j)** Monitoring, Records and Reporting Requirements
- **(k)** Bypass
- **(l)** Conditions Applicable to POTWs
- **(m)** Effluent Limitation Violations
- **(n)** Enforcement
- **(o)** Resource Conservation
- **(p)** Spill Prevention and Control
- **(q)** Instrumentation, Alarms, Flow Recorders
- **(r)** Equalization

Section 22a-430-4 Procedures and Criteria

- **(a)** Duty to Apply
- **(b)** Duty to Reapply
- **(c)** Application Requirements
- **(d)** Preliminary Review
- **(e)** Tentative Determination
- **(f)** Draft Permits, Fact Sheets
- **(g)** Public Notice, Notice of Hearing

(h) Public Comments
(i) Final Determination
(j) Public Hearings
(k) Submission of Plans and Specifications. Approval.
(l) Establishing Effluent Limitations and Conditions
(m) Case-by-Case Determinations
(n) Permit Issuance or Renewal
(o) Permit or Application Transfer
(p) Permit Revocation, Denial or Modification
(q) Variances
(r) Secondary Treatment Requirements
(s) Treatment Requirements
(t) Discharges to POTWs - Prohibitions

(C) Violations of any of the terms, conditions, or limitations contained in this permit may subject the permittee to enforcement action including, but not limited to, seeking penalties, injunctions and/or forfeitures pursuant to applicable sections of the CGS and RCSA.

(D) Any false statement in any information submitted pursuant to this Section of the permit may be punishable as a criminal offense under Section 22a-438 or 22a-131a of the CGS or in accordance with Section 22a-6, under Section 53a-157b of the CGS.

(E) The permittee shall comply with Section 22a-416-1 through Section 22a-416-10 of the RCSA concerning operator certification.

(F) No provision of this permit and no action or inaction by the Commissioner shall be construed to constitute an assurance by the Commissioner that the actions taken by the permittee pursuant to this permit will result in compliance or prevent or abate pollution.

(G) Nothing in this permit shall relieve the permittee of other obligations under applicable federal, state and local law.

(H) An annual fee shall be paid for each year this permit is in effect as set forth in Section 22a-430-7 of the RCSA. As of October 1, 2009 the annual fee is $ 1722.50.

SECTION 2: DEFINITIONS

(A) The definitions of the terms used in this permit shall be the same as the definitions contained in Section 22a-423 of the CGS and Section 22a-430-3(a) and 22a-430-6 of the RCSA, except for "Composite" and "No Observable Acute Effect Level (NOAEL)" which are redefined below.

(B) In addition to the above, the following definitions shall apply to this permit:

"------" in the limits column on the monitoring tables in Attachment 1 means a limit is not specified but a value must be reported on the DMR, MOR, and/or the ATMR.

"Annual" in the context of any sampling frequency, shall mean the sample must be collected in the month of February.

"Average Monthly Limit" means the maximum allowable "Average Monthly Concentration" as defined in Section 22a-430-3(a) of the RCSA when expressed as a concentration (e.g. mg/l); otherwise, it means "Average Monthly Discharge Limitation" as defined in Section 22a-430-3(a) of the RCSA.

"Bi-Weekly" in the context of any sampling frequency, shall mean once every two weeks.

"Composite" or **"(C)"** means a sample consisting of a minimum of eight aliquot samples collected at equal intervals of no less than 30 minutes and no more than 60 minutes and combined proportionally to flow over the sampling period provided that during the sampling period the peak hourly flow is experienced.

"Critical Test Concentration" or **"(CTC)"** means the specified effluent dilution at which the permittee is to conduct a single-concentration Aquatic Toxicity Test.

"Daily Composite" or **"(DC)"** means a composite sample taken over a full operating day consisting of grab samples collected at equal intervals of no more than sixty (60) minutes and combined proportionally to flow; or, a composite sample continuously collected over a full operating day proportionally to flow.

"Daily Concentration" means the concentration of a substance as measured in a daily composite sample, or, arithmetic average of all grab sample results defining a grab sample average.

"Daily Quantity" means the quantity of waste discharged during an operating day.

"Geometric Mean" is the "n"th root of the product of "n" observations.

"Infiltration" means water other than wastewater that enters a sewer system (including sewer system and foundation drains) from the ground through such means as defective pipes, pipe joints, connections, or manholes. Infiltration does not include, and is distinguished from, inflow.

"Inflow" means water other than wastewater that enters a sewer system (including sewer service connections) from sources such as, but not limited to, roof leaders, cellar drains, yard drains, area drains, drains from springs and swampy areas, cross connections between storm sewers and sanitary sewers, catch basins, cooling towers, storm waters, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration.

"Instantaneous Limit" means the highest allowable concentration of a substance as measured by a grab sample, or the highest allowable measurement of a parameter as obtained through instantaneous monitoring.

"In-stream Waste Concentration" or **"(IWC)"** means the concentration of a discharge in the receiving water after mixing has occurred in the allocated zone of influence.

"MGD" means million gallons per day.

"Maximum Daily Limit" means the maximum allowable "Daily Concentration" (defined above) when expressed as a concentration (e.g. mg/l), otherwise, it means the maximum allowable "Daily Quantity" as defined above, unless it is expressed as a flow quantity. If expressed as a flow quantity it means "Maximum Daily Flow" as defined in Section 22a-430-3(a) of the RCSA.

"Monthly Minimum Removal Efficiency" means the minimum reduction in the pollutant parameter specified when the effluent average monthly concentration for that parameter is compared to the influent average monthly concentration.

"NA" as a Monitoring Table abbreviation means "not applicable".

"NR" as a Monitoring Table abbreviation means "not required".

"No Observable Acute Effect Level" or **"(NOAEL)"** means any concentration equal to or less than the critical test concentration in a single concentration (pass/fail) toxicity test, conducted pursuant to Section 22a-430-3(j)(7)(A)(i) of the RCSA, demonstrating 90% or greater survival of test organisms at the CTC.

"Quarterly" in the context of any sampling frequency, shall mean sampling is required in the months of February, May, August and November.

"Range During Sampling" or **"(RDS)"** as a sample type means the maximum and minimum of all values recorded as a result of analyzing each grab sample of; 1) a Composite Sample, or, 2) a Grab Sample Average. For those permittees with pH meters that provide continuous monitoring and recording, Range During Sampling means the maximum and minimum readings recorded with the continuous monitoring device during the Composite or Grab Sample Average sample collection.

"Range During Month" or **"(RDM)"** as a sample type means the lowest and the highest values of all of the monitoring data for the reporting month.

"Sanitary Sewage" means wastewaters from residential, commercial and industrial sources introduced by direct connection to the sewerage collection system tributary to the treatment works including non-excessive inflow/infiltration sources.

"Twice per Month" in the context of any sampling frequency, mean two samples per calendar month collected no less than 12 days apart.

"ug/l" means micrograms per liter.

"Work Day" in the context of a sampling frequency means, Monday through Friday excluding holidays.

SECTION 3: COMMISSIONER'S DECISION

(A) The Commissioner of Energy and Environmental Protection ("Commissioner") has issued a final decision and found continuance of the existing system to treat the discharge will protect the waters of the state from pollution. The Commissioner's decision is based on application

#200900547 for permit reissuance received on February 24, 2009 and the administrative record established in the processing of that application.

(B) The Commissioner hereby authorizes the Permittee to discharge in accordance with the provisions of this permit, the above referenced application, and all approvals issued by the Commissioner or his authorized agent for the discharges and/or activities authorized by, or associated with, this permit.

(C) The Commissioner reserves the right to make appropriate revisions to the permit, if required after Public Notice, in order to establish any appropriate effluent limitations, schedules of compliance, or other provisions which may be authorized under the Federal Clean Water Act or the CGS or regulations adopted thereunder, as amended. The permit as modified or renewed under this paragraph may also contain any other requirements of the Federal Clean Water Act or CGS or regulations adopted thereunder which are then applicable.

SECTION 4: GENERAL LIMITATIONS AND OTHER CONDITIONS

(A) The Permittee shall not accept any new sources of non-domestic wastewater conveyed to its POTW through its sanitary sewerage system or by any means other than its sanitary sewage system unless the generator of such wastewater; (a) is authorized by a permit issued by the Commissioner under Section 22a-430 CGS (individual permit), or, (b) is authorized under Section 22a-430b (general permit), or, (c) has been issued an emergency or temporary authorization by the Commissioner under Section 22a-6k. All such non-domestic wastewaters shall be processed by the POTW via receiving facilities at a location and in a manner prescribed by the permittee which are designed to contain and control any unplanned releases.

(B) No new discharge of domestic sewage from a single source to the POTW in excess of 6,000 gallons per day shall be allowed by the permittee until the permittee has notified in writing the Municipal Facilities Section of said new discharge.

(C) The permittee shall maintain a system of user charges based on actual use sufficient to operate and maintain the POTW (including the collection system) and replace critical components.

(D) The permittee shall maintain a sewer use ordinance that is consistent with the Model Sewer Ordinance for Connecticut Municipalities prepared by the Department of Energy and Environmental Protection. The Commissioner of Energy and Environmental Protection alone may authorize certain discharges which may not conform to the Model Sewer Ordinance.

(E) No discharge shall contain or cause in the receiving stream a visible oil sheen, floating solids, visible discoloration, or foaming.

(F) No discharge shall cause acute or chronic toxicity in the receiving water body beyond any Zone Of Influence (ZOI) specifically allocated to that discharge in this permit.

(G) The permittee shall maintain an alternate power source adequate to provide full operation of all pump stations in the sewerage collection system and to provide a minimum of primary treatment and disinfection at the water pollution control facility to insure that no discharge of untreated wastewater will occur during a failure of a primary power source.

(H) The average monthly effluent concentration shall not exceed 10% of the average monthly influent concentration for BOD_5 and Total Suspended Solids for all daily composite samples taken in any calendar month.

(I) Any new or increased amount of sanitary sewage discharge to the sewer system is prohibited where it will cause a dry weather overflow or exacerbate an existing dry weather overflow.

(J) Sludge Conditions

(1) The permittee shall comply with all existing federal and state laws and regulations that apply to sewage sludge use and disposal practices, including but not limited to 40 CFR Part 503.

(2) If an applicable management practice or numerical limitation for pollutants in sewage sludge more stringent than existing federal and state regulations is promulgated under Section 405(d) of the Clean Water Act (CWA), this permit shall be modified or revoked and reissued to conform to the promulgated regulations.

(3) The permittee shall give prior notice to the Commissioner of any change(s) planned in the permittees' sludge use or disposal practice. A change in the permittees' sludge use or disposal practice may be a cause for modification of the permit.

(4) Testing for inorganic pollutants shall follow "Test Methods for Evaluating Solid Waste, Physical/Chemical Methods", EPA Publication SW-846 as updated and/or revised.

(K) This permit becomes effective on the 1st day of the month following the date of signature.

(L) When the arithmetic mean of the average daily flow from the POTW for the previous 180 days exceeds 90% of the design flow rate, the permittee shall develop and submit within one year, for the review and approval of the Commissioner, a plan to accommodate future increases in flow to the plant. This plan shall include a schedule for completing any recommended improvements and a plan for financing the improvements.

(M) When the arithmetic mean of the average daily BOD_5 or TSS loading into the POTW for the previous 180 days exceeds 90% of the design load rate, the permittee shall develop and submit for the review of the Commissioner within one year, a plan to accommodate future increases in load to the plant. This plan shall include a schedule for completing any recommended improvements and a plan for financing the improvements.

(N) On or before July 31st of each calendar year the main flow meter shall be calibrated by an independent contractor in accordance with the manufacturer's specifications. The actual record of the calibration shall be retained onsite and, upon request, the permittee shall submit to the Commissioner a copy of that record.

(O) The permittee shall operate and maintain all processes as installed in accordance with the approved plans and specifications and as outlined in the associated operation and maintenance manual. This includes but is not limited to all preliminary treatment processes, primary treatment processes, recycle pumping processes, anaerobic treatment processes, anoxic treatment processes, aerobic treatment processes, flocculation processes, effluent filtration processes or any other processes necessary for the optimal removal of pollutants. The permittee shall not bypass or fail to operate any of the aforementioned processes without the written approval of the Commissioner.

(P) The temperature of any discharge shall not increase the temperature of the receiving stream above 85°F, or, in any case, raise the normal temperature of the receiving stream more than 4°F.

SECTION 5: SPECIFIC EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

(A) The discharge(s) shall not exceed and shall otherwise conform to the specific terms and conditions listed in this permit. The discharge is restricted by, and shall be monitored in accordance with Tables **A** through **F** incorporated in this permit as Attachment 1.

(B) The Permittee shall monitor the performance of the treatment process in accordance with the Monthly Operating Report (MOR) incorporated in this permit as Attachment 2.

SECTION 6: SAMPLE COLLECTION, HANDLING and ANALYTICAL TECHNIQUES

(A) Chemical Analysis

(1) Chemical analyses to determine compliance with effluent limits and conditions established in this permit shall be performed using the methods approved pursuant to the Code of Federal Regulations, Part 136 of Title 40 (40 CFR 136) unless an alternative method has been approved in writing pursuant to 40 CFR 136.4 or as provided in Section 22a-430-3-(j)(7) of the RCSA. Chemicals which do not have methods of analysis defined in 40 CFR 136 or the RCSA shall be analyzed in accordance with methods specified in this permit.

(2) All metals analyses identified in this permit shall refer to analyses for Total Recoverable Metal, as defined in 40 CFR 136 unless otherwise specified.

(3) Grab samples shall be taken during the period of the day when the peak hourly flow is normally experienced.

(4) Samples collected for bacteriological examination shall be collected between the hours of 11 a.m. and 3 p.m. or at that time of day when the peak hourly flow is normally experienced.

(5) The Minimum Levels specified below represent the concentrations at which quantification must be achieved and verified during the chemical analyses for the parameters identified in Attachment 1, Tables A and C. Analyses for these parameters must include check standards within ten percent of the specified Minimum Level or calibration points equal to or less than the specified Minimum Level.

Parameter	Minimum Level
Aluminum	0.050 mg/l
Ammonia Nitrogen	0.010 mg/l
Arsenic, Total	0.005 mg/l
Beryllium, Total	0.001 mg/l
Cadmium, Total	0.0005 mg/l
Chlorine, Total Residual	0.050 mg/l

Chromium, Total	0.005 mg/l
Chromium, Total Hexavalent	0.010 mg/l
Copper, Total	0.005 mg/l
Cyanide, Total	0.010 mg/l
Iron, Total	0.040 mg/l
Lead, Total	0.005 mg/l
Mercury, Total	0.0002 mg/l
Nickel, Total	0.005 mg/l
Phosphorus, Total	0.10 mg/l
Selenium, Total	0.005 mg/l
Silver, Total	0.002 mg/l
Thallium, Total	0.005 mg/l
Zinc, Total	0.020 mg/l

(6) The value of each parameter for which monitoring is required under this permit shall be reported to the maximum level of accuracy and precision possible consistent with the requirements of this Section of the permit.

(7) Effluent analyses for which quantification was verified during the analysis at or below the minimum levels specified in this Section and which indicate that a parameter was not detected shall be reported as "less than x" where 'x' is the numerical value equivalent to the analytical method detection limit for that analysis.

(8) Results of effluent analyses which indicate that a parameter was not present at a concentration greater than or equal to the Minimum Level specified for that analysis shall be considered equivalent to zero (0.0) for purposes of determining compliance with effluent limitations or conditions specified in this permit.

(B) Acute Aquatic Toxicity Test

(1) Samples for monitoring of Acute Aquatic Toxicity shall be collected and handled as prescribed in "Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms" (EPA-821-R-02-012).

(a) Composite samples shall be chilled as they are collected. Grab samples shall be chilled immediately following collection. Samples shall be held at 0 - 6°C until Acute Aquatic Toxicity testing is initiated.

(b) Effluent samples shall not be dechlorinated, filtered, or, modified in any way, prior to testing for Acute Aquatic Toxicity unless specifically approved in writing by the Commissioner for monitoring at this facility. Facilities with effluent dechlorination and/or filtration designed as part of the treatment process are not required to obtain approval from the Commissioner.

(c) Samples shall be taken at the final effluent for Acute Aquatic Toxicity unless otherwise approved in writing by the Commissioner for monitoring at this facility.

(d) Chemical analyses of the parameters identified in Attachment 1, Table C shall be conducted on an aliquot of the same sample tested for Acute Aquatic Toxicity.

(i) At a minimum, pH, specific conductance, total alkalinity, total hardness, and total residual chlorine shall be measured in the effluent sample and, during Acute Aquatic Toxicity tests, in the highest concentration of the test and in the dilution (control) water at the beginning of the test and at test termination. If total residual chlorine is not detected at test initiation, it does not need to be measured at test termination. Dissolved oxygen, pH, and temperature shall be measured in the control and all test concentrations at the beginning of the test, daily thereafter, and at test termination.

(e) Tests for Acute Aquatic Toxicity shall be initiated within 36 hours of sample collection.

(2) Monitoring for Acute Aquatic Toxicity to determine compliance with the permit limit on Acute Aquatic Toxicity (invertebrate) shall be conducted for 48 hours utilizing neonatal (less than 24 hours old) *Daphnia pulex*.

(3) Monitoring for Acute Aquatic Toxicity to determine compliance with the permit limit on Acute Aquatic Toxicity (vertebrate) shall be conducted for 48 hours utilizing larval (1 to 14-day old with no more than 24 hours range in age) *Pimephales promelas*.

(4) Tests for Acute Aquatic Toxicity shall be conducted as prescribed for static non-renewal acute tests in "Methods for measuring the Acute Aquatic Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms" (EPA/821-R-02-012), except as specified below.

(a) For Acute Aquatic Toxicity limits, and for monitoring only conditions, expressed as a NOAEL value, Pass/Fail (single concentration) tests shall be conducted at a specified Critical Test Concentration (CTC) equal to the Aquatic Toxicity limit, (100% in the case of monitoring only conditions), as prescribed in Section 22a-430-3(j)(7)(A)(i) of the RCSA.

(b) Organisms shall not be fed during the tests.

(c) Synthetic freshwater prepared with deionized water adjusted to a hardness of 50±5 mg/L as $CaCO_3$ shall be used as dilution water in the tests.

(d) Copper nitrate shall be used as the reference toxicant.

(5) For limits expressed as NOAEL = 100%, compliance shall be demonstrated when the results of a valid pass/fail Acute Aquatic Toxicity Test indicate 90% or greater survival in the effluent sample at the CTC (100%).

(C) Chronic Aquatic Toxicity Test for Freshwater Discharges

(1) Chronic Aquatic Toxicity testing of the discharge shall be conducted annually during July, August, or September of each year.

(2) Chronic Aquatic Toxicity testing shall be performed on the discharge in accordance with the test methodology established in "Short-Term Methods for Estimating The Chronic Toxicity of Effluents and Receiving Water to Freshwater Organisms" (EPA-821-R-02-013) as referenced in 40 CFR 136 for *Ceriodaphnia* survival and reproduction and Fathead minnow larval survival and growth.

(a) Chronic Aquatic Toxicity tests shall utilize a minimum of five effluent dilutions prepared using a dilution factor of 0.5 (100% effluent, 50% effluent, 25% effluent, 12.5% effluent, 6.25% effluent).

(b) Norwalk River water collected immediately upstream of the area influenced by the discharge shall be used as control (0% effluent) and dilution water in the toxicity tests.

(c) A laboratory water control consisting of synthetic freshwater prepared in accordance with EPA-821-R-02-013 at a hardness of 50±5 mg/l shall be used as an additional control (0% effluent) in the toxicity tests.

(d) Daily composite samples of the discharge (final effluent following disinfection) and grab samples of the Norwalk River, for use as site water control and dilution water, shall be collected on day 0 for test solution renewal on day 1 and day 2 of the test; day 2, for test solution renewal on day 3 and day 4 of the test; and day 4, for test solution renewal for the remainder of the test. Samples shall not be pH or hardness adjusted, or chemically altered in any way.

(3) All samples of the discharge and Norwalk River water used in the Chronic Aquatic Toxicity test shall, at a minimum, be analyzed and results reported in accordance with the provisions listed in Section 6(A) of this permit for the parameters listed in Attachment 1, Table C included herein, excluding Acute Aquatic Toxicity organism testing.

SECTION 7: RECORDING AND REPORTING REQUIREMENTS

(A) The results of chemical analyses and any aquatic toxicity test required above in Section 5 and the referenced Attachment 1 shall be entered on the Discharge Monitoring Report (DMR) and reported to the Bureau of Water Protection and Land Reuse. The report shall also include a detailed explanation of any violations of the limitations specified. The DMR must be received at the following address by the 15th day of the month following the month in which samples are collected.

ATTN: Municipal Wastewater Monitoring Coordinator
Connecticut Department of Energy and Environmental Protection
Bureau of Water Protection and Land Reuse, Planning and Standards Division
79 Elm Street
Hartford, Connecticut 06106-5127

(1) For composite samples, from other than automatic samplers, the instantaneous flow and the time of each aliquot sample collection shall be recorded and maintained at the POTW.

(B) Complete and accurate test data, including percent survival of test organisms in each replicate test chamber, LC_{50} values and 95% confidence intervals for definitive test protocols, and all supporting chemical/physical measurements performed in association with any aquatic toxicity test, shall be entered on the Aquatic Toxicity Monitoring Report form (ATMR) and sent to the Bureau of Water Protection and Land Reuse at the address specified above in Section 7 (A) of this permit by the 15th day of the month following the month in which samples are collected.

(C) The results of the process monitoring required above in Section 5 shall be entered on the Monthly Operating Report (MOR) form, included herein as Attachment 2, and reported to the Bureau of Water Protection and Land Reuse. The MOR report shall also be accompanied by a detailed explanation of any violations of the limitations specified. The MOR, must be received at the address specified above in Section 7 (A) of this permit by the 15th day of the month following the month in which the data and samples are collected.

(D) A complete and thorough report of the results of the chronic toxicity monitoring outlined in Section 6(C) shall be prepared as outlined in Section 10 of EPA-821-R-02-013 and submitted to the Department for review on or before December 31 of each calendar year to the address specified above in Section 7 (A) of this permit.

(E) NetDMR Reporting Requirements

(1) Unless otherwise approved in writing by the Commissioner, no later than one-hundred and twenty (120) days after the issuance of this permit, the Permittee shall begin reporting to the Department electronically using NetDMR, a web-based tool that allows Permittees to electronically submit discharge monitoring reports (DMRs) and other required reports through a secure internet connection. Specific requirements regarding subscription to NetDMR and submittal of data and reports in hard copy form and for submittal using NetDMR are described below:

(a) NetDMR Subscriber Agreement

On or before fifteen (15) days after the issuance of this permit, the Permittee and/or the person authorized to sign the Permittee's discharge monitoring reports ("Signatory Authority") as described in RCSA Section 22a-430-3(b)(2) shall contact the Department and initiate the subscription process for electronic submission of Discharge Monitoring Report (DMR) information. On or before ninety (90) days after issuance of this permit the Permittee shall submit a signed and notarized copy of the *Connecticut DEP NetDMR Subscriber Agreement* to the Department.

(b) Submittal of Reports Using NetDMR

Unless otherwise approved by the Commissioner, on or before one-hundred and twenty (120) days after issuance of this permit, the Permittee and/or the Signatory Authority shall electronically submit DMRs and reports required under this permit to the Department using NetDMR in satisfaction of the DMR submission requirement of this permit. DMRs shall be submitted electronically to the Department no later than the 15th day of the month following the completed reporting period.

(c) Submittal of NetDMR Opt-Out Requests

If the Permittee is able to demonstrate a reasonable basis, such as technical or administrative infeasibility, that precludes the use of NetDMR for electronically submitting DMRs and reports, the Commissioner may approve the submission of DMRs and other required reports in hard copy form ("opt-out request"). Opt-out requests must be submitted in writing to the Department for written approval on or before fifteen (15) days prior to the date a Permittee would be required under this permit to begin filing DMRs and other reports using NetDMR. This demonstration shall be valid for twelve (12) months from the date of the Department's approval and shall thereupon expire. At such time, DMRs and reports shall be submitted electronically to the Department using NetDMR unless the Permittee submits a renewed opt-out request and such request is approved by the Department.

All opt-out requests and requests for the NetDMR subscriber form should be sent to the following address:

Attn: NetDMR Coordinator
Connecticut Department of Energy and Environmental Protection
Water Permitting and Enforcement Division – 2nd Floor
79 Elm Street
Hartford, CT 06106-5127

SECTION 8: RECORDING AND REPORTING OF VIOLATIONS, ADDITIONAL TESTING REQUIREMENTS, BYPASSES, MECHANICAL FAILURES, AND MONITORING EQUIPMENT FAILURES

(A) If any Acute Aquatic Toxicity sample analysis indicates toxicity, or that the test was invalid, an additional sample of the effluent shall be collected and tested for Acute Aquatic Toxicity and associated chemical parameters, as described above in Section 5 and Section 6, and the results reported to the Bureau of Water Protection and Land Reuse (Attn: Aquatic Toxicity) via the ATMR form (see Section 7 (B)) within 30 days of the previous test. These test results shall also be reported on the next month's DMR report pursuant to Section 7 (A). The results of all toxicity tests and associated chemical parameters, valid and invalid, shall be reported.

(B) If any two consecutive Acute Aquatic Toxicity test results or any three Acute Aquatic Toxicity test results in a twelve month period indicates toxicity, the permittee shall immediately take all reasonable steps to eliminate toxicity wherever possible and shall submit a report, to the

Bureau of Water Protection and Land Reuse (Attn: Aquatic Toxicity), for the review and written approval of the Commissioner in accordance with Section 22a-430-3(j)(10)(c) of the RCSA describing proposed steps to eliminate the toxic impact of the discharge on the receiving water body. Such a report shall include a proposed time schedule to accomplish toxicity reduction and the permittee shall comply with any schedule approved by the Commissioner.

(C) Section 22a-430-3(k) of the RCSA shall apply in all instances of bypass including a bypass of the treatment plant or a component of the sewage collection system planned during required maintenance. The Department of Energy and Environmental Protection, Bureau of Water Protection and Land Reuse, Planning and Standards Division, Municipal Facilities Section (860) 424-3704, the Department of Public Health, Water Supply Section (860) 509-7333 and Recreation Section (860) 509-7297, and the local Director of Health shall be notified within 2 hours of the permittee learning of the event by telephone during normal business hours. If the discharge or bypass occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday), notification shall be made within 2 hours of the permittee learning of the event to the Emergency Response Unit at (860) 424-3338 and the Department of Public Health at (860) 509-8000. A written report shall be submitted to the Department of Energy and Environmental Protection, Bureau of Water Protection and Land Reuse, Planning and Standards Division, Municipal Facilities Section within five days of the permittee learning of each occurrence, or potential occurrence, of a discharge or bypass of untreated or partially treated sewage.

The written report shall contain:

(i) The nature and cause of the bypass, permit violation, treatment component failure, and/or equipment failure,

(ii) the time the incident occurred and the anticipated time which it is expected to continue or, if the condition has been corrected, the duration,

(iii) the estimated volume of the bypass or discharge of partially treated or raw sewage,

(iv) the steps being taken to reduce or minimize the effect on the receiving waters, and

(v) the steps that will be taken to prevent reoccurrence of the condition in the future.

(D) Section 22a-430-3(j) 11 (D) of the RCSA shall apply in the event of any noncompliance with a maximum daily limit and/or any noncompliance that is greater than two times any permit limit. The permittee shall notify in the same manner as in paragraph C of this Section, the Department of Energy and Environmental Protection, Bureau of Water Protection and Land Reuse Planning and Standards Division, Municipal Facilities Section except, if the noncompliance occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday) the permittee may wait to make the verbal report until 10:30 am of the next business day after learning of the noncompliance.

(E) Section 22a-430-3(j) 8 of the RCSA shall apply in all instances of monitoring equipment failures that prevent meeting the requirements in this permit. In the event of any such failure of the monitoring equipment including, but not limited to, loss of refrigeration for an auto-sampler or lab refrigerator or loss of flow proportion sampling ability, the permittee shall notify in the same manner as in paragraph C of this Section, the Department of Energy and Environmental Protection, Bureau of Water Protection and Land Reuse, Planning and Standards Division, Municipal Facilities Section except, if the failure occurs outside normal working hours (8:30 a.m. to 4:30 p.m. Monday through Friday) the permittee may wait to make the verbal report until 10:30 am of the next business day after learning of the failure.

(F) In addition to the reporting requirements contained in Section 22a-430-3(i), (j), and (k) of the Regulations of Connecticut State Agencies, the permittee shall notify in the same manner as in paragraph C of this Section, the Department of Energy and Environmental Protection, Bureau of Water Protection and Land Reuse, Planning and Standards Division, Municipal Facilities Section concerning the failure of any major component of the treatment facilities which the permittee may have reason to believe would result in an effluent violation.

SECTION 9: COMPLIANCE SCHEDULES

(A) The permittee shall achieve the final water quality-based effluent limits for **phosphorus** for DSN 001-1 established in Section 5 of this permit, in accordance with the following:

(1) The permittee has retained AECOM Technical Services, Inc., to prepare the documents and implement or oversee the actions required by this permit. The permittee shall retain one or more qualified consultant(s) acceptable to the Commissioner until this permit is fully complied with, and, within ten days after retaining any consultant other than the one originally identified under this paragraph, the permittee shall notify the Commissioner in writing of the identity of such other consultant. The consultant(s) retained shall be a qualified professional engineer licensed to practice in Connecticut. The permittee shall submit to the Commissioner a description of a consultant's education, experience and training which is relevant to the work required by this permit within ten days after a request for such a description. Nothing in this paragraph shall preclude the Commissioner from finding a previously acceptable consultant unacceptable.

(2) On or before 240 days after the date of issuance of this permit, the permittee shall submit for the Commissioner's review and written approval a comprehensive and thorough engineering report which describes and evaluates alternative actions as outlined below which may be taken by permittee to achieve compliance with the Phosphorus limitations in Section 5 of this permit. Such report shall contain at a minimum:

(a) A summary of existing facilities, service areas, and conditions in Sewer District 1 and 2;

(b) The findings of the in-plant sampling efforts;

(c) An evaluation of existing flows and loadings, including an assessment of the magnitude of Inflow & Infiltration in Sewer Districts 1 and 2;

(d) A summary of findings of the smoke testing and recommendations for subsequent dye testing and due water flooding;

(e) Identification of areas where extension of the collection system is recommended;

(f) An updated review of the Route 7 WWTF Influent Pump Station and Quail Ridge Pump Station condition and summary of their needs;

(g) A summary of the evaluation of the Sewer District 1 collection system bottlenecks and recommendations;

(h) A projection of anticipated future flows and loadings in Sewer District 1 and Sewer District 2;

(i) An assessment of the two (South Street and Route 7) WWTFs capacities, identification of the treatment processes limiting the capacities, consideration of modifications to remove the capacity limitations, and an assessment of the potential to rerate the WWTFs capacity to a higher level;

(j) An assessment of the feasibility of land application of a portion of the effluent from the South Street WWTF; and,

(k) A Phase 2 Scope of Work will be prepared and submitted to include the items listed below in Section 9 (A) (3).

(3) On or before 730 days after the date of issuance of this permit, the permittee shall submit for the Commissioner's review and written approval a comprehensive and thorough engineering report which describes and evaluates alternative actions which may be taken by permittee to achieve compliance with the Phosphorus limitations in Section 5 of this permit. Such report shall:

(a) List all permits and approvals required for each alternative, including but not limited to any permits required under Sections 22a-32, 22a-42a, 22a-342, 22a-361, 22a-368 or 22a-430 of the CGS,

(b) Propose a preferred alternative or combination of alternatives with supporting justification therefore,

(c) State in detail the most expeditious schedule for performing each alternative, and

(l) Propose a detailed program and schedule to perform all actions required to implement the preferred alternative, including but not limited to a schedule for submission of engineering plans and specifications for any new equipment, the start and completion of any construction activities and applying for and obtaining all permits and approvals required for such actions.

(4) Unless another deadline is specified in writing by the Commissioner, on or before 180 days after approval of the engineering report in Section 9(A)(3), the permittee shall (1) submit for the Commissioner's review and written approval, contract plans and specifications for the approved remedial actions, a revised list of all permits and approvals required for such actions and a revised schedule for applying for and obtaining such permits and approvals; and (2) submit applications for all permits and approvals required under Sections 22a-430 and 22a-416 of the CGS. The permittee shall obtain all required permits and approvals.

(5) In accordance with the schedule approved in writing by the Commissioner, but in no event later than 1,800 days after the issuance of this permit, the permittee shall complete the actions approved in writing by the Commissioner necessary to comply with the requirements for phosphorus in Table A of this permit. Within fifteen days after completing such actions, the permittee shall certify to the Commissioner in writing that such actions, as required by this paragraph, have been completed.

(B) The permittee shall achieve the final water quality-based effluent limits for **Escherichia coli** for DSN 001-1 established in Section 5 of this permit, in accordance with the following:

(1) On or before 300 days after the date of issuance of this permit, the permittee shall submit for the Commissioner's review and written approval a comprehensive and thorough report which describes the actions to be taken by the permittee necessary to achieve compliance with the requirements in Table A of this permit for Escherichia coli. Such report shall include a schedule for implementation of such actions not to exceed 730 days after the date of issuance of this permit.

(2) In accordance with the schedule approved in writing by the Commissioner, but in no event later than 730 days after the date of issuance of this permit, the permittee shall perform the actions approved in writing by the Commissioner necessary to comply with the requirements in Table A of this permit for Escherichia coli. Within fifteen days after completing such actions, the permittee shall certify to the Commissioner in writing that the actions have been completed as approved by the Commissioner.

(C) The permittee shall submit to the Commissioner semi-annual status reports beginning sixty days after the date of approval of the either report referenced in paragraphs 9(A) and 9(B) of this Section. Status reports shall include, but not be limited to, a detailed description of progress made by the permittee in performing actions required by this Section of the permit in accordance with the approved schedule including, but not limited to, development of engineering plans and specifications, construction activity, contract bidding, operational changes, preparation and submittal of permit applications, and any other required under paragraph(s) 9(A) and 9(B) of this Section.

(D) The permittee shall use best efforts to submit to the Commissioner all documents required by this Section of the permit in a complete and approvable form. If the Commissioner notified the permittee that any document or other action is deficient, and does not approve it with conditions or modifications, it is deemed disapproved, and the permittee shall correct the deficiencies and resubmit it within the time specified by the Commissioner or, if no time is specified by the Commissioner, within thirty days of the Commissioner's notice of deficiencies. In approving any document or other action under this Compliance Schedule, the Commissioner may approve the document or other action as submitted or performed or with such conditions or modifications as the Commissioner deems necessary to carry out the purposes of this Section of the permit. Nothing in this paragraph shall excuse noncompliance or delay.

(E) Dates. The date of submission to the Commissioner of any document required by this section of the permit shall be the date such document is received by the Commissioner. The date of any notice by the Commissioner under this section of the permit, including but not limited to notice of approval or disapproval of any document or other action, shall be the date such notice is personally delivered or the date three days after it is mailed by the Commissioner, whichever is earlier. Except as otherwise specified in this permit, the word "day" as used in this Section of the permit means calendar day. Any document or action which is required by this Section only of the permit, to be submitted, or performed, by a date which falls on, Saturday, Sunday, or, a Connecticut or federal holiday, shall be submitted or performed on or before the next day which is not a Saturday, Sunday, or Connecticut or federal holiday.

(F) Notification of noncompliance. In the event that the permittee becomes aware that it did not or may not comply, or did not or may not comply on time, with any requirement of this Section of the permit or of any document required hereunder, the permittee shall immediately notify the Commissioner and shall take all reasonable steps to ensure that any noncompliance or delay is avoided or, if unavoidable, is minimized to the greatest extent possible. In so notifying the Commissioner, the permittee shall state in writing the reasons for the noncompliance or delay and propose, for the review and written approval of the Commissioner, dates by which compliance will be achieved, and the permittee shall comply with any dates which may be approved in writing by the Commissioner. Notification by the permittee shall not excuse noncompliance or delay, and the Commissioner's approval of any compliance dates proposed shall not excuse noncompliance or delay unless specifically so stated by the Commissioner in writing.

(G) Notice to Commissioner of changes. Within fifteen days of the date the permittee becomes aware of a change in any information submitted to the Commissioner under this Section of the permit, or that any such information was inaccurate or misleading or that any relevant information was omitted, the permittee shall submit the correct or omitted information to the Commissioner.

(H) Submission of documents. Any document, other than a DMR, ATMR or MOR required to be submitted to the Commissioner under this Section of the permit shall, unless otherwise specified in writing by the Commissioner, be directed to:

Carlos Esguerra, Sanitary Engineer 2
Department of Energy and Environmental Protection
Bureau of Water Protection and Land Reuse, Planning and Standards Division
79 Elm Street
Hartford, Connecticut 06106-5127

This permit is hereby issued on September 18, 2014.

Betsey Wingfield
Bureau Chief
Bureau of Water Protection and Land Reuse

ATTACHMENT 1

Tables A through F

TABLE A

Discharge Serial Number (DSN): **001-1** | Monitoring Location: **1**

Wastewater Description: **Sanitary Sewage**

Monitoring Location Description: **Final Effluent**

Allocated Zone of Influence (ZOI): **0.70 cfs** | In-stream Waste Concentration (IWC): **20.97 %**

PARAMETER	Units	FLOW/TIME BASED MONITORING				INSTANTANEOUS MONITORING			REPORT FORM	Minimum Level Analysis See Section 6
		Average Monthly Limit	Maximum Daily Limit	Sample Freq.	Sample type	Instantaneous Limit or Required Range [3]	Sample Freq.	Sample Type		
Alkalinity	mg/l	NA	NA	NR	NA	------	Monthly	Grab	MOR	
Biochemical Oxygen Demand (5 day) [1] See remark (E) below	mg/l	20	40	Weekly	Daily Composite	NA	NR	NA	DMR/MOR	
Fecal coliform May 1st through September 30th [4]	Colonies per100 ml	NA	NA	NR	NA	see remarks (B) and (C) below	Weekly	Grab	DMR/MOR	
Escherichia coli May 1st through September 30th [5] See remark (D) below.	Colonies per100 ml	NA	NA	NR	NA	410	Weekly	Grab	DMR/MOR	
Flow	MGD	------	------	Continuous[2]	Average Daily Flow	NA	NR	NA	DMR/MOR	
Nitrogen, Ammonia (total as N):										
June	mg/l	6.7	------	Weekly	Daily Composite	NA	NR	NA	DMR/MOR	
July, August and September		2.5	------	Weekly						
October		4.4	------	Weekly						
November through May		------	------	Monthly						
Nitrogen, Nitrate (total as N)	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	MOR	
Nitrogen, Nitrite (total as N)	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	MOR	
Nitrogen, Total Kjeldahl	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	MOR	
Nitrogen, Total	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	MOR	
Nitrogen, Total	lbs/day	NA	------	Monthly	Daily Composite	NA	NR	NA	MOR	
Oxygen, Dissolved	mg/l	NA	NA	NR	NA	------	Work Day	Grab	MOR	
pH	S.U.	NA	NA	NR	NA	6 - 9	Weekly	Grab	DMR/MOR	
Phosphate, Ortho	mg/l	NA	------	Monthly	Daily Composite	NA	NR	NA	MOR	

Phosphorus, Total. April 1st through October 31st [6] April 1st through October 31st November 1st through March 30th	mg/l	1.55 NA	3.11 ------	Weekly Monthly	Daily Composite	NA	NR	NA	DMR/MOR	*
Phosphorus, Total April 1st through October 31st	lbs/day	------	------	Weekly	Daily Composite	NA	NA	NA	MOR	
Phosphorus, Total (Average Seasonal Load Cap)[7] October	lbs/day	------	NA	Weekly	Calculated	NA	NA	NA	DMR/MOR	
Solids, Settleable	ml/l	NA	NA	NR	NA	------	Work Day	Grab	MOR	
Solids, Total Suspended[1] See remark (E) below	mg/l	20	40	Weekly	Daily Composite	NA	NA	NA	DMR/MOR	
Temperature	°F	NA	NA	NR	NA	------	Work Day	Grab	MOR	
Turbidity	NTU	NA	NA	NR	NA	------	Work Day	Grab	MOR	
UV Intensity May 1st through September 30th. See remark A	mW/cm^2	NA	NA	NR	NA	------	2/Work Day	Grab	DMR/MOR	

TABLE A – CONDITIONS

Footnotes:

[1] The discharge shall not exceed an average monthly 20 mg/l or a maximum daily 40 mg/l.

[2] The permittee shall record and report on the monthly operating report the minimum, maximum and total flow for each day of discharge and the average daily flow for each sampling month. The permittee shall report, on the discharge monitoring report, the average daily flow and maximum daily flow for each sampling month.

[3] The instantaneous limits in this column are maximum limits.

[4] During the period beginning at the date of issuance of this permit and lasting until the implementation of Escherichia coli monitoring at the Water Pollution Control Facility, the discharge shall not exceed and shall otherwise conform to specific terms and conditions listed.

[5] During the period beginning after the implementation of Escherichia coli monitoring, but no later than **730 days** after permit issuance, lasting until expiration, the discharge shall also not exceed and shall otherwise conform to the specific terms and conditions listed.

[6] During the period beginning after the implementation of phosphorus removal but no later than **1,800 days** after permit issuance, lasting until expiration, the discharge shall also not exceed and shall otherwise conform to the specific terms and conditions listed.

[7] During the period beginning after the implementation of phosphorus removal but no later than **1,800 days** after permit issuance, lasting until expiration, the discharge shall not exceed the total phosphorus Average Seasonal Load of 1.0 lb/day. Calculate the Average Seasonal Load by adding all sample results during each April 1st through October 31st season in pounds per day and dividing by the total number of those samples in that season.

Remarks:

(A) Ultraviolet disinfection shall be utilized from May 1st through September 30th.

(B) The geometric mean of the Fecal coliform bacteria values for the effluent samples collected in a period of a calendar month during the period from May 1st through September 30th shall not exceed 200 per 100 milliliters.

Remarks. (Continued)

(C) The geometric mean of the Fecal coliform bacteria values for the effluent samples collected in a period of a <u>calendar week</u> during the period from May 1st through September 30th shall not exceed 400 per 100 milliliters.

(D) The geometric mean of the Escherichia coli bacteria values for the effluent samples collected in a period of a <u>calendar month</u> during the period from May 1st through September 30th shall not exceed 126 per 100 milliliters.

(E) The Average Weekly discharge Limitation for BOD_5 and Total Suspended Solids shall be 1.5 times the Average Monthly Limit listed above.

TABLE B

Discharge Serial Number (DSN): **001-1**					Monitoring Location: **K**
Wastewater Description: **Sanitary Sewage**					
Monitoring Location Description: **Final Effluent**					
Allocated Zone of Influence (ZOI): **0.70 cfs**		In-stream Waste Concentration (IWC): **20.97 %**			
PARAMETER	**Units**	**FLOW/TIME BASED MONITORING**			**REPORT FORM**
		Average Monthly Minimum	**Sample Freq.**	**Sample type**	
Biochemical Oxygen Demand (5 day) Percent Removal [1]	% of Influent	90	Weekly	Calculated[2]	DMR/MOR
Solids, Total Suspended Percent Removal [1]	% of Influent	90	Weekly	Calculated[2]	DMR/MOR

TABLE B – CONDITIONS

Footnotes:

[1] The discharge shall be less than or equal to 10% of the average monthly influent BOD_5 and total suspended solids (Table E, Monitoring Location G).

[2] Calculated based on the average monthly results described in Table A. Removal efficiency = $\frac{\text{Inf.BOD or TSS} - \text{Effluent BOD or TSS}}{\text{Inf.BOD or TSS}}$ X 100

TABLE C

Discharge Serial Number (DSN): **001-1**				Monitoring Location: **T**		
Wastewater Description: **Sanitary Sewage**						
Monitoring Location Description: **Final Effluent (after completion of UV disinfection)**						
Allocated Zone of Influence (ZOI): **0.70 cfs**			In-stream Waste Concentration (IWC): **20.97 %**			
PARAMETER	**Units**	**Maximum Daily Limit**	**Sampling Frequency**	**Sample Type**	**Reporting form**	**Minimum Level Analysis See Section 6**
Aluminum, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Antimony, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
NOAEL Static 48Hr Acute D. Pulex[1]	% survival	$\geq$90%	Semi-annually	Daily Composite	ATMR/DMR	
NOAEL Static 48Hr Acute Pimephales[1]	% survival	$\geq$90%	Semi-annually	Daily Composite	ATMR/DMR	
Arsenic, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Beryllium, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
BOD_5	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Cadmium, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Chromium, Hexavalent	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Chromium, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Chlorine, Total Residual	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Copper, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Cyanide, Amenable	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Cyanide, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Iron, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Lead, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Mercury, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Nickel, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Nitrogen, Ammonia (total as N)	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Nitrogen, Nitrate, (total as N)	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Nitrogen, Nitrite, (total as N)	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Phosphorus, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Phenols, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Selenium, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Silver, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Suspended Solids, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	
Thallium, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*
Zinc, Total	mg/l	------	Semi-annually	Daily Composite	ATMR/DMR	*

TABLE C - CONDITIONS

Remarks:[1]The results of the Toxicity Tests are recorded in % survival. The permittee shall report <u>% survival</u> on the DMR based on criteria in Section 6(B) of this permit.

ATMR – Aquatic Toxicity Monitoring Report

TABLE D

Discharge Serial Number: **001-1**			Monitoring Location: **G**				
Wastewater Description: **Sanitary Sewage**							
Monitoring Location Description: **Influent**							
PARAMETER	**Units**	**DMR REPORTING FORMAT**	**FLOW/TIME BASED MONITORING**		**INSTANTANEOUS MONITORING**		**REPORTING FORM**
			Sample Frequency	**Sample Type**	**Sample Frequency**	**Sample Type**	
Biochemical Oxygen Demand (5 day)	mg/l	Monthly average	Weekly	Daily Composite	NA	NA	DMR/MOR
Nitrogen, Ammonia (total as N)	mg/l		Monthly	Daily Composite	NA	NA	MOR
Nitrogen, Nitrate (total as N)	mg/l		Monthly	Daily Composite	NA	NA	MOR
Nitrogen, Nitrite (total as N)	mg/l		Monthly	Daily Composite	NA	NA	MOR
Nitrogen, Total Kjeldahl	mg/l		Monthly	Daily Composite	NA	NA	MOR
Nitrogen, Total	mg/l		Monthly	Daily Composite	NA	NA	MOR
Phosphate, Ortho	mg/l		Monthly	Daily Composite	NA	NA	MOR
Phosphorus, Total	mg/l		Monthly	Daily Composite	NA	NA	MOR
pH	S.U.		NA	NA	Work Day	Grab	MOR
Solids, Total Suspended	mg/l	Monthly average	Weekly	Daily Composite	NA	NA	DMR/MOR
Temperature	°F		NA	NA	Work Day	Grab	MOR

TABLE E

Discharge Serial Number: **001-1**					Monitoring Location: **P**			
Wastewater Description: **Primary Effluent**								
Monitoring Location Description: **Primary Sedimentation Basin Effluent**								
PARAMETER	**Units**	**REPORTING FORMAT**	**TIME/FLOW BASED MONITORING**		**INSTANTANEOUS MONITORING**		**REPORTING FORM**
			Sample Frequency	**Sample Type**	**Sample Frequency**	**Sample type**	
Alkalinity, Total	mg/l		NA	NA	Monthly	Grab	MOR
Biochemical Oxygen Demand (5 day)	mg/l	Monthly average	Weekly	Composite	NA	NA	MOR
Nitrogen, Ammonia (total as N)	mg/l		Monthly	Composite	NA	NA	MOR
Nitrogen, Nitrate (total as N)	mg/l		Monthly	Composite	NA	NA	MOR
Nitrogen, Nitrite (total as N)	mg/l		Monthly	Composite	NA	NA	MOR
Nitrogen, Total Kjeldahl	mg/l		Monthly	Composite	NA	NA	MOR
Nitrogen, Total	mg/l		Monthly	Composite	NA	NA	MOR
pH	S.U.		NA	NA	Monthly	Grab	MOR
Solids, Total Suspended	mg/l	Monthly average	Weekly	Composite	NA	NA	MOR

TABLE F

Discharge Serial Number: **001-1**	Monitoring Location: **SL**		
Wastewater Description: **Thickened Sludge**			
Monitoring Location Description: **Thickened Sludge**			
PARAMETER	**INSTANTANEOUS MONITORING**		**REPORTING FORM**
	Units	**Grab Sample Freq.**	
Arsenic, Total	mg/kg	Annual	DMR
Beryllium, Total	mg/kg	Annual	DMR
Cadmium, Total	mg/kg	Annual	DMR
Chromium, Total	mg/kg	Annual	DMR
Copper, Total	mg/kg	Annual	DMR
Lead, Total	mg/kg	Annual	DMR
Mercury, Total	mg/kg	Annual	DMR
Nickel, Total	mg/kg	Annual	DMR
Nitrogen, Ammonia *	mg/kg	Annual	DMR*
Nitrogen, Nitrate (total as N) *	mg/kg	Annual	DMR*
Nitrogen, Organic *	mg/kg	Annual	DMR*
Nitrogen, Nitrite (total as N) *	mg/kg	Annual	DMR*
Nitrogen, Total *	mg/kg	Annual	DMR*
pH *	S.U.	Annual	DMR*
Polychlorinated Biphenyls	mg/kg	Annual	DMR
Solids, Fixed	%	Annual	DMR
Solids, Total	%	Annual	DMR
Solids, Volatile	%	Annual	DMR
Zinc, Total	mg/kg	Annual	DMR

(*) required for composting or land application only
Testing for inorganic pollutants shall follow "Test Methods for Evaluating Solid Waste, Physical/Chemical Methods", EPA Publication SW-846 as updated and/or revised.

ATTACHMENT 2

MONTHLY OPERATING REPORT FORM

Ridgefield Route 7 WPCF Permit No. CT0101451

Sample month/year:

	Daily Flow			Primary Sludge			Waste sludge	BOD (5-day)			Suspended Solids			Settleable Solids	Turbidity	UV Intensity		Fecal Coliform	E. Coli	Ammonia			Nitrite			Nitrate		
	Max.	Min.	Total	Vol.	%	wt.		Inf.	Prim. Eff.	Final Eff.	Inf.	Prim. Eff.	Final Eff.	Eff.	Eff.	1	2			Inf.	Prim. Eff.	Final Eff.	Inf.	Prim. Eff.	Final Eff.	Inf.	Prim. Eff.	Final Eff.
Units	mgd			gal.	solids	lbs.	lbs	mg/l			mg/l			ml/l	NTU	mW/cm2		#/100 ml	#/100 ml	mg/l			mg/l			mg/l		
Freq	daily			work day			work day	Weekly			Weekly			work day	work day	4/work day		Weekly	Weekly	Monthly			Monthly			Monthly		
1																												
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Total																												
Ave																												
Limit																												

TKN Inf.	TKN Prim. Eff.	TKN Final Eff.	Total N Inf.	Total N Prim. Eff.	Total N Final Eff.	Total N Eff.	Lowest D.O. Eff.	pH Inf.	pH Eff.	Total P Inf.	Total P Prim. Eff.	Total P Final Eff.	Total P Eff.	Ortho P Inf.	Ortho P Prim. Eff.	Ortho P Final Eff.	Temp. Inf.	Temp. Eff.	Alkalinity Prim. Eff.	Alkalinity Eff.
mg/l			mg/l			lb/d	mg/l	S.U.		mg/l			lb/d	mg/l			°F			
Monthly			Monthly			monthly	4 per workday	work day		Monthly			April - October	Monthly			work day		Monthly	

Sludge Disposal Location:

Please return forms to:
DEEP - Water Bureau
ATTN: Municipal Wastewater Monitoring Coordinator
Municipal Facilities
79 Elm Street
Hartford, CT 06106-5127

Statement of Acknowledgement

I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information including the possibility of fine and imprisonment for knowing violations.

Authorized Official:

Title:

Signature: ____________________

Date: __________

DATA TRACKING AND TECHNICAL FACT SHEET

Permittee: Town of Ridgefield

PERMIT, ADDRESS, AND FACILITY DATA

PERMIT #: CT0101451 **APPLICATION #:** 200900547 **FACILITY ID.** 118-002

Mailing Address:	Location Address:
Street: 66 Prospect Street	Street: 9101 Ethan Allen Hwy (Route 7)
City: Ridgefield ST: CT Zip: 06877	City: Ridgefield ST: CT Zip: 06877
Contact Name: Amy Seibert	Contact Name: Jorge Pereira, United Water
Phone No.: 203-431-2734	Phone No.: 203-438-8615
E-mail:	DMR Contact email address: Jorge.pereira@unitedwater.com

PERMIT INFORMATION

DURATION 5 YEAR X 10 YEAR ___ 30 YEAR ___

TYPE New _ Reissuance X Modification __

CATEGORIZATION POINT (X) NON-POINT ( ) GIS #

NPDES (X) PRETREAT () GROUND WATER(UIC) ( ) GROUND WATER (OTHER) ( )

NPDES MAJOR(**MA**) ___
NPDES SIGNIFICANT MINOR or PRETREAT SIU **(SI)** X
NPDES or PRETREATMENT MINOR **(MI)** ____

COMPLIANCE SCHEDULE YES X NO__
POLLUTION PREVENTION __ TREATMENT REQUIREMENT__
WATER QUALITY REQUIREMENT X OTHER ___

OWNERSHIP CODE
Private __ Federal __ State __ Municipal (town only) X Other public__

DEP STAFF ENGINEER Carlos Esguerra **DATE DRAFTED:** 4/23/2014

PERMIT FEES

Discharge Code	DSN Number	Annual Fee
111000c	00-1	1,722.50

FOR NPDES DISCHARGES
Water Quality Classification Goal: B
Segment: Norwalk River - 7300

NATURE OF BUSINESS GENERATING DISCHARGE
Municipal Sanitary Sewage Treatment

PROCESS AND TREATMENT DESCRIPTION (by DSN)
Secondary Municipal wastewater treatment plant with nitrification/denitrification and seasonal UV disinfection.

RESOURCES USED TO DRAFT PERMIT

__ *Federal Effluent Limitation Guideline* 40CFR 133 *Secondary Treatment Category*

__ *Performance Standards*

_ *Federal Development Document*

name of category

_X_ *Department File Information*

_X_ *Connecticut Water Quality Standards*

_X_ *Anti-degradation Policy*

_ *Coastal Management Consistency Review Form*

_X_ *Other – Explain: Interim Phosphorus Management Strategy*

BASIS FOR LIMITATIONS, STANDARDS OR CONDITIONS

_X_ *Secondary Treatment (Section 22a-430-4(r) of the Regulations of Connecticut State Agencies)*

_ *Case-by-Case Determination (See Other Comments)*

_X_ *In order to meet in-stream water quality (See General Comments)*

_ *Anti-degradation policy*

GENERAL COMMENTS

The Town of Ridgefield ("Ridgefield") operates a municipal water pollution control facility ("the facility") located at 9101 Ethan Allen Highway (Route 7) in Ridgefield, CT. The facility is designed to treat and discharge up to 0.12 million gallons a day of effluent into Norwalk River. The facility currently uses secondary treatment with seasonal UV disinfection to treat effluent before being discharged. Pursuant to Conn. Gen. Stat. § 22a-430, the Department of Energy and Environmental Protection has issued Ridgefield a permit for the discharge from this facility. Ridgefield has submitted an application to renew its permit. The Department has made a tentative determination to approve Ridgefield's application and has prepared a draft permit consistent with that determination.

The most significant changes from the current permit are the inclusion of phosphorus limits, revised bacteria monitoring requirements (e. coli), Aluminum monitoring to be consistent with the most recent CT Water Quality Standards and Iron monitoring to be consistent with EPA's National Recommended Water Quality Criteria.

SPECIFIC REQUIREMENTS OR REVISIONS

The Department reviewed the application for consistency with Connecticut's Water Quality Standards and determined that with the limits in the draft permit, including those discussed below, that the draft permit is consistent with maintenance and protection of water quality in accordance with the Tier I Anti-degradation Evaluation and Implementation Review provisions of such Standards.

The need for inclusion of water quality based discharge limitations in this permit was evaluated consistent with Connecticut Water Quality Standards and criteria, pursuant to 40 CFR 122.44(d). Discharge monitoring data was evaluated for consistency with the available aquatic life criteria (acute and chronic) and human health (fish consumption only) criteria, considering the zone of influence allocated to the facility where appropriate. In addition to this review, the statistical procedures outlined in the EPA Technical Support Document for Water Quality-based Toxics Control (EPA/505/2-90-001) were employed to calculate the need for such limits. Comparison of the attached monitoring data and its inherent variability with the calculated water quality based limits indicates a low statistical probability of exceeding such limits. Therefore, no water quality based limits were included in the permit at this time.

A compliance schedule is included for the reduction of phosphorus in the effluent. The town has retained the services of AECOM Technical Services, Inc., to conduct a facilities planning evaluation of the two wastewater treatment facilities in Ridgefield. This evaluation is being conducted in two phases: the phase I evaluation is currently underway with a report completion date targeted for the end of 2014; and initiation of the phase II study is expected to start in the second quarter of 2015. The town, as part of the phase II assessment, will look into the possibility of decommissioning the Route 7 plant with conveyance of flows to the main Ridgefield plant. For this *reason, the draft permit includes an extended compliance timeframe (of up to 1,800 days) for the implementation of phosphorus monitoring which will allow the town to complete the above-referenced studies and adopt a plan to address the new monitoring requirements.*

Phosphorus Permitting Approach

Phosphorus is a naturally occurring element that is essential to support plant growth. When present in excessive amounts, phosphorus can impair both aquatic life and recreational use of Connecticut's water resources. Excess nutrient enrichment is a serious threat to water quality in Connecticut. Excessive loading of phosphorus to surface waters as a result of discharges from wastewater treatment plants or non point sources such as runoff from urban and agricultural lands, can lead to algal blooms, including blooms of noxious blue green algae, reduction in water clarity, and in extreme cases depletion of oxygen, fish kills, and other impairments to aquatic life. Currently, 21 water body segments have been identified on Connecticut's List of Waters Not Meeting Water Quality Standards where nutrient enrichment is a contributing cause of the impairment.

The Connecticut Water Quality Standards (WQS) do not include numeric criteria for nutrients but rather incorporate narrative standards and criteria for nutrients. These narrative policy statements direct the Connecticut Department of Environmental Protection to impose discharge limitations or other reasonable controls on point and non point sources to support maintenance or attainment of designated uses. In the absence of numeric criteria for phosphorus, the Department has developed an interim nutrient management strategy for freshwater non-tidal streams based on the narrative policy statements in the WQS to meet the pressing need to issue NPDES permits and be protective of the environment. The strategy includes methods that focus on phosphorus because it is the primary limiting nutrient in freshwater systems. These methods were approved by the United States Environmental Protection (EPA) in their letter dated October 26, 2010 as an interim strategy to establish water quality based phosphorus limits in non-tidal freshwater for industrial and municipal water pollution control facilities (WPCFs) national pollutant discharge elimination system (NPDES) permits.

The method in the interim strategy uses best available science to identify phosphorus enrichment levels in waste receiving rivers and streams that adequately support aquatic life uses. The methodology focuses on algal communities as the key aquatic life nutrient response variable and phosphorus enrichment factors that represent significant changes in communities based on data collected statewide. Ongoing work is currently being conducted to refine the approach through additional data collection and by expanding the methodology to include non-waste receiving streams. It is expected that the ongoing work will lead to numeric nutrient criteria for all freshwater rivers and streams in the next WQS review cycle. The current approach provides for a major statewide advancement in the level of phosphorus control that is expected to meet all freshwater designated uses. The adaptive nature of Connecticut's strategy allows for revisions to permit limits in future permit cycles without delaying action that we know needs to be taken today.

The current approach follows a watershed based framework incorporating many of the elements from the U.S. EPA Watershed –Based National Pollutant Discharge Elimination System (NPDES) Permitting Technical Guidance (2007). Consistent with the 2007 Guidance, the approach "explicitly considers the impact of multiple pollutant sources and stressors, including nonpoint source contributions, when developing point source permits". Expected current conditions are based on the probability of excess phosphorus export from land cover and municipal and industrial facilities in the upstream drainage basin. Connecticut's policy for phosphorus management is translated into a numeric expression through geo-spatial and statistical analyses that determines the maximum acceptable seasonal phosphorus mass load per unit area of watershed contributing flow to the point of assessment.

The goal of the interim strategy is to achieve or maintain an enrichment factor (EF) of 8.4 or below throughout a watershed. An EF is representative of the amount of anthropogenic phosphorus loading to river and streams. It is calculated by dividing the current total seasonal phosphorus load by a modeled total phosphorus load under complete forested conditions at a particular point along the river. An enrichment factor is representative of the amount of anthropogenic phosphorus loading to rivers and streams. The goal of an 8.4 enrichment factor represents a threshold at which a significant change is seen in the algal communities indicating highly enriched conditions and impacts to aquatic life uses.

The analysis was conducted using benthic algae collected in rivers and streams throughout CT under varying enrichment conditions. The approach targets the critical 'growing' season (April through October) when phosphorus is more likely to be taken up by sediment and biomass because of low flow and warmer conditions. During winter months aquatic plants are dormant and flows are higher providing constant flushing of phosphorus *through aquatic systems with a less likely chance that it will settle out into the sediment. Limiting the phosphorus export from industrial and municipal facilities offers a targeted management strategy for achieving aquatic life designated uses within a waterbody. The export of some phosphorus from facilities and other land sources is considered normal use of the land recognizing that humans are part of the environment.*

A seasonal load was established by the Department for each facility discharging to non-tidal waters based on the current degree of enrichment of the receiving water body at the point of discharge and the facilities contribution to the total watershed enrichment at the point of discharge.

Town of Ridgefield (Rte. 7) Permit Requirements
A nutrient watershed analysis was conducted for the Norwalk River watershed below facilities discharging phosphorus into the river. The facilities discharging to this river include: Ridgefield (Rte. 7) WPCF, Ridgefield (Main) WPCF and Georgetown (Redding) WPCF. The seasonal (April 1st through October 31st) nutrient loading from each facility discharging to the watershed was reduced to achieve an enrichment factor of 8.4 or lower throughout the river.

The current enrichment factor at the Town of Ridgefield (Rte. 7) discharge is 24.2. The final proposed seasonal load allocation for Town of Ridgefield (Rte. 7) POTW is 1 lbs/day. This load equates to a proposed treatment performance limit of 1 mg/L multiplied by the current seasonal average flow of 0.12 MGD. When this strategy is fully implemented by combining reductions at all facilities located in the same watershed, it is expected that an enrichment factor below 8.4 will be achieved in the Naugatuck River.

Federal regulations at 40 CFR 122.44(d) indicate that permit issuers are required to determine whether a given point source discharge causes, has the reasonable potential to cause, or contributes to an in-stream excursion above a narrative or numeric criteria within a State water quality standard after consideration of existing controls on point and non-point sources of pollution. If a discharge is found to cause an excursion of a numeric or narrative state water quality criterion, NPDES regulations implementing section 301(b)(1)(C) of the Clean Water Act provide that a permit must contain effluent limits as necessary to achieve state water quality standards. The limit in the permit and the strategy are consistent with the narrative policy statements in the CT WQS and are expected to result in the attainment and maintenance of all designated uses for the water body when the strategy is fully implemented. If the Department develops numeric criteria in the future, or it is found that the current limit under the strategy is not sufficient to achieve designated uses, the goal will be modified and the WPCF will be expected to meet the more stringent water quality goal.

Translating the average performance level of 1 lb/day into enforceable permit limits requires consideration of effluent variability and frequency of monitoring in order to comply with federal permitting regulations. The procedure used is as follows:

1. Consider the permit performance level 1.0 mg/L to be equivalent to the Long Term Average (LTA)

2. Calculate the Maximum Daily Limit by multiplying the LTA by the 99th percentile LTA Multiplier appearing in Table 5-2 of the Technical Support Document (page 103 of EPA/505/2-90-001) corresponding to a CV of 0.6% to account for effluent variability:

*Maximum Daily Limit: 1 mg/L * 3.11 = 3.11 mg/L*

3. Calculate the Average Monthly Limit by multiplying the LTA by the 95th percentile LTA Multiplier appearing in Table 5-2 of the Technical Support Document corresponding to a CV of 0.6% to account for effluent variability and either n=4 samples/month or n=10 samples/month as appropriate for the facility to account for the precision of estimating the true monthly average based on an average for the days the effluent was sampled:

Average Monthly Limit= 1 mg/l X 1.55 = 1.55 mg/l

Summary of Limits for Ridgefield (Rte. 7) POTW:
Average Daily Load = 1 lb/day
Total Seasonal Load = 1 lbs/day * 214 Days/Season = 214 lbs/season
Maximum Daily Limit = 3.11 mg/L

APPENDIX C

TECHNICAL MEMORANDUM NO. 7 – FUTURE FLOWS AND LOADS

AECOM
701 Edgewater Drive
Wakefield, MA 01880
www.aecom.com

781 246 5200 tel
781 224 6676 fax

Draft Technical Memorandum No. 7

To	Ridgefield WPCA	Page	1 of 25
CC	C. Fisher, J. O'Brien, J. Pereira, J. Pennell		
Subject	Town of Ridgefield, CT Phase 1 Wastewater Facilities Plan Draft Technical Memorandum No. 7 – Future Flows and Loads		
From	Jon Pearson/Alberto Angles		
Date	March 26, 2015		

INTRODUCTION

The Town of Ridgefield Water Pollution Control Authority (WPCA) owns and operates two municipal wastewater treatment facilities (WWTFs), the South Street (Main) WWTF which serves Sewer District No. 1 and the Route 7 WWTF which serves Sewer District No. 2. The South Street facility was originally constructed in the early 1970s and underwent a major upgrade and expansion that was completed in 1992. The Route 7 WWTF was constructed in 1985. Both WWTFs have been in continuous operation since, and as a result, a significant portion of the Town's wastewater treatment facility equipment has exceeded its anticipated 20 year service life.

Discharges from both plants are regulated by the Connecticut Department of Energy and Environmental Protection (DEEP) through permits issued under the National Pollutant Discharge Elimination System (NPDES) program. The existing permits for both plants expired in 2009 and the Town submitted renewal applications as required by the program rules. The DEEP deferred issuing new permits for both WWTFs until DEEP's Phosphorus Reduction Strategy for Inland Non-Tidal Waters could be developed and finalized. In the meantime, the expired NPDES permits were administratively continued and remained in effect.

As a result of the implementation of the Phosphorus Reduction Strategy, in October 2014 DEEP issued a new NPDES permit for the Route 7 WWTF that includes an effluent phosphorus limit that the existing treatment facility cannot meet without modifications. The permit also includes a compliance schedule that defers the implementation of the new limit until August 2019 to allow time for the Town to complete the ongoing facilities planning effort and implement modifications to the Route 7 WWTF to meet the new phosphorus limit. It is anticipated that the South Street WWTF permit, once issued, will also contain a more stringent limit on effluent phosphorus as well as a compliance schedule to meet the new limit. It is anticipated that the existing WWTFs will not be able to meet their future permit limits without some modifications.

A condition of the NPDES permit for both plants requires if the 180 day rolling average for the plant average daily influent flow "exceeds 90 percent of the design flow rate, the permittee shall develop and submit for the review of the Commissioner within one year, a plan to accommodate future increases in flow to the plant." Historically, the South Street WWTF has operated below the design capacity of 1.0 mgd, except for occasional storm induced high flows. In July 2011, the six month rolling average daily (ADF) flow at the South Street WWTF was 1.06 mgd, and 0.06 mgd for the Route 7 WWTF. This represents 106% of the design capacity of the South Street WWTF, and 52% of the design capacity of 0.12 mgd for the Route 7 WWTF.

To respond to the NPDES permit requirement to initiate planning to address the increases in flow, and to address the aging equipment and components at the two WWTFs, the Town has undertaken preparation of this facilities plan. The facilities planning effort is being completed in two phases. In Phase 1 the current and future needs of the collection system for Sewer District 1 and Sewer District 2 have been identified, the capacities of the two WWTFs have been evaluated and the feasibility of land applying of treated effluent at the South Street WWTF has been assessed. In Phase 2 the condition and the current and future needs of both the South Street and Route 7 WWTFs will be evaluated. In addition, the cost effectiveness of eliminating the Route 7 WWTF by pumping collected flow to the South Street WWTF will be considered. A recommended plan to address the Town's wastewater treatment needs for the planning period will be prepared.

PURPOSE

The purpose of this Technical Memorandum is to develop the projected future flows and loads for both treatment plants for the next twenty years.

BACKGROUND

The Ridgefield WWTFs treat wastewater that is generated from domestic and commercial sources, and also treats extraneous flow from infiltration and inflow. To determine the size of the treatment facilities needed to accommodate anticipated growth in the wastewater collection systems over the next 20 years, projections of future flows and loads are required.

Sewer District 1

The existing South Street WWTF was sized to accommodate flows in Sewer District 1 based on a report entitled "Report on Wastewater Treatment and Sewer System Rehabilitation Needs" prepared by Stearns & Wheler, Inc. dated November 1987. That report projected growth within the existing sewer district as well as identified areas of potential need for extension of sewer service to address health related septic system failures. As part of that report, the Ridgefield Planning Department prepared a buildout analysis for the district, and the plant was sized to accommodate flows based on 70 percent of the projected buildout development being realized within the 20 year planning period. The report did not allocate flows to specific parcels within the district, but rather projected flows based on the aggregate growth and the assumption that 70 percent of the buildout would occur during the planning period.

The report also identified 3 potential areas outside of the existing sewer district where extensions of the sewer system could be needed to address health or pollution resulting from failures of on-site septic systems. These areas were:

- The Ramapoo Road area
- The Soundview Road/Marcardon Avenue/Creamery Lane area
- The New Street area

Since the report was prepared, sewer service was extended to the Ramapoo Road area in 1999. Sewers have not been constructed to provide sewer service to the other needs areas. Figure 1 presents a plan showing the current limits of Sewer District 1.

Sewer District 2

The planning for the existing Route 7 WWTF serving Sewer District 2 was completed using a different approach than that for Sewer District 1. The Town was ordered to construct the Route 7 WWTF by the Connecticut DEEP to address documented water pollution problems from specific parcels in the vicinity of the intersection Route 7 and Route 35. To respond to the order, the Town prepared a report entitled “Town of Ridgefield Connecticut, Facilities Plan for Route7/Route 35 Area” prepared by Albertson, Sharp and Ewing dated April 1979 which outlined the sewer service area and projected flows, defined the details of the then proposed sewer collection system, as well as identifying the size and treatment process for the proposed Route 7 WWTF.

To fund the construction of the sewer system and WWTF, all of the parcels to be served formed the basis for Sewer District 2, and each parcel was allocated a flow allowance. The owner of each parcel then purchased the allocated flow allowance which represented their share of the plant capacity. The Route 7 WWTF and collection system was then constructed by the Town. Nearly all of the parcels in Sewer District 2 have since connected to the sewer system, although many of the parcels have not been developed at the density of development permitted by current zoning of the District. As a result, all of the current Route 7 WWTF capacity has been allocated to the existing users, with no capacity available for extension of the collection system. Figure 2 presents a plan of the current limits of Sewer District 1.

DESIGN PARAMETERS

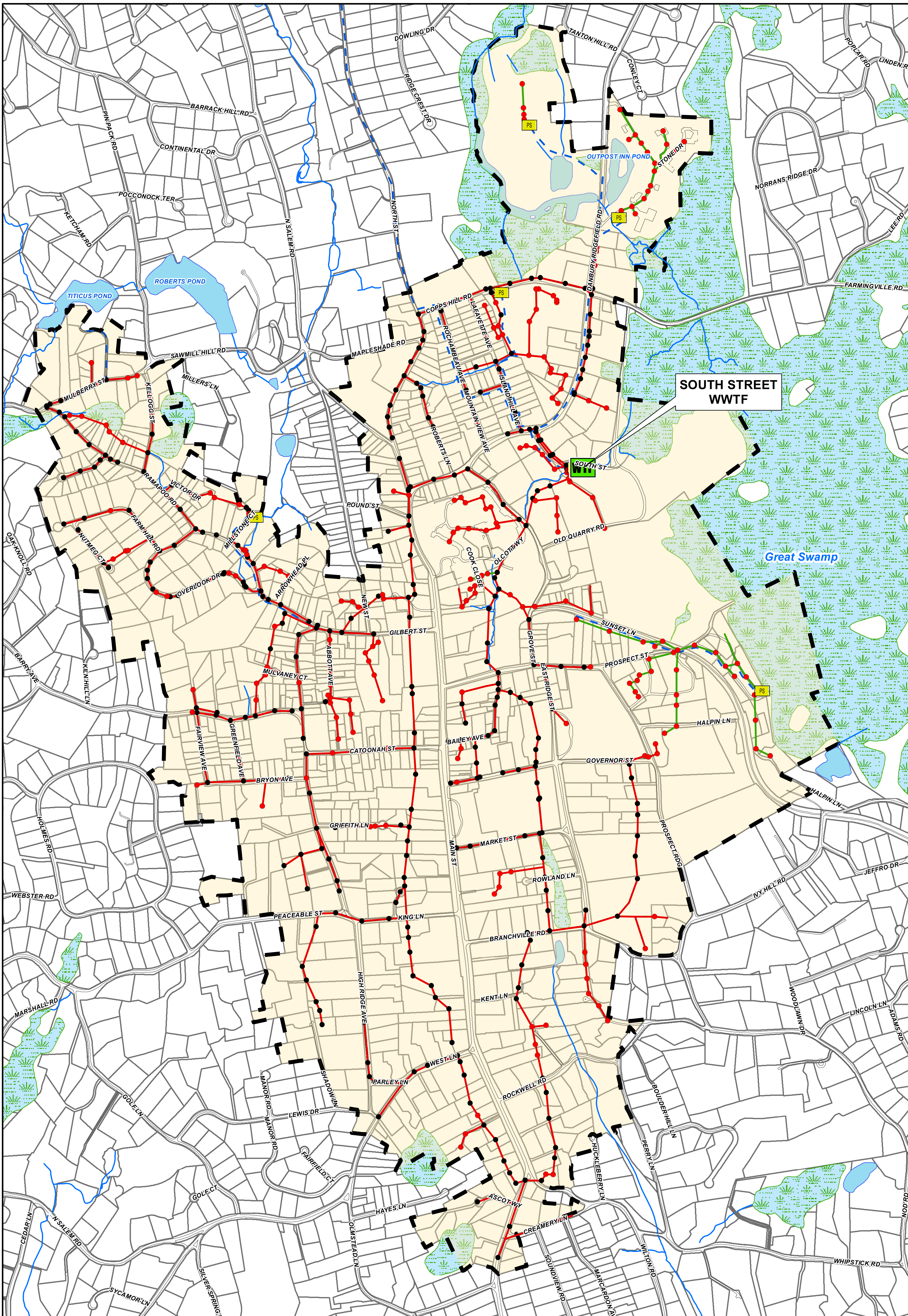
Criteria and parameters used in developing the projections of future wastewater flows are presented in this section.

Period of Design

Treatment facilities are normally designed to accommodate flows expected 20 years in the future. The design year for this projection is 2035.

Population

According to the 2010 census from the U.S. Census Bureau, Ridgefield’s population is 24,638. Ridgefield’s 2010 Plan of Conservation and Development indicates future growth in Ridgefield can be expected to occur at a rate of 3% to 6% each decade. If growth occurs at this rate, projections indicate that Ridgefield’s population could reach approximately 26,000 to 28,000 by 2035.



AECOM

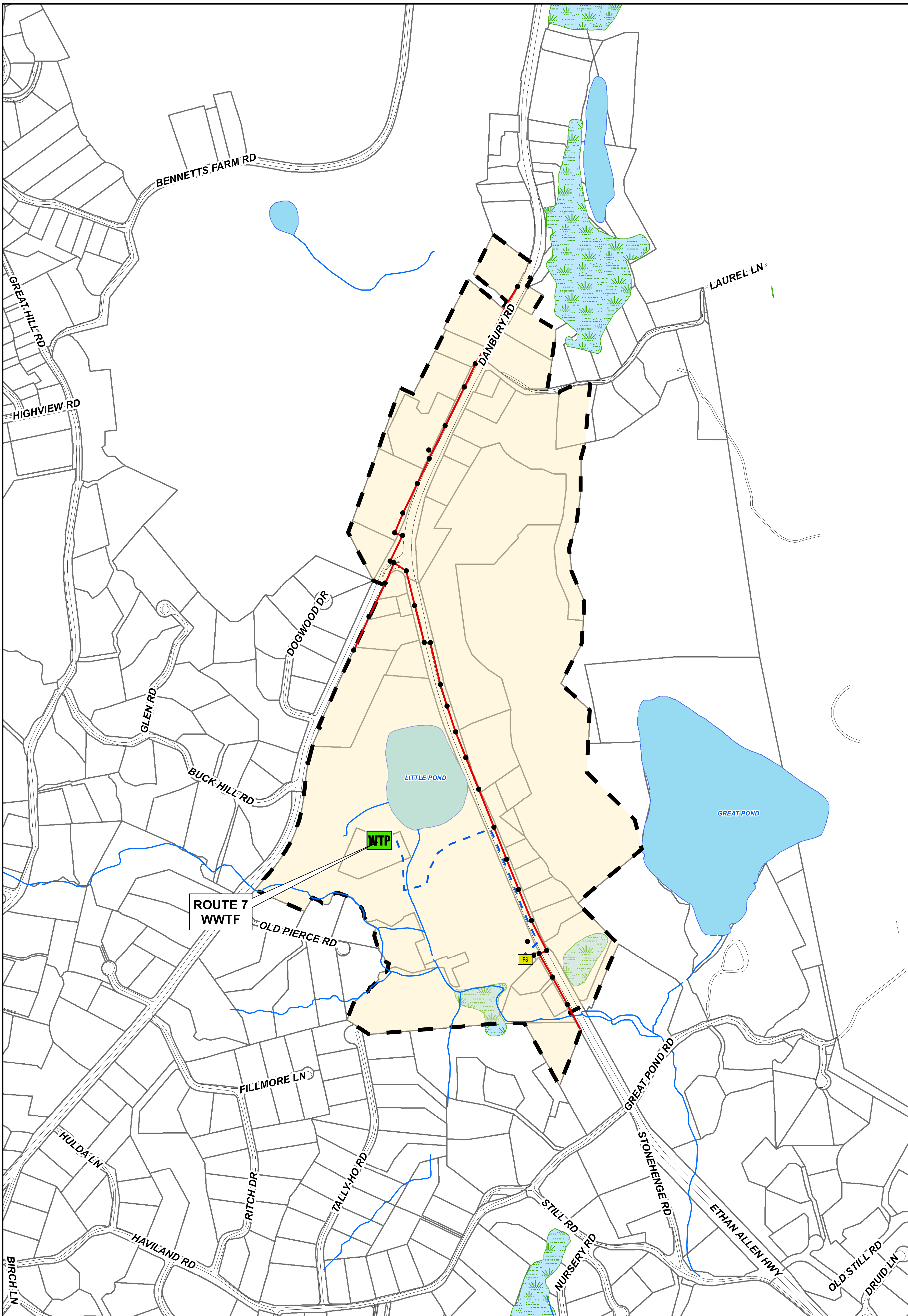
Drawn: BC 3/25/2015

Approved: AA 3/25/2015

FIGURE 1: EXISTING SEWER DISTRICT 1
MARCH 2015

RIDGEFIELD, CT
PHASE I WASTEWATER FACILITIES PLAN
TECHNICAL MEMORANDUM NO. 7

G:\Projects\MUNI\60299267RID\Tech Memo 7\Maps\Figure 1 - Existing Sewer District 1.mxd

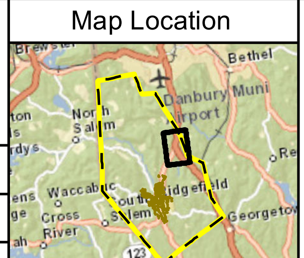


AECOM

Drawn: BC 3/25/2015

Approved: AA 3/25/2015

Map Location



Legend

- WTP WWTF
- PS Pump Station
- Manhole
- Municipal Sewer
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 300 600 1,200 Feet

FIGURE 2: EXISTING SEWER DISTRICT 2
MARCH 2015

RIDGEFIELD, CT
PHASE I WASTEWATER FACILITIES PLAN
TECHNICAL MEMORANDUM NO. 7

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Domestic (Residential) Wastewater Flows

The quantity of domestic wastewater produced is directly related to the population served and the per capita water consumption. The typical per capita wastewater production rate varies between 60 and 70 gallons per person per day. To estimate a per capita wastewater production rate for Ridgefield, water consumption data for several residential areas for the three year period between January 1, 2011 and December 31, 2013 were reviewed. The per capita water consumption as calculated from the three years of data is approximately 68 gallons per day (gpd). The per capita sewage production was estimated at 61 gpd which is equivalent to a 90 percent return to the sewer of water used. This recognizes that some water used is for irrigation and other uses that is not returned as sewage flow. As noted previously, the 2010 U.S. Census tract and block data indicates that the average household size in Sewer District 1 is approximately 2.7 people. Accordingly, 2.7 people per household has been used in projecting the future domestic wastewater flows.

Non-Residential Commercial Flows

Flows from commercial properties depend primarily on the nature of the activity conducted on each sewered property. Typical flow allocations for industrial and commercial land use are between 1,000 and 1,500 gallons per acre per day (gpad). However, a buildout analysis for Sewer District 1 prepared by Planimetrics, Inc. (report included as Appendix A) presents the buildout potential for commercial or business uses in terms of potential additional floor area, represented in square feet (sf). Therefore, water use records as well as floor areas for existing commercial properties were reviewed to develop a typical flow allocation for commercial or business users in gallons per square foot of floor area. As a result, the allowance for commercial or business flows of 121 gallons/1000 sf of floor area has been used in projecting future flows from these land uses. Since many of the commercial flows are from retail commercial properties or offices, it has been assumed that 100% of the water consumed will be returned to the sewer.

Infiltration/Inflow (I/I)

Infiltration is the leakage of groundwater into the collection system, and inflow is the entry of surface water into the collection system. The amount of I/I in a collection system depends on the length of the sewer, and the number of joints and manholes, and condition of the system. The I/I rate also varies depending on the groundwater level, the proximity of water courses, the porosity of the soil, and other topographic and geological features. In projecting future flows resulting from extension of the collection system, an allowance for I/I should be made. An allowance of 200 gallons per acre of service area per day (gpad) has been used projecting infiltration associated with new sewers. As future flows will result largely from construction of new sewers, and in light of the Town's focus on control of extraneous flows, no allowance for inflow in the future flow projections has been made.

Peak Flow Rates and Peaking Factors

Peak flow rates are important so that the unit operations and processes and their interconnecting conduits can be sized appropriately to handle the maximum flow rates. Peak flow is comprised of three elements, peak sanitary wastewater, peak infiltration, and peak inflow. Sanitary wastewater is composed of domestic wastewater and business/commercial (non-domestic) wastewater. Sanitary wastewater excludes infiltration and inflow. Peak sanitary wastewater flow is associated with the diurnal pattern of water use. Infiltration flow rates vary seasonally as groundwater levels fluctuate. Peak infiltration is considered infiltration during a high groundwater non-rainfall period, typically in the spring. Peak inflow is the amount of inflow during a significant rain event.

The peaking factor is the ratio of peak flow rate to average flow rate. Sanitary wastewater peaking factors are larger for small populations, and become smaller as the population increases. This relationship is the result of the variability of times of peaking of individual sources and the attenuation of peaks as the sanitary wastewater flows through the system. To estimate the existing sanitary wastewater peaking factor, the minimum, maximum, and average day flow rates at both of the WWTFs for dry, seasonally low groundwater periods were reviewed. Ninety percent of the minimum flow associated with a dry, low groundwater day is considered to be infiltration recognizing that even in the early morning hours, there is some sanitary wastewater flow in the collection system. This infiltration is subtracted from the maximum and average day flow during the same dry, low groundwater period prior to establishing a ratio between the maximum and average sanitary wastewater flows. Based on this evaluation, the existing sanitary wastewater peaking factors for the South Street and the Route 7 WWTFs are estimated to be approximately 2.8 and 3.0, respectively.

To estimate the existing infiltration peaking factor, the infiltration rates at both of the WWTFs were reviewed for dry, seasonally high groundwater (peak infiltration) and low (average infiltration) periods. The ratio between the maximum and average infiltration rates is the infiltration peaking factor. Based on this evaluation, the existing infiltration peaking factors for the South Street and the Route 7 WWTFs are estimated to be approximately 1.81 and 1.79, respectively. These infiltration peaking factors were also used for projecting peak flows associated with future infiltration.

No additional inflow has been included in the projected flows since any future extensions of the collection system would not allow the connection of inflow sources like sump pumps and roof leaders.

SPATIAL REPRESENTATION OF INFORMATION COLLECTION

As the initial step in the development of the projected flows and loads, available information relevant to the wastewater collection system was obtained. Throughout the development of the projected flows and loads, a geographic information system (GIS) was used to spatially review the data collected. The Town provided electronic base mapping data with the parcel information in GIS. Other information obtained from the Town for use in GIS included zoning information and the current wastewater collection system map.

DEVELOPMENT OF PROJECTIONS OF FUTURE FLOWS – DISTRICT 1

Projected future flows for Sewer District 1 have been developed in steps as follows. First, flows resulting from new connections to the sewer system in the existing district were estimated. Next, flows resulting from redevelopment of existing sewered properties in Sewer District 1 based on current zoning designations were estimated. Lastly, data were reviewed to identify areas where extension of the Sewer District 1 collection system to address pollution or health issues with the continued use of on-site septic system may be warranted. Each of these steps are described below.

Infilling

Projected flows from parcels within the current Sewer District 1 boundaries, termed infilling, were developed first. Infilling flows result from both new connections to the sewer system from properties within the district and from redeveloped parcels that have sewer service with an increased density of development based on current zoning. To estimate the infilling flows, an analysis of zoning and land use was required, referred to as a buildout analysis.

District 1 Buildout Analysis. A sewer service area buildout analysis was conducted by Planimetrics, Inc. of Simsbury, CT. The buildout analysis was conducted for Sewer District 1. The buildout report is included in Appendix A. The analysis was based on the current zoning and environmental constraints and estimates of the potential for additional residential growth (in units) and commercial growth (in square footage of building) in Sewer District 1. In developing the buildout analysis, as directed by the Town, no allowance has been included by Planimetrics for any development under the Connecticut Affordable Housing Appeals Act, Section 8-30g of the Connecticut General Statutes. Currently, the Town has a four year moratorium on CGA 8-30g projects. Consequently, projections of future flows have been based on existing zoning.

Once the Buildout Analysis was prepared, AECOM met with representatives of the Ridgefield Planning and Zoning Department, the Town Sanitarian, the Town Engineering Department, and the WPCA to review the findings and obtain input on areas of the Town that are anticipated to develop over the next twenty years, and the need for sewer service. This included data from the Town on large parcels within the sewer districts that could be subdivided. At this meeting, it was noted that the future flow projections for Sewer District 1 should reflect 70 percent of the projected buildout occurring within the 20 year planning period for the facilities plan.

The buildout analysis did not address the redevelopment of the former Schlumberger parcel at 26 Old Quarry Road (parcel E14-0162). It is a 40 acre parcel, which the Town has indicated under current zoning may have the potential for 80 residential units on the 10 acres zoned MFDD and potential for approximately 80,000 square feet of office space on the remaining acres in the B-2 district. Another former Schlumberger parcel on Old Quarry Road (parcel E14-0159) currently has a proposal for a 48 room hotel, 11 units of residential, 20,825 sf of commercial space, and 48,836 sf of storage space. These properties and uses are reflected in the projected infilling flows. Projected flows from the infilling properties were then developed in two steps by first considering the existing development condition, and then the potential development condition.

The buildout analysis also did not address any planned growth in Town owned facilities such as schools, public works, etc. With the small projected increase in the town population of 6 to 12 percent over the 20 year planning period, and the projected decline of the student population in the Ridgefield school system, no projected increase in wastewater flows from municipal facilities has been included.

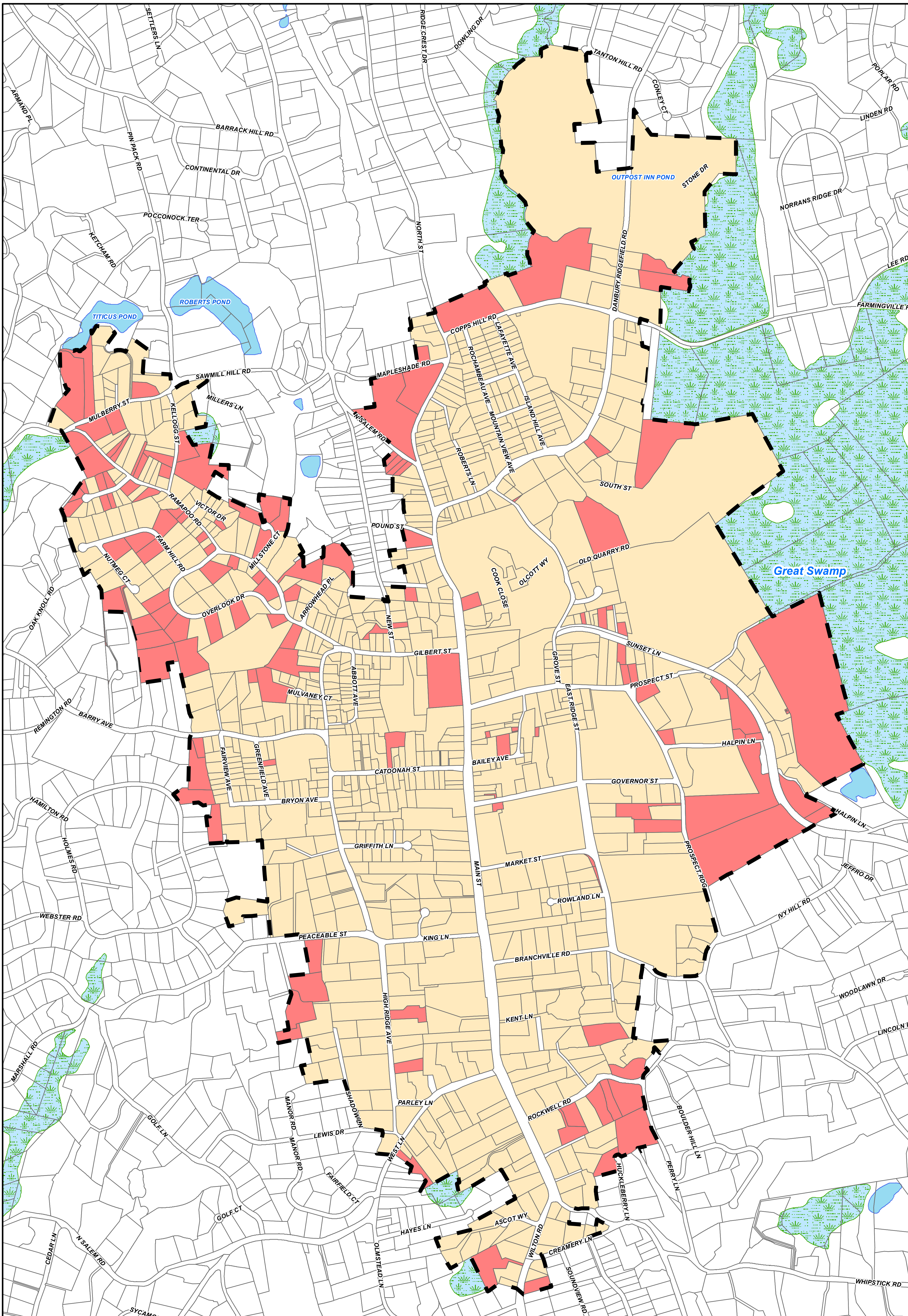
Infilling – Existing Development Condition. The Town provided a spreadsheet containing a list of the properties in the Sewer District 1 that are currently billed for sewer service. The sewer user listings contained the address and owner of each property. The list of sewered properties was linked to the GIS information, also obtained from the Town, and the unsewered properties were identified. Based on this data there are a total of 1,498 parcels within Sewer District 1, of which 138 parcels are not connected to the existing collection system. Figure 3 presents the location of unsewered parcels within Sewer District 1.

In developing the flow projections under the existing development condition, it has been assumed that 100 percent of the residential and non-residential existing infill properties would connect to the sewer within the 20 year planning horizon. The number of residential units was tabulated and the floor area (square footage of building) for each of the non-residential (business/commercial) was tabulated for the properties. The resulting flows are shown in Table 1. As indicated, if the existing unsewered infilling properties were connected to the sewer collection system, an average daily sanitary wastewater flow of approximately 32,500 gallons per day (gpd) would be generated. An additional 19,500 gpd of infiltration would be generated for a total of approximately 52,000 gpd of additional flow from infill properties under existing conditions.

TABLE 1. INFILL FLOWS - EXISTING DEVELOPMENT CONDITION

Flow Component	Acreage	Number of Residential Units	Non-Residential Floor Area (sf)	Average Daily Flow (gpd)
Sanitary - Non-Sewered Residential	80.02	87	-	14,329
Sanitary - Non-Sewered Non-Residential	17.58	-	148,451	17,973
Total Sanitary Flow	97.60	87	148,451	32,302
Infiltration				19,520
Total Flow				51,822

Infilling - Potential Development Condition. The flow projections under the potential development condition, based on discussion with the Town Engineer, Planning Department, and WPCA, are based on 70 percent of the additional residential units and additional non-residential floor area being developed within the 20 year planning horizon. Currently, many of the infilling properties have not been developed at the density permitted by zoning. To assess the flows that could be generated if these properties were developed as zoned, the potential number of units for each residential parcel were estimated in the Buildout Analysis prepared by Planimetrics based on consideration of the zoning requirements and environmental constraints. To assess the flows that could be generated from non-residential properties, the potential additional floor area for non-residential parcels were estimated by Planimetrics given the zoning requirements. The resulting flows are presented in Table 2. As indicated, if 70 percent of the potential infilling properties were developed as allowed by current zoning and connect to the sewer, and additional average daily sanitary wastewater flow of approximately 57,000 gpd would be generated. An additional 3,000 gpd of infiltration would be generated for a total of approximately 60,000 gpd of additional flow from infill properties under potential development conditions.

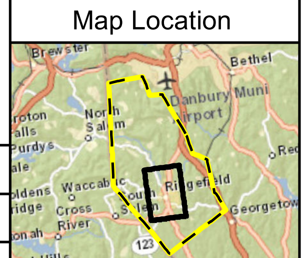


AECOM

Drawn: BC 3/26/2015

Approved: AA 3/26/2015

Map Location



Legend

- Sewered Properties
- Unsewered Properties
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 500 1,000 2,000 Feet

FIGURE 3: SEWER DISTRICT 1
UNSEWERED PROPERTIES
MARCH 2015

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G:\Projects\MUNI\60299267RID\Tech Memo 7\Maps\Figure 3 - Sewer District 1 Unsewered Parcels.mxd

TABLE 2. INFILL FLOWS - POTENTIAL DEVELOPMENT CONDITION

Flow Component	Acreage	Number of Residential Units	Non-Residential Floor Area (sf)	Average Daily Flow (gpd)
Sanitary - Sewered Residential	-	113	-	18,562
Sanitary - Sewered Non-Residential	-	-	288,202	34,872
Sanitary - Non-Sewered Residential	15.28	20	-	3,228
Sanitary - Non-Sewered Non-Residential	-	-	4,681	566
Total Sanitary Flow	15.28	133	292,883	57,228
Infiltration				3,056
Total Flow				60,284

Table 3 presents a summary of projected infill flows from both existing and potential development conditions.

TABLE 3. SUMMARY OF PROJECTED FLOWS ASSOCIATED WITH INFILLING

Flow Component	Average Daily Flow (gpd)
Sanitary - Existing Development Condition	32,302
Sanitary - Potential Development condition	57,228
Total Sanitary Flow	89,530
Infiltration	22,576
Total Flow	112,106

Potential Sewer Needs Areas

In addition to flows resulting from infilling within Sewer District 1, the other component of future wastewater flows would be extensions of the collection system to serve areas outside the current sewer district. The WPCA has directed that only sewer extensions to address documented public health or pollution issues from existing development be considered. Sewer extensions to promote additional development have not been considered, in accordance with both the Ridgefield Plan of Conservation and Development, and the State of Connecticut Plan of Conservation and Development.

Ridgefield Plan of Conservation & Development. The Town's Plan of Conservation and Development (POCD) was updated in 2010. As noted in Chapter 14 of the plan:

> "Infrastructure, particularly public sewers, should not dictate development intensity or patterns. Rather, infrastructure should support the development patterns and intensities desired by the community. Extending sewers can address public health issues and enhance economic development and overall development goals. However, extending sewers could

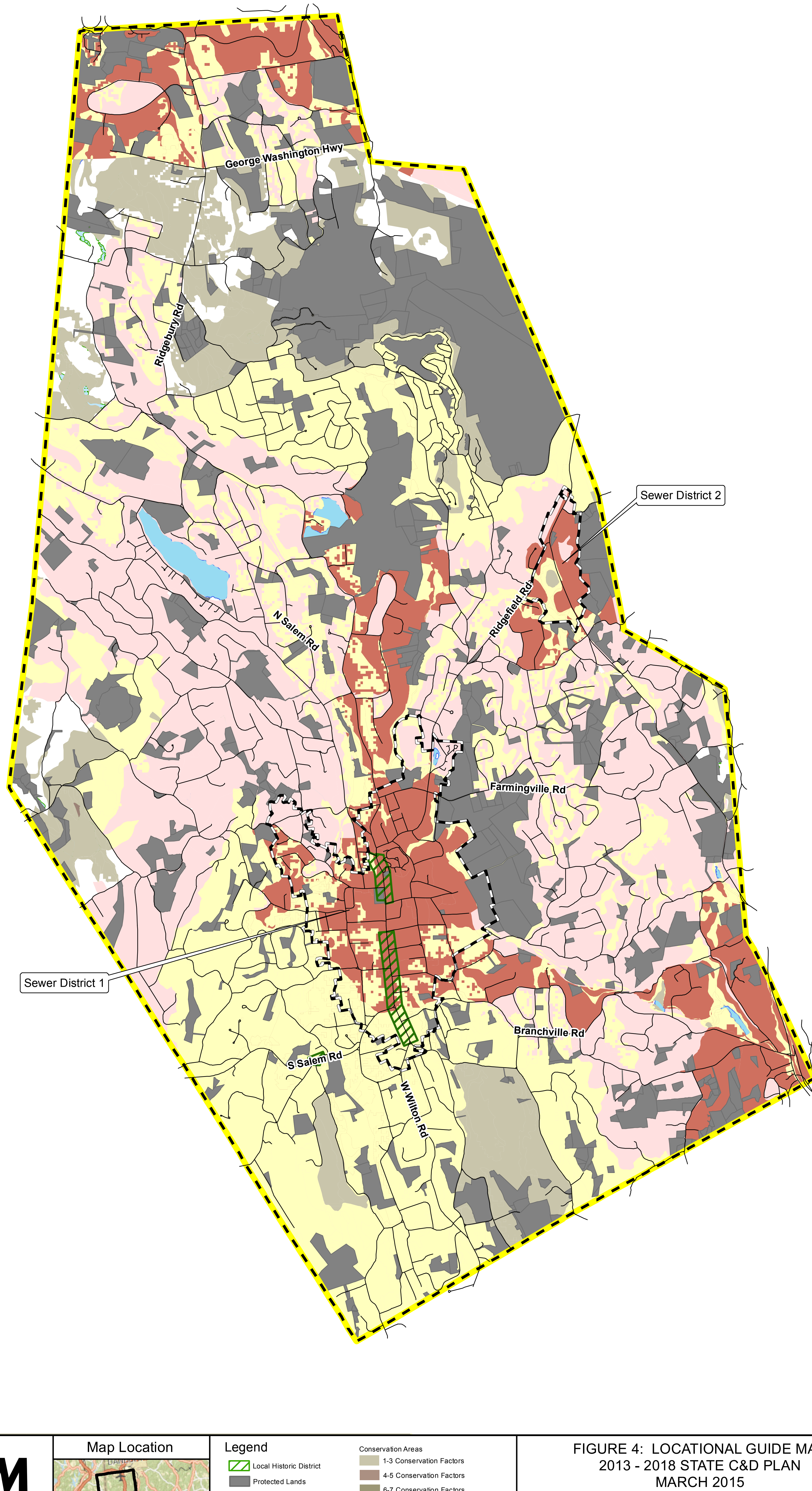
> lead to increased densities in areas where such density is not desired. Thus, coordinating sewer extensions with land use goals is critical."

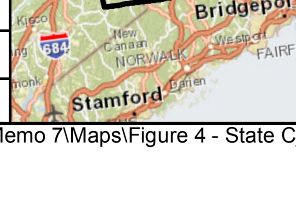
During the preparation of the Phase 1 Facilities Plan, a joint meeting of the WPCA, Board of Selectman, and Planning and Zoning Commission was held on September 26, 2013. One of the discussion items was criteria for inclusion of areas for potential sewer needs. The consensus at the meeting was that extensions of sewers in Sewer District 1 should only be considered to address documented health issues with existing development in close proximity to the existing Sewer District. No extensions should be considered for areas that may have problems with septic systems that are remote from the existing Sewer District, and that sewer extensions to facilitate increased development of undeveloped areas should not be considered.

Conservation and Development Policies Plan for Connecticut, 2013-2018. The Connecticut Office of Policy & Management has released updated versions of the statewide Conservation and Development (C&D) Plan. This plan identifies areas where development is encouraged as well as areas where development is discouraged. Any future sewer service areas will be considered in light of the State plan as there are funding assistance implications for sewer extensions into areas designated for preservation. The limits of the various land use designations in the State C&D plan have been added as a data layer to the base map. Figure 4 presents the C&D map for Ridgefield.

In 2013, the Connecticut House of Representatives and the Senate adopted the Conservation and Development Policies Plan for Connecticut, 2013-2018 (C&D Plan). This plan was developed by the Office of Policy and Management (OPM) as a policy guide for state planning, programs, and regulation. It serves as a statement of the development, resource management, and public investment policies for the state. This update of the C&D plan, which is the sixth revision to the initial plan prepared in 1979, is highly focused on growth management using an incentive based approach. In essence, the plan encourages municipalities and regional planning agencies to concentrate development around existing infrastructure and discourages construction of new infrastructure in outlying areas, an approach that has been termed "smart growth". The plan includes a map of the state identifying areas where growth is encouraged (Priority Funding Areas), and areas where conservation of existing resources is encouraged (Conservations Areas). In areas shown as conservation areas extension of the sewer system to support new development is not encouraged under the C&D Plan. Extension of the wastewater collection system to address pollution or failures of on-site disposal systems in conservation areas serving existing development is allowed provided that it is approved by the OPM which administers the C&D Plan.

The incentive aspect of the plan relates to the ability of a municipality to obtain funding assistance from the state. Any request for grant or loan from the state for a capital project by a municipality is required to be approved by the State Bond Commission. As part of the review and approval process by the Bond Commission, the compliance of the project requesting funds with the C&D plan is reviewed. Simply stated, projects that are not in compliance with the C&D plan may not be eligible for funding assistance. A municipality may still proceed with implementing a project that is not in compliance with the C&D plan using local funds. However, through discussions with the DEEP, it has been indicated that if this approach is taken with the extension of sewer service to areas defined as conservation areas in the C&D plan, this may jeopardize any future state funding related to municipal wastewater facilities in that community. Extension of the collection system to serve areas of existing





Legend

- Local Historic District
- Protected Lands

Priority Funding Areas

- 1 - 2 Criteria
- 2 - 4 Criteria
- 4 - 6 Criteria
- Village Priority Funding Area

Conservation Areas

- 1-3 Conservation Factors
- 4-5 Conservation Factors
- 6-7 Conservation Factors
- Balanced Priority Funding Area
- Undesignated Areas
- Sewer District 1 & 2
- Roads
- Waterbodies

N

0 2,000 4,000 8,000 Feet

FIGURE 4: LOCATIONAL GUIDE MAP
2013 - 2018 STATE C&D PLAN
MARCH 2015

RIDGEFIELD, CT
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TECHNICAL MEMORANDUM NO. 7

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Drawn:	BC	3/25/2015
Approved:	AA	3/25/2015

G:\Projects\MUNI\60299267RID\Tech Memo 7\Maps\Figure 4 - State C_D Plan.mxd development that have identified problems or failure of on-site disposal systems in areas designated as conservation can be allowed with the approval by the DEEP and OPM.

The focus of the C&D Locational Guide Map (LGM) is to limit extensions of the collection system into conservation areas for new development. As a result, the DEEP is strongly recommending that communities carefully consider plans for future extensions of the wastewater collection system in relation to the C&D plan requirements.

The Locational Guide Map is applied as follows:

1.) For any growth-related project, the sponsoring State Agency must: a) document how the proposed action is classified on the LGM; b) consult the municipal plan of conservation and development if a proposed project falls outside of a Priority Funding Area (PFA); and c) determine whether to seek OPM's approval for an exception under Connecticut General Statutes (CGS) Section 16a-35d;

2.) The sponsoring State Agency, at its discretion, determines whether to provide funding for any growth-related project that has been deemed consistent with the Conservation and Development Policies, regardless of its PFA designation on the Locational Guide Map. Table 4 presents a summary of how the LGM is applied.

TABLE 4. APPLICATION OF THE LOCATIONAL GUIDE MAP (LGM)

Priority Funding Areas	Balanced Priority Funding Areas	Village Priority Funding Areas	Conservation Areas	Undesignated Areas
Growth-related projects may proceed without an exception	Growth-related projects may proceed without an exception, if the sponsoring agency documents how it will address any potential policy conflicts	Growth-related projects may proceed without an exception, if the sponsoring agency documents how it will help sustain village character	Growth-related projects may proceed with an exception*	Growth-related projects may proceed with an exception*

* Note: In order for a growth-related project to be funded outside of a PFA, CGS Section 16a-35d requires the project to be supported by the municipal plan of conservation and development. Furthermore, CGS Section 8-23(b) makes municipalities ineligible for discretionary state funding, effective July 1, 2014, if they have not updated their local plans within the required ten-year timeframe.

3.) The sponsoring State Agency must report annually on any grants it provides for growth-related projects located outside of PFAs.

As indicated in Figure 4, Ridgefield's LGM is comprised of Priority Funding Areas, Conservation Areas, Protected Lands, and a Local Historic District. The majority of Sewer Districts 1 and 2 are considered Priority Funding Areas, however, they do contain Conservation Areas. Areas that meet the criteria of both Priority Funding Areas and Conservation Areas are considered Balanced Funding

Areas. As noted previously, extensions of sewers in Sewer District 1 will only be considered to address documented health issues with existing development in close proximity to the existing Sewer District. In addition, all of the current Route 7 WWTF (Sewer District 2) capacity has been allocated to the existing users, with no capacity available for extension of the collection system. Potential sewer needs areas will be evaluated for consistency with the C&D Plan.

Identification of Potential Sewer Needs Areas. To identify the potential sewer needs areas that may contribute wastewater to Sewer District 1 through potential future sewer extensions, the prior facilities planning data were reviewed, the Town Sanitarian was consulted to obtain input and data on septic system problem areas, septage pumping data was obtained and reviewed, and public input was obtained. Each of these items is described below.

Previous Facilities Planning Data. Areas with sewer needs that were documented in the previous 1987 Facilities Plan that have not been sewered include the Soundview Road, Creamery Lane, and Marcardon Avenue area as well as the New Street area.

Collection and Review of Septic System Data. Areas with sewer needs are those that have been documented as having a number of failing septic systems. Input from the Director of Health on areas of concern were an important part of the identification of sewer needs areas. A meeting with the Director of Health was held. Areas of concern for long term viability of septic systems based on historical observations include New Street, and the Creamery Lane, Soundview Road, Marcardon Avenue areas.

Septic system repair/replacement data from the Health Department were obtained and reviewed. Locations of system repairs/replacements have been plotted on the base map and are shown on Figure 5. It is reported that the areas of septic system repair/replacements indicated consist of hardpan soils and high groundwater tables. Repairs to the septic systems required special designs by engineers, curtain drains, and/or large fill profiles. As indicated the repair/replacements are clustered in the Soundview Road/Marcardon Avenue neighborhood. Documents provided by the Town Sanitarian are included in Appendix B.

Collection and Review of Septage Data. Four years of septage hauler data were obtained and reviewed from the South Street WWTF files. The locations of systems that are pumped frequently have been plotted on the base map and are shown on Figure 6. A review of this information shows no apparent trends or clusters of systems with frequent pumping histories. Three properties, however, have been identified with pumping frequencies in excess of 4 pump outs per year. These properties are shown in red on Figure 6 and include the following:

Address	Parcel ID	Description	Acreage	No. Buildings
223 West Mountain Road	C13-0001	Ridgefield Academy	42	4
439 Silver Spring Road	D17-0091	Silver Spring Country Club	276.88	5
720 Branchville Road	I17-0093	CVS	2.59	1

Though these properties have septic system pumping frequencies greater than the norm, it does not necessarily indicate a need for a sewer extension to these areas. These properties are miles apart from one another, they are not surrounded by other properties with similar

FIGURE 5: SEPTIC SYSTEM REPAIRS/REPLACEMENTS (1974 - 2014)
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Legend

WTF	WWTF
PS	Pump Station
●	Manhole
●	Manhole - Not Field Located
	Municipal Sewer
	Private Sewer
	Sewer District 1
	Roads
	Streams
	Swamps
	Waterbodies
	Parcel Polygon
	Septic System Repair/Replacement

0 150 300 600 Feet

N

Map Location

AECOM

Drawn: BC 3/25/2015

Approved: AA 3/25/2015

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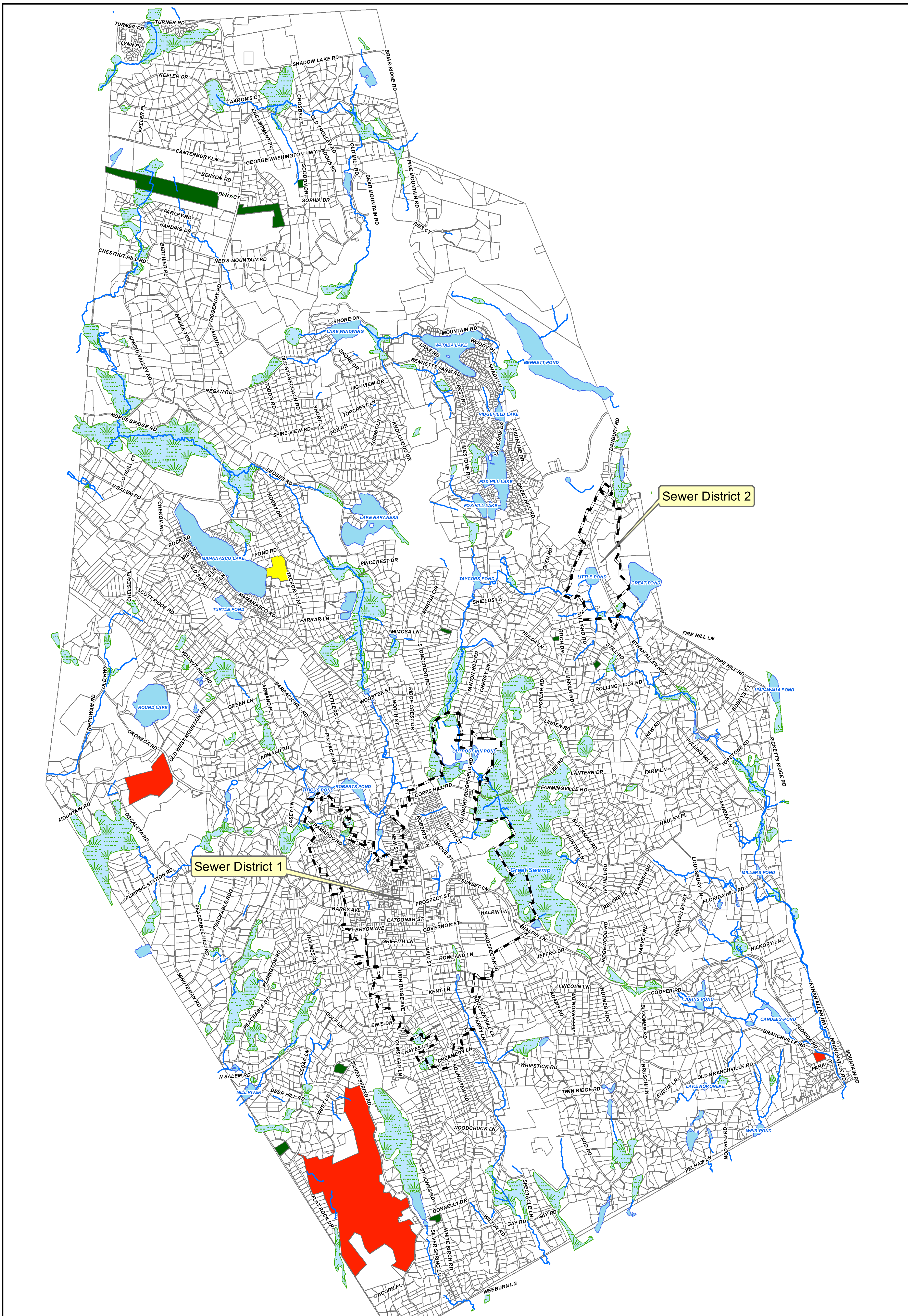

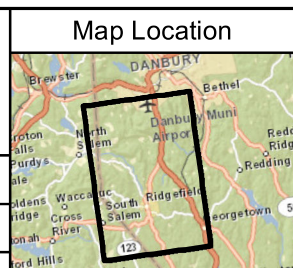

Map Location

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Approved:	AA	3/25/2015

Legend

Septage Pumping Frequency/Yr

- 1 - 2
- 2 - 3
- > 4
- Sewer District 1 & 2
- Roads
- Force Main
- Streams
- Swamps
- Waterbodies
- Parcel Polygon

N

0 1,750 3,500 7,000 Feet

FIGURE 6: SEPTAGE PUMPING FREQUENCY (APRIL 2010 - AUGUST 2013)

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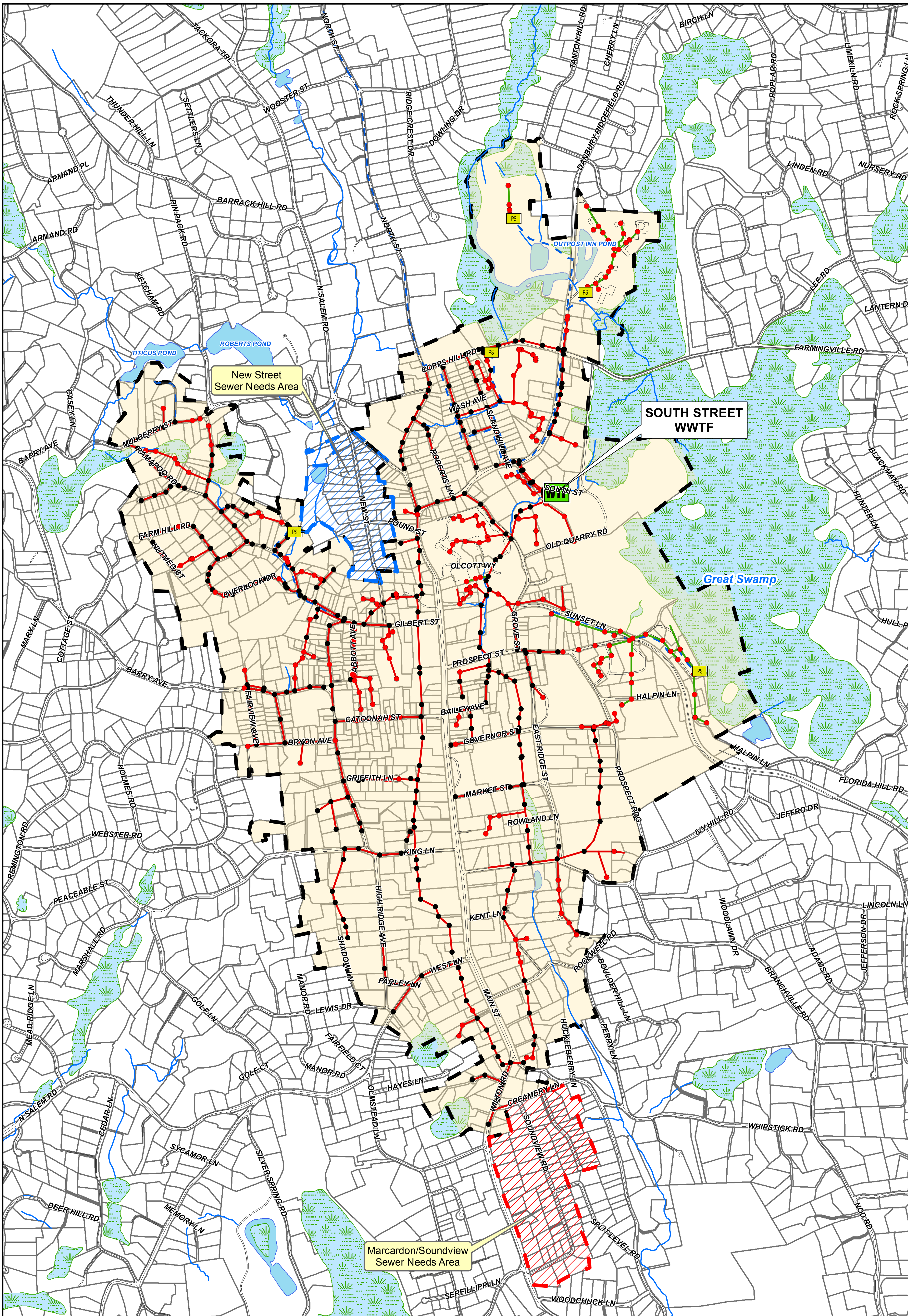
G:\Projects\MUNI\60299267RID\Tech Memo 7\Maps\Figure 6 - Septage Pumping Frequency.mxd



Legend

WTP	WWTF		Roads
PS	Pump Station		Force Main
	Manhole		Streams
	Manhole - Not Field Located		Swamps
	Municipal Sewer		Waterbodies
	Private Sewer		Parcel Polygon

0 625 1,250 2,500 Feet

N

Map Location

AECOM

Drawn: BC 3/25/2015

Approved: AA 3/25/2015

FIGURE 7: SEWER NEEDS AREAS
MARCH 2015

RIDGEFIELD, CT
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TECHNICAL MEMORANDUM NO. 7

G:\Projects\MUNI\60299267RID\Tech Memo 7\Maps\Figure 7 - Sewer Needs Areas.mxd pumping frequencies, and are not adjacent to sewered areas. As these are isolated properties, remote from the existing sewer district, they have not been identified as sewer needs areas.

Sewer Needs Public Input Meeting. A public meeting was held on December 4, 2014 to allow residents to provide input on areas in Town where existing conditions may warrant extension of the sewer system in Sewer District 1. Public input was obtained. Residents requested extension of the sewer system to serve Olmstead Lane, Wilton Road East, Marcardon Avenue, Soundview Road, and New Street citing poor soils, frequent pump outs of septic systems, high groundwater, and surface runoff. Minutes of the sewer needs public meeting are included in Appendix C. Based on the public input, the Director of Health recommended that the properties on Wilton Road West adjacent to Soundview Drive be included in this sewer needs area.

Identified Sewer Needs Areas. Based on the preceding review, areas of potential sewer needs were identified. These areas are shown in Figure 7 and include the New Street Area and the Marcardon/Soundview Area. Each area is discussed in more detail in the following sections.

New Street Area. New Street is located to the north of the existing Sewer District 1. The unsewered portion of New Street, between Silver Birch Road and Saw Mill Road, consists of approximately 50 parcels, ranging in size between approximately 0.13 acre and approximately 6.5 acres. New Street is an established residential area. Public water supply is available for the entire length of the street.

Health Department Data. Current health department records were not available at the time of this report. However, the 1987 Facilities Plan noted that some of the original septic systems dated back to the 1940's and consisted of small tanks and undersized cesspools. It also reported numerous repairs due to septic overflows and high groundwater conditions.

Public Input. The need for municipal sewer service was expressed at the public input meeting, citing surface runoff as a problem.

C&D Policies. The entire sewer needs area is within the Priority Funding Area as shown on the C&D LGM. Portions of the sewer needs area also meet the criteria as Conservation Areas which include the following factors:

- 100 year Flood Zones
- Undeveloped Prime, Statewide Important and locally important agricultural soils greater than 25 acres.

Areas that meet the criteria of both Priority Funding Areas and Conservation Areas are considered Balanced Priority Funding Areas. Therefore, growth-related projects may proceed without an exception, if the sponsoring agency documents how it will address any potential policy conflicts.

Potential policy conflicts could be addressed by limiting the construction of public infrastructure to within the existing right of way, thereby avoiding the Conservation Areas.

Projected Flows - Existing Development Condition. In developing the flow projections under the existing development condition, it has been assumed that 100 percent of the residential and non-residential existing sewer needs area properties would connect to the sewer within the 20 year planning horizon. The number of residential units was tabulated and the floor area (square footage of building) for each of the non-residential (business/commercial) was tabulated for the properties. The resulting flows are shown in Table 5. As indicated, if the existing New Street sewer needs area properties were connected to the sewer, an average daily sanitary wastewater flow of approximately 8,000 gallons per day (gpd) would be generated. An additional 5,000 gpd of infiltration would be generated for a total of approximately 13,000 gpd of additional flow from the New Street sewer needs area properties under existing conditions.

TABLE 5. NEW STREET SEWER NEEDS AREA FLOWS - EXISTING DEVELOPMENT CONDITION

Flow Component	Acreage	Number of Residential Units	Non-Residential Floor Area (sf)	Average Daily Flow (gpd)
Sanitary - Residential	23.03	47		7,741
Sanitary - Non-Residential	0.27	-	1,484	180
Total Sanitary Flow	23.30	47	1,484	7,921
Infiltration				4,660
Total Flow				12,581

Projected Flows - Potential Development Condition. In developing the flow projections under the potential development condition, it has been assumed that 70 percent of the additional residential units and additional non-residential floor area would be developed within the 20 year planning horizon. Some of the New Street sewer needs area properties have not been developed at the density permitted by zoning. To assess the flows that could be generated if these properties were developed as zoned, the potential number of units for each residential parcel was calculated by Planimetrics given the zoning requirements and environmental constraints. To assess the flows that could be generated from non-residential properties, the potential additional floor area for non-residential parcels was calculated by Planimetrics given the zoning requirements. The resulting flows are indicated in Table 6. As indicated, if 70 percent of the potential New Street sewer needs area properties were developed as allowed by current zoning and connected to the sewer, an additional average daily sanitary wastewater flow of approximately 1,000 gpd would be generated. An additional 2,000 gpd of infiltration would be generated for a total of approximately 3,000 gpd of additional flow from the New Street sewer needs area properties under potential development conditions.

TABLE 6. NEW STREET SEWER NEEDS AREA FLOWS - POTENTIAL DEVELOPMENT CONDITION

Flow Component	Acreage	Number of Residential Units	Non-Residential Floor Area (sf)	Average Daily Flow (gpd)
Sanitary - Residential	9.03	6	-	1,021
Total Sanitary Flow	9.03	6	-	1,021
Infiltration				1,806
Total Flow				2,827

Marcardon/Soundview Area. The Marcardon/Soundview area is located to the south of the existing Sewer District 1. It is made up of all of Marcardon Avenue, all of Soundview Road, a portion of Wilton Road West between Street Johns Road and Olmstead Lane, and Creamery Lane. It consists of approximately 76 parcels. All of Marcardon Avenue, Creamery Lane and Wilton Road West are served by public water. Only the homes on the northernmost half of Soundview Road have public water service. Lot sizes range from 0.18 to 2.8 acres.

Health Department Data. Health department records indicate that a number of properties had septic failures. They are in areas consisting of hardpan soils and high groundwater tables. Repairs required special designs by engineers, curtain drains, and/or large fill profiles.

Public Input. The need for municipal sewer service was expressed at the public input meeting. Proponents of extending the sewer to this area cited high groundwater, ledge, surface water effecting septic systems, and multiple pump outs per year.

C&D Policies. The entire sewer needs area is within the Priority Funding Area as shown on the C&D LGM. The entire sewer needs area is also within the Conservation Area which includes the following factors:

- Water Supply Watershed
- Undeveloped Prime, Statewide Important and locally important agricultural soils greater than 25 acres
- Wetland Soils greater than 25 acres

Areas that meet the criteria of both Priority Funding Areas and Conservation Areas are considered Balanced Priority Funding Areas. Therefore, growth-related projects may proceed without an exception, if the sponsoring agency documents how it will address any potential policy conflicts.

Potential policy conflicts could be addressed by limiting the construction of public infrastructure to within the existing right of way, thereby avoiding the Conservation Areas.

Projected Flows - Existing Development Condition. In developing the flow projections under the existing development condition, it has been assumed that 100 percent of the residential and non-residential existing sewer needs area properties would connect to the sewer within the 20 year planning horizon. The number of residential units was tabulated and the floor area (square footage of building) for each of the non-residential (business/commercial) was tabulated for the properties. The resulting flows are shown in Table 7. As indicated, if the existing Marcardon/Soundview sewer needs area properties were connected to the sewer system, an average daily sanitary wastewater flow of approximately 13,000 gallons per day (gpd) would be generated. An additional 10,000 gpd of infiltration would be generated for a total of approximately 23,000 gpd of additional flow from the New Street sewer needs area properties under existing conditions.

TABLE 7. MARCARDON/SOUNDVIEW SEWER NEEDS AREA FLOWS - EXISTING DEVELOPMENT CONDITION

Flow Component	Acreage	Number of Residential Units	Non-Residential Floor Area (sf)	Average Daily Flow (gpd)
Sanitary - Residential	51.16	76	-	12,517
Total Sanitary Flow	51.16	76	-	12,517
Infiltration				10,232
Total Flow				22,749

Projected Flows - Potential Development Condition. In developing the flow projections under the potential development condition, it has been assumed that 70 percent of the additional residential units and additional non-residential floor area would be developed within the 20 year planning horizon. One of the Marcardon/Soundview sewer needs area properties has not been developed at the density permitted by zoning. To assess the flows that could be generated if these properties were developed as zoned, the potential number of units for each residential parcel was calculated by Planimetrics given the zoning requirements and environmental constraints. To assess the flows that could be generated from non-residential properties, the potential additional floor area for non-residential parcels was calculated by Planimetrics given the zoning requirements. The resulting flows are indicated on Table 8. As indicated, if 70 percent of the potential Marcardon/Soundview sewer needs area properties were developed as allowed by current zoning and connect to the sewer, and additional average daily sanitary wastewater flow of approximately 115 gpd would be generated. No additional infiltration would be generated for a total of approximately 115 gpd of additional flow from the Marcardon/Soundview sewer needs area properties under potential development conditions.

TABLE 8. MARCARDON/SOUNDVIEW SEWER NEEDS AREA FLOWS - POTENTIAL DEVELOPMENT CONDITION

Flow Component	Acreage	Number of Residential Units	Non-Residential Floor Area (sf)	Average Daily Flow (gpd)
Sanitary - Residential	-	1	-	115
Total Sanitary Flow	-	1	-	115
Infiltration				-
Total Flow				115

Table 9 presents a summary of projected flows from the identified Sewer Needs Areas in Sewer District 1.

TABLE 9. SUMMARY OF SEWER DISTRICT 1 PROJECTED FLOWS ASSOCIATED WITH SEWER NEEDS AREAS

Flow Component	Average Daily Flow (gpd)
Sanitary - New Street	8,942
Sanitary - Marcardon/Soundview	12,632
Total Sanitary Flow	21,591
Infiltration	16,698
Total Flow	38,289

Summary of Projected Flows – Sewer District 1

The additional projected 2035 average daily flow to the South Street WWTF from infilling and potential sewer needs areas is summarized in Table 10.

TABLE 10. SOUTH STREET WWTF PROJECTED AVERAGE DAILY FLOW IN GALLONS PER DAY (GPD)

Category	Infilling	Sewer Needs Areas		Total Average Daily Flow
		New Street Area	Marcardon/Soundview Area	
Residential	36,000	9,000	13,000	58,000
Non Residential	54,000	200	-	54,200
Subtotal				112,200
Infiltration	22,000	6,000	10,000	38,000
Total	112,000	15,200	23,000	150,200

The current average daily flow at the South Street WWTF is approximately 0.85 mgd. Adding the projected average daily future flow of 0.150 mgd to the existing 0.850 mgd, results in 1.00 mgd projected average daily flow to the South Street WWTF.

Projected Peak Flow – Sewer District 1

To estimate a total peak flow rate for the year 2035, the projected peak flow from the potential sewer needs areas is added to the current peak flows. As described previously, the peaking factor for the sanitary wastewater component of the flow at the South Street WWTF is 2.8 and the peaking factor of the future infiltration component of the flow is 1.81. Based on projected flows in Table 10, the projected additional average sanitary wastewater flow at the South Street WWTF for 2035 is approximately 112,000 gpd with a peak of 314,000 gpd using a peaking factor of 2.8. The projected additional average infiltration flow for 2035 is approximately 39,000 gpd with a peak flow of 71,000 gpd based on a peaking factor of 1.81.

The current sanitary wastewater average daily flow at the South Street WWTF is 0.592 mgd with a peaking factor of 2.8 for an existing sanitary wastewater peak flow of approximately 1.66 mgd. The current peak infiltration is approximately 363,000 gpd. The current peak inflow following a significant rain event, like the March and April, 2011 and June and July 2013 rain events, is estimated to be approximately 3,900,000 gpd. The peak inflow was calculated by subtracting the average sanitary wastewater flow and average daily infiltration flow from the peak flow during the rain event. Summing the various components of the peak flow, the total year 2035 peak flow is projected to be approximately 6.3 mgd. Table 11 presents a summary of the components for the projected peak flow at the South Street WWTF.

TABLE 11. SOUTH STREET WWTF PROJECTED PEAK FLOW

Flow Component	Average Daily Flow (gpd)	Peaking Factor	Peak Flow (gpd)
Current Wastewater	592,000	2.8	1,658,000
Current Infiltration	201,000	1.81	363,000
Current Inflow	57,000	-	3,859,000
Projected Wastewater	111,100	2.8	311,000
Projected Infiltration	39,300	1.81	71,000
Total	1,000,400		6,262,000

DEVELOPMENT OF PROJECTIONS OF FUTURE FLOWS – DISTRICT 2

As noted in the introduction, the capacity of the existing Route 7 WWTF has been fully allocated to the existing parcels that comprise the district, and each parcel owner has purchased their share of the plant capacity. The existing average daily flow at the Route 7 WWTF is approximately 0.054 mgd and the permitted design capacity for the WWTF is 0.12 mgd. There have been no public health or pollution issues identified by the Town from existing development in the area of Sewer District 2.

There are 42 parcels in Sewer District 2, and of those there are 4 parcels not yet connected to the collection system. The 2010 Ridgefield Plan of Conservation & Development notes the following related to the Route 7 WWTF:

> "Due to the limited flow of the Norwalk River, the sewer system at Routes 7/35 generally should be reserved for addressing public health concerns in existing residential areas. However, there may be circumstances where the Town considers allowing businesses to connect to the sewer. Connections should occur in

limited situations where well-defined economic development goals will be advanced."

Since the capacity of the WWTF is fully allocated to the existing parcels in the District, the projected increase in the average daily flow to the Route 7 WWTF would be from the development of undeveloped or underdeveloped parcels within the existing service area. No allowance for sewer extensions to serve parcels outside the existing sewer district has been included. Consequently, the projected future average daily flow for Sewer District 2 is the current plant capacity of 0.12 mgd.

Similarly, the projected future peak flow for the Route 7 WWTF would be the current plant peak flow capacity. The current peak flow is approximately 0.36 mgd. As noted in Technical Memorandum No. 6, both the Route 7 Influent Pump Station and the Route 7 WWTF headworks have a maximum capacity of 0.72 mgd. Consequently, the projected future average peak flow for Sewer District 2 is 0.72 mgd.

PROJECTED FUTURE WASTEWATER LOADS – SEWER DISTRICTS 1 AND 2

The existing average influent pollutant concentrations at the South Street WWTF for 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) are 219 mg/l and 232 mg/l, respectively. Multiplying these concentrations by the projected 2035 average daily flow of 1.00 mgd gives a BOD_5 load of approximately 1,830 lbs/day and a TSS load of 1,940 lbs/day. The existing influent total Kjeldahl nitrogen (TKN) and total phosphorus (TP) concentrations are 24.8 mg/l and 4.0 mg/l, respectively. The resultant projected 2035 average daily loads of TKN and TP are approximately 210 lbs/day and 35 lbs/day, respectively.

The existing average influent pollutant concentrations at the Route 7 WWTF for 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) are 280 mg/l and 226 mg/l, respectively. Multiplying these concentrations by the projected 2035 average daily flow of 0.120 mgd gives a BOD_5 load of approximately 280 lbs/day and a TSS load of 230 lbs/day. The existing influent total Kjeldahl nitrogen (TKN) and total phosphorus (TP) concentrations are 33.0 mg/l and 6.0 mg/l, respectively. The resultant projected 2035 average daily loads of TKN and TP are approximately 33 lbs/day and 6.0 lbs/day, respectively.

The projected average daily loads for the South Street and Route 7 WWTFs are summarized in Table 12.

TABLE 12. PROJECTED AVERAGE DAILY LOADS

WWTF	Existing Average Influent Concentration (gm/l)				Projected Average Daily Flow (mgd)	Projected Average Daily Load (lbs./day)			
	BOD_5	TSS	TKN	TP		BOD_5	TSS	TKN	TP
South St.	219	232	24.8	4.0	1.00	1,830	1,940	210	35
Route 7	280	226	33.0	6.0	0.12	280	230	33	6.0

APPENDIX A
TOWN OF RIDGEFIELD, CT
PLANIMETRICS
SEWER SERVICE AREA BUILDOUT ANALYSIS
JUNE 30, 2014

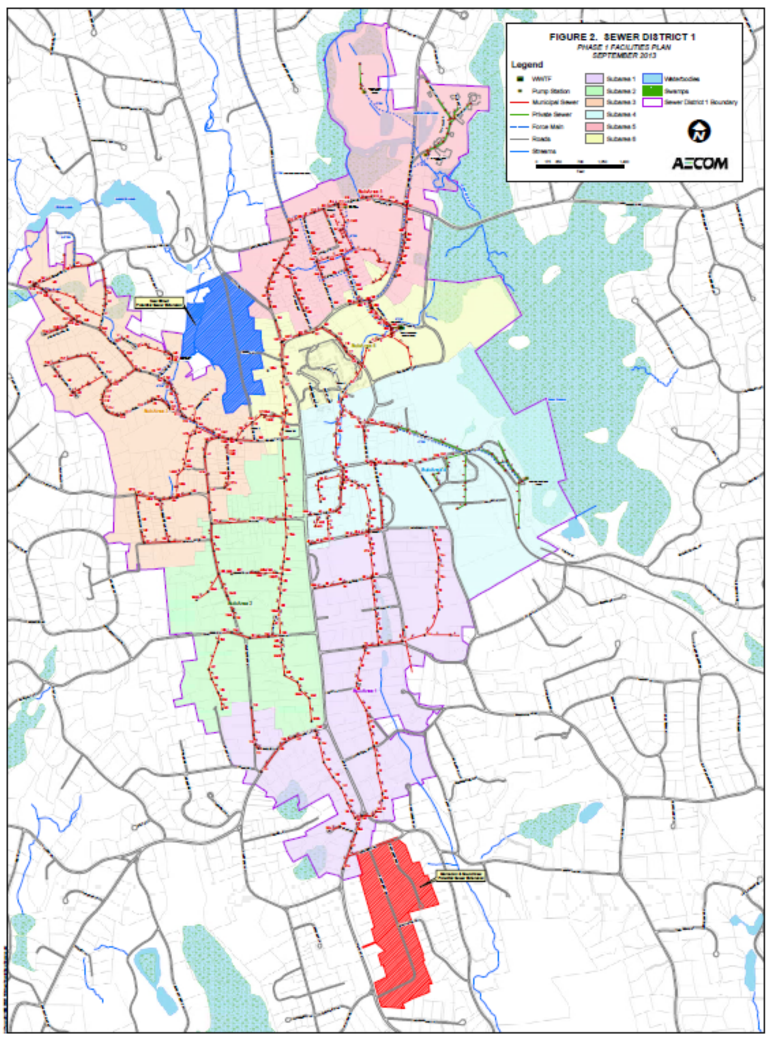
Planimetrics
70 County Road, Simsbury, CT 06070 860-913-4080

June 30, 2014

TOWN OF RIDGEFIELD, CT
SEWER SERVICE AREA BUILDOUT ANALYSIS

Overview

The Town of Ridgefield is considering improving the main sewage treatment facility in Ridgefield to improve its level of treatment and provide sewage capacity for existing users in the service area and two possible expansion areas.



Scope of Work

The Town of Ridgefield retained AECOM to prepare a Phase 1 Wastewater Facilities Plan for the Town of Ridgefield. As part of the facility planning effort, AECOM is responsible for making projections of future wastewater flow from Sewer District 1, which serves the center of Town.

The Town and AECOM sought a buildout analysis from Planimetrics for parcels within the existing Sewer District and two (2) additional areas where extension of the sewer system may be considered. Planimetrics was asked to estimate the possible development potential within these areas. It is understood that this estimate will assist in sizing the treatment facility and other infrastructure.

The analysis is based upon the current zoning and environmental constraints and estimates the potential for additional residential growth (in units) and commercial growth (in square footage of building). The analysis also identifies municipal and institutional uses so that the Town and AECOM can evaluate how much sewage capacity to allocate for those uses.

An Excel spreadsheet (named "Master_Table 063014") accompanies this report. The spreadsheet contains the data received from the Town and the buildout estimates prepared by Planimetrics.

Methodology

A. Data Receipt / Collection

1. Get GIS data from Town or AECOM in ESRI-compatible shapefile / geo-database format
2. Get GIS data from DEEP or other sources
3. Get Assessor database from Town in Excel spreadsheet format

The assessor database was relied upon for the parcel area, number of residential dwelling units and the floor area of buildings. Where changes to the assessor database were necessary or where better information was available, the cells were highlighted in yellow and/or information was placed in the notes column. The land use categorizations were not suitable for the intended purpose and were refined based on information in the assessor database and/or field investigation.

B. Data Analysis

1. Create Excel spreadsheet with data fields
2. Create basic land use map based on assessor codes
3. Create natural resources map
4. Create aerial photo working maps (as warranted) for field investigation

C. **Estimate Residential Buildout** (Predicated upon existing zoning only)

1. Estimate residential buildout potential of *vacant parcels* by evaluating whether one or more residential units could be established given zoning requirements and environmental constraints
2. Estimate residential buildout potential of *"over-sized" parcels* by evaluating whether one or more *additional* residential units could be established given zoning requirements and environmental constraints
3. Review buildout estimates with Town Staff and AECOM and adjust for approved developments, Staff knowledge of parcels, etc.

The buildout estimates are based on existing zoning regulations and requirements. No provision was made in the buildout estimates for expansion of existing residential units, addition of accessory apartments, conversion of single-family uses to two-family uses or three-family uses, or rezoning to a more intensive residential use. This potential for this type of cannot be assigned to an individual property and so has not been incorporated into the buildout estimates. The Town and/or AECOM may wish to carry an allowance to address the possibility of such activities in the future.

D. **Estimate Business Buildout** (Predicated upon existing zoning only)

1. Estimate buildout potential of *vacant parcels* (if any)
2. Estimate buildout potential of *developed parcels*
3. Review buildout estimates with Town Staff and AECOM and adjust for approved developments, Staff knowledge of parcels, etc.

The buildout estimates are based on existing zoning regulation and requirements. The analysis found that many properties are built to their maximum potential under existing zoning and that the yield (square feet of building per acre of land) could be used to identify properties with a particular use which might have additional development potential. In addition, this could also be used to determine the buildout potential if a use was to change from a less intensive use to a more intensive use.

E. **Documentation**

1. Prepare spreadsheet with buildout potential identified by map and lot number
2. Prepare summary report of methodology and results

Findings - Residential Buildout

The investigation found 2,209 residential units within the study area:
- 2,089 within the existing Sewer District #1,
- 66 within the Marcadon Soundview Future Sewer District, and
- 54 within the New Street Future Sewer District).

In addition, there are two properties on Nutmeg Court (#24 and #26) where the dwelling units are outside the service area but the accessways are within the sewer service area.

The investigation estimates that 119 additional residential units may be possible within the study area in the future:
- 109 within the existing Sewer District #1,
- 1 within the Marcadon Soundview Future Sewer District, and
- 9 within the New Street Future Sewer District.

Overall, it is estimated that the sewer district may eventually contain 2,328 residential units:
- 2,198 within the existing Sewer District #1,
- 67 within the Marcadon Soundview Future Sewer District, and
- 63 within the New Street Future Sewer District).

Please note that additional residential units may be possible on Town land (ie – Schlumberger), Housing Authority sites, or on institutional lands. The Town and/or AECOM may wish to carry an allowance to address the possibility of such activities in the future.

As was stated previously, this is based on the assessor database and the information reported there. Some adjustments were made based upon approved developments and other situations. Such adjustments were noted by highlighting the cell in yellow and/or adding clarification in the notes column. The buildout estimates are based on existing zoning regulations and requirements.

No provision was made in the buildout estimates for expansion of existing residential units, addition of accessory apartments, conversion of single-family uses to two-family uses or three-family uses, or rezoning to a more intensive residential use. This potential for this type of cannot be assigned to an individual property and so has not been incorporated into the buildout estimates. The Town and/or AECOM may wish to carry an allowance to address the possibility of such activities in the future.

Findings - Business Buildout

Most business properties appeared to be well utilized at the present time. However, each business category had a different "yield" factor (calculated as the number of square feet of building per acre of land). This value is a reflection of regulatory requirements (especially parking requirements) and other factors. From discussions with the Town and AECOM, it was determined that there might be two types of buildout potential:

- Type 1 = properties used less intensively than the average yield for their use category might have potential to support additional floor area, and
- Type 2 = properties in lower yield categories might, over time, be redeveloped to a higher yield category.

The average yield for the different use categories (sorted by yield) is presented below:

Use Type (column Y)	#	Floor Area (SF) (column AH)	Land Area (acres) (column M)	Average Yield (SF / acre)
Retail Store	25	249,061	20.16	12,354
Office	36	316,989	29.21	10,852
Community Shopping Center	4	219,391	21.87	10,032
Bank Branch	6	24,258	2.45	9,901
Warehouse	1	9,414	1.38	6,822
Gas/Convenience	4	13,354	2.13	6,269
Contractors Garage/Storage	1	5,763	0.94	6,131
Car Dealership / Repair Garage	4	26,472	4.59	5,767
Restaurant	6	34,499	6.17	5,591
Fuel Oil	2	6,468	1.21	5,345
Lodging	1	7,181	1.93	3,721
Lumberyard	1	28,000	7.83	3,576
TOTALS	**91**	**940,850**	**99.87**	**9,421**

There are approximately 42 properties which might have some *Type 1 buildout* potential (they are "underdeveloped" compared to other properties in their use category). Estimates indicate that this potential could total as much as 186,437 square feet. However, some potential floor area additions are less than 1,000 square feet and it might not be economical for a property owner to add such a modest amount of floor area. Similarly, some estimates of floor area additions might only be possible with removal of the existing building and redevelopment of the entire site in a different configuration, including adding additional floors. Whether it makes economic sense to redevelop the entire site including the loss of the rental income during the construction period is unclear. The Town and AECOM may wish to carry an allowance for some of these sites.

There are approximately 19 properties which might have some *Type 2 buildout* potential (the use category is "underdeveloped" compared to other use categories in Ridgefield). Estimates indicate that this potential may total 172,875 square feet. Over time, it is considered more likely that lower intensity uses might be converted to more intensive uses. For example, the lumberyard property at 29 Prospect Street (E14-0178) could be repurposed to office space or other uses with significantly more floor area than exists on the property today. Again, the Town and AECOM may wish to carry an allowance for the potential redevelopment of these sites.

Some properties have buildout potential in both categories. Overall, it is estimated there are 53 properties with some buildout potential (either Type 1 or Type 2) and the total buildout potential may result in total an additional 338,405 square feet. As stated previously, there are different levels of buildout potential and the Town and AECOM may wish to carry an allowance for future development of these sites.

As was stated previously, this is based on the assessor database and the information reported there. Some adjustments were made based upon approved developments and other situations. Such adjustments were noted by highlighting the cell in yellow and/or adding clarification in the notes column. The buildout estimates are based on existing zoning regulations and requirements.

Findings – Mixed Use Buildout

Mixed use properties are unique because they contain both residential units and business floor area. As the following table indicates, the average yield for mixed use properties is lower than the Town-wide average yield of 9,421 square feet per acre. However any additional yield potential is essentially consumed by floor space utilized by the residential uses.

Use Type (column Y)	#	Floor Area (SF) (column AH)	Land Area (acres) (column M)	Average Yield (SF / acre)
Mixed Use	24	154,373	16.65	9,271

Overall, the mixed use properties in Ridgefield are not felt to have significant additional development potential.

Findings - Institutional Buildout

A number of properties within the sewer district are owned by the Town of Ridgefield or by institutional uses (municipal facilities, housing authority, churches, museums, parochial schools, etc.).

Following discussions with the Town and AECOM, it was decided that the potential future use and/or estimates of future development potential of these properties will occur separately as the sewer planning process unfolds.

At this time, the most significant parcel in terms of buildout potential may be the Schlumberger parcel at 36 Old Quarry Road recently acquired by the Town (E14-0162). Under current zoning, the property may have potential for 60 residential units on the 10 acres zoned MFDD (or perhaps up to 80 units if affordable housing is included) and potential for approximately 80,000 square feet of office space on the 20 acres in the B-2 district (the yield is moderated at this site due to the steep slopes and environmental constraints. **These buildout estimates on the Town-owned Schlumberger parcels ARE NOT INCLUDED in the Excel spreadsheet at this time pending further input from the Town of Ridgefield**.

Excel Spreadsheet

The land use information and buildout information is represented in an Excel spreadsheet entitled "Master Table 063014". The spreadsheet contains 36 columns and 1,599 rows.

The columns in the Excel spreadsheet are described below (key data fields highlighted in yellow and key work products are highlighted in green):

Column	Column Title	Description
A	Parcel_ID	The parcel identification number as contained in the Assessor's database
B	Feat_type	The type of feature (generally a "PARCEL", a "CONDO", or "CONDOMAIN" which is the common land in a condominium)
C	Sublot	No data provided (not used)
D	OtherIDTex	Information provided from the Assessor's database (not used)
E	OtherIDNum	Information provided from the Assessor's database (not used)
F	StreetAddr	The street address of the property as contained in the Assessor's database
G	LandUse	The land use category as contained in the Assessor's database
H	LandUse2	Information provided from the Assessor's database
I	ID2	The parcel ID as contained in the Assessor's database with the dash removed (ie – E14-0152 becomes E140152) (not used)
J	Uniqueid_V	Same as ID2 but not for all parcels (not used)
K	Name_Value	Owner's Name
L	Name2_Valu	Name of second owner (if any) (not used)
M	Acres_Valu	The parcel area as contained in the Assessor's database
N	TotalLivin	The floor area of buildings on the property as contained in the Assessor's database
O	BuildingNu	The number designation for the specific building on a property as contained in the Assessor's database
P	TotalBuild	The number of buildings on the property as contained in the Assessor's database
Q	Unit_Value	No data provided (not used)
R	IncomeExpe	No data provided (not used)
S	ComSize_Va	No data provided (not used)
T	PropertyTy	Letter code for property type (not used)
U	Zone_Value	Letter code for zone type
V	BuildingTy	The building type designation as contained in the Assessor's database (not used)
W	NoOfFamili	The number of residential units on the property as contained in the Assessor's database

Column	Column Title	Description
X	ComUnits_V	No data provided (not used)
Y	Final_Land	The final land use category based upon the Assessor's database and additional refinement and clarification by Planimetrics
Z	Town Facility Code	A unique letter code assigned by Planimetrics to distinguish Town facilities (for mapping purposes)
AA	Town Land Code	A unique number code assigned by Planimetrics to distinguish Town land (for mapping purposes)
AB	GIS_Acre	An estimate of the parcel area as represented by the GIS polygon (not used)
AC	SD_NAME	The name of the sewer district the parcel falls within (Sewer District #1, Marcadon Soundview FSD, or New Street FSD)
AD	Existing Residential Units	The estimate of the number of existing residential units based upon the Assessor's database and other Town records
AE	Potential Future Residential Units	The estimate of the number of possible future residential units (ie – *in addition to* the units reported in column AD) based upon the Assessor's database, Planimetrics investigation, other Town information, and discussion with Town Staff
AF	Residential - MLS	A column reporting the minimum lot size requirement based on the Assessors zoning designation (column U)
AG	Theoretical of Lots	A calculation which divides the parcel area by the minimum lot size requirement to estimate if there is sufficient land area to support an additional lot
AH	Non-Residential Floor Area	For non-residential uses, the floor area of the building based upon the Assessor's database or other sources. For mixed uses, an estimate of the floor area of the building devoted to non-residential uses based upon the Assessor's database or other sources.
AI	Buildout_Potential	For residential uses, a "Yes" or "No" designation for whether the property is estimated to have the potential to support additional residential units. For non-residential uses, an identification of the category of land use (ie – "Business", "Institutional", "Mixed Use", "Parking", "State", "Town", "Utility", Unknown")
AJ	B_Buildout Type 1 - Low Yield Site	If the yield (SF/acre) for a particular site is lower than the average yield for that use category, the model assumes that the parcel may be "underdeveloped" and may have capacity for additional floor area within that use category. The number value is the number of *additional square feet* that might be able to be built on that site to achieve the average yield for the use category

Column	Column Title	Description
AK	B_BuildoutType 2 - Low Yield Category	If the yield (SF/acre) for a use category is lower than the average yield for all business use categories in Ridgefield, the model assumes that the parcel may be "redeveloped" in the future since it could support more floor area. The number value is the number of ***additional square feet*** that might be able to be built on that site to achieve the average yield for all business use categories in Ridgefield (9,421 SF/acre)
AL	Final_ B_Buildout	The higher value between column AJ and column AK
AM	B_Buildout_Type	A designation of which calculation produced the higher value of buildout potential: • Type 1 – Site • Type 2 – Category • No
AN	Notes	A description of information used to refine the findings of the report.

Town of Ridgefield - Land

Parcel_ID	StreetAddr	Name_Value	Name2_Valu	Acres_Valu	TotalLivin	Final_Land	Town Facility
E13-0175	193 DANBURY RD					Town Facility	A
E14-0182	GROVE ST	RIDGEFIELD TOWN OF		1.28	1,841	Town Facility	B
E15-0262	316 MAIN ST	TOWN OF RIDGEFIELD		4.18	8,992	Town Facility	C
F15-0056	PROSPECT ST EXT					Town Facility	D
E15-0203	90 EAST RIDGE	RIDGEFIELD TOWN OF		16	69,249	Town Facility - Annex / Gymnasium / C	E
E15-0151	316 MAIN ST	RIDGEFIELD TOWN OF		13.01	49,239	Town Facility - Elementary School	F
E15-0098	CATOONAH ST	RIDGEFIELD TOWN OF		0.32	12,316	Town Facility - Fire Station (Staffed)	G
E16-0095	EAST RIDGE	RIDGEFIELD TOW	EAST RIDGE JR HIGH SCHOOL	14.17	183,028	Town Facility - Junior High School	H
E15-0204	76 EAST RIDGE	RIDGEFIELD TOWN OF		1.9	10,921	Town Facility - Police Station	I
E14-0162	36 OLD QUARRY RD	RIDGEFIELD TOWN OF		30.4	22,362	Town Facility - Schlumberger Site	J
E14-0158	SOUTH ST	RIDGEFIELD TOWN OF		32.35	14,215	Town Facility - Sewer Plant / Town Gar	K
F15-0059	HALPIN LA	RIDGEFIELD TOWN OF		0.21	3,111	Town Facility - Theater barn	L
E15-0160	400 MAIN ST	RIDGEFIELD TOWN OF		0.4	11,008	Town Facility - Town Hall	M
F15-0061	34 HALPIN LA	RIDGEFIELD TOWN OF		9.88	2,598	Town Facility / Ridgefield Guild of Artis	N

Town of Ridgefield - Land

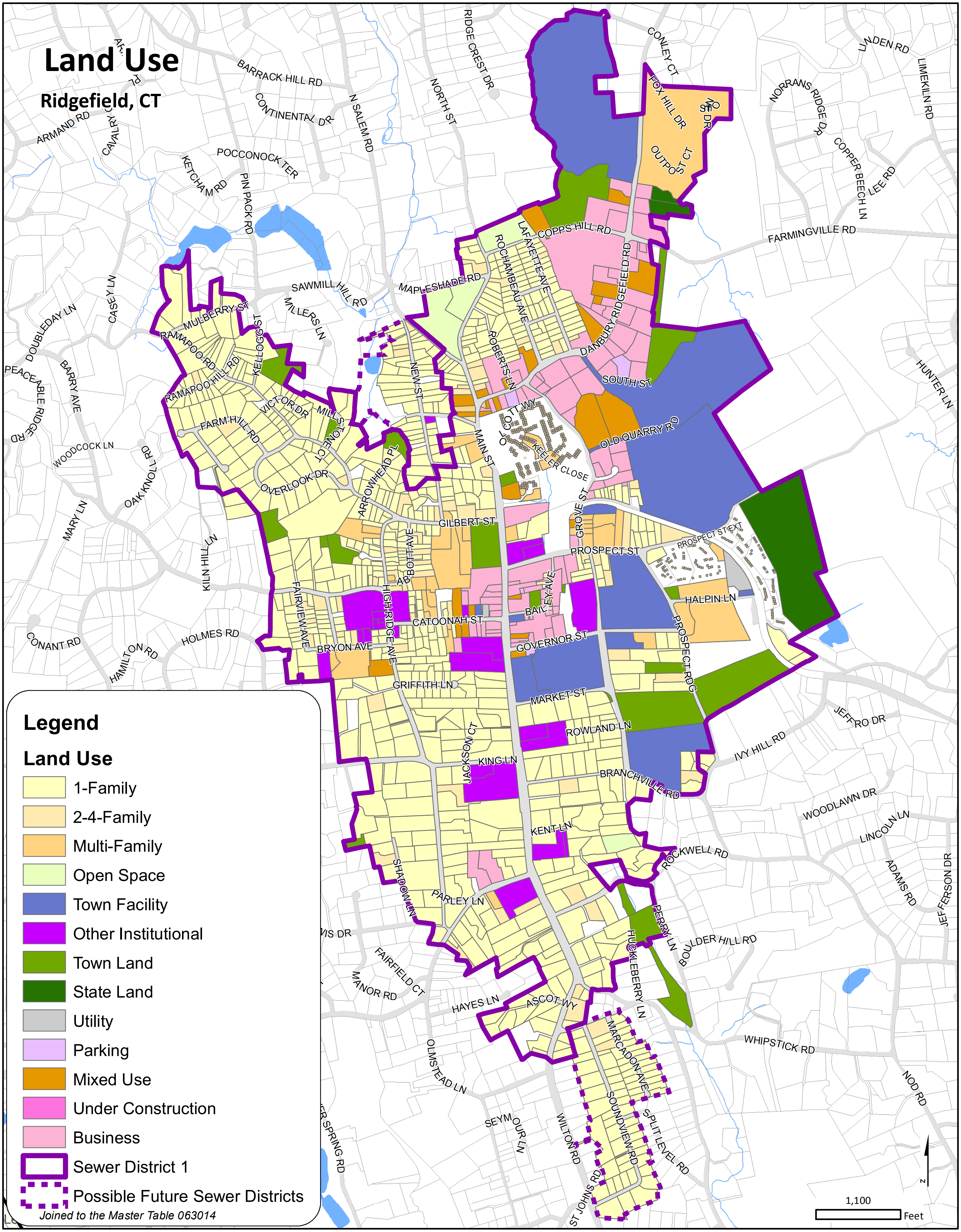
Town Land C	Existing Residentia	Potential Future Reside	Non-Residential Flo	Buildout_Potential	Notes
	0			Town	
	1		1,841	Town	
	1		8,992	Town	
	0			Town	
	0		69,249	Town	
	0		49,239	Town	
	0		12,316	Town	
	0		183,028	Town	
	0		10,921	Town	
	0		22,362	Town	60 to 80 residential units allowed on 10 acres rezoned to MFDD
	0		14,215	Town	
	0		3,111	Town	
	0		11,008	Town	
	1		2,598	Town	

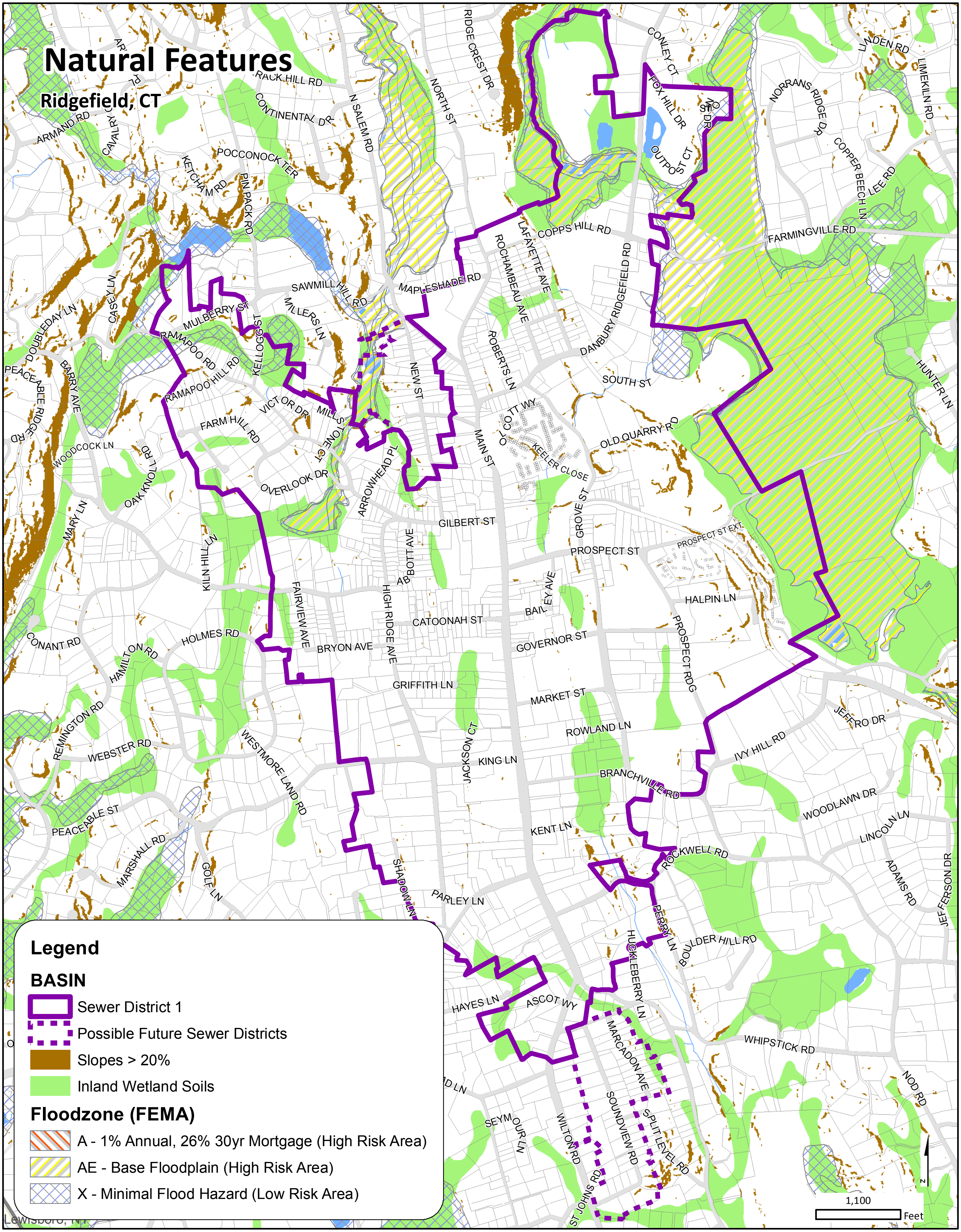
Town of Ridgefield - Land

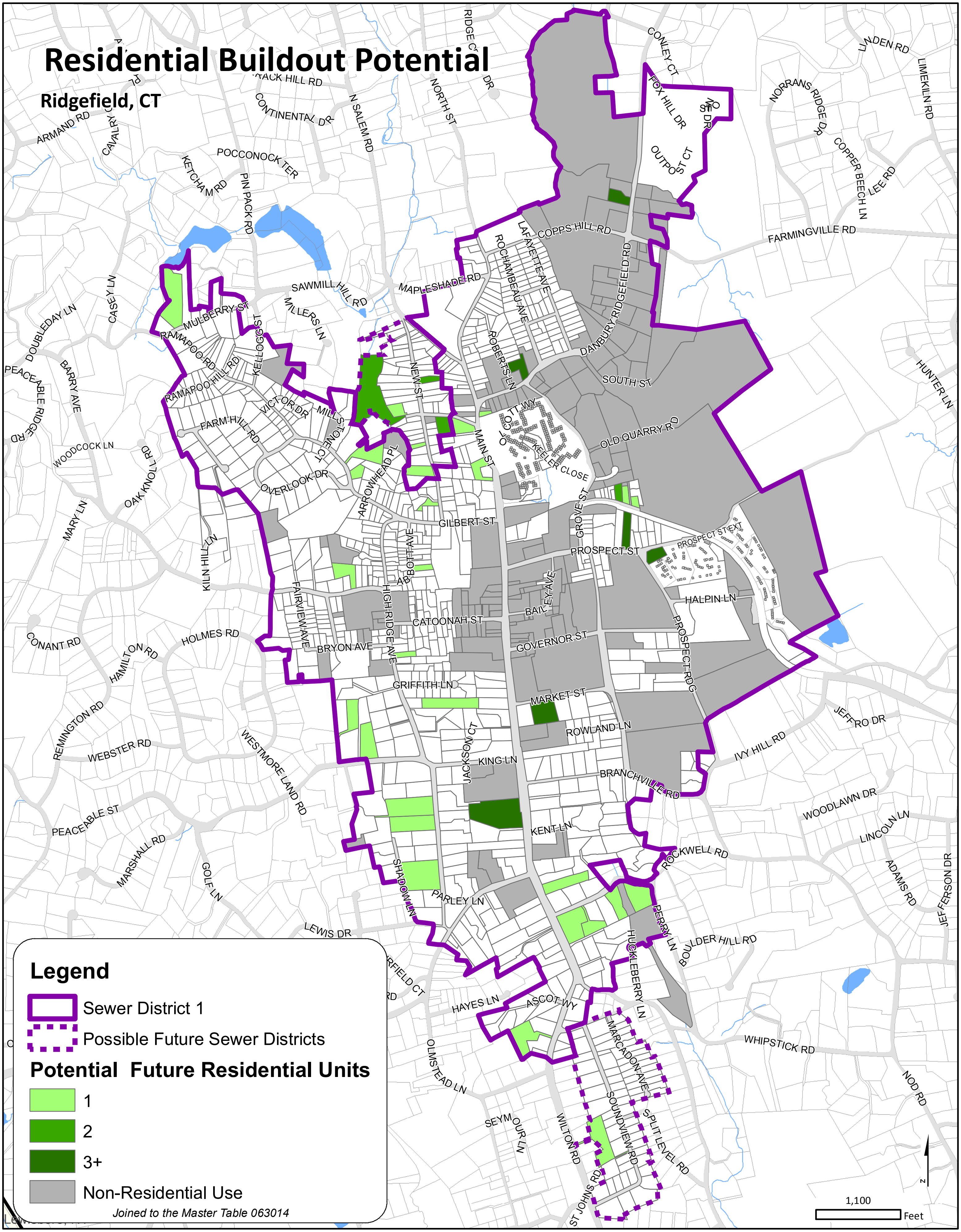
Parcel_ID	StreetAddr	Name_Value	Acres_Valu	TotalLivin	Final_Land	Town Facility	Town Land C
D14-0144	RAMAPOO RD				Town Land		1
D14-0152	KELLOGG ST				Town Land		2
D14-0167	BARRY AVE				Town Land		3
D14-0172	BARRY AVE				Town Land		4
E13-0165	COPPS HILL				Town Land		5
E13-0171	COPPS HILL RD				Town Land		6
E14-0114	GILBERT ST				Town Land		7
E14-0120	MAIN ST				Town Land		8
E14-0229	ARROWHEAD PL				Town Land		9
E15-0209	EAST RIDGE				Town Land		10
E15-0225	LANDLOCKED				Town Land		11
E15-0233	EAST RIDGE				Town Land		12
E15-0259	MULVANEY CT				Town Land		13
E16-0143	SHADOW LA				Town Land		14
E16-0152	ROCKWELL RD				Town Land		15
F13-0036	FARMINGVILLE RD				Town Land		16
F15-0010	PROSPECT RIDGE				Town Land		17
F15-0050	PROSPECT RIDGE				Town Land		18
F15-0065	PROSPECT RIDGE				Town Land		19
F16-0013	PERRY LA				Town Land		20
E15-0167	BAILEY AVE				Town Land - CBD Parking		21
E15-0191	GOVERNOR ST				Town Land - Parking		22

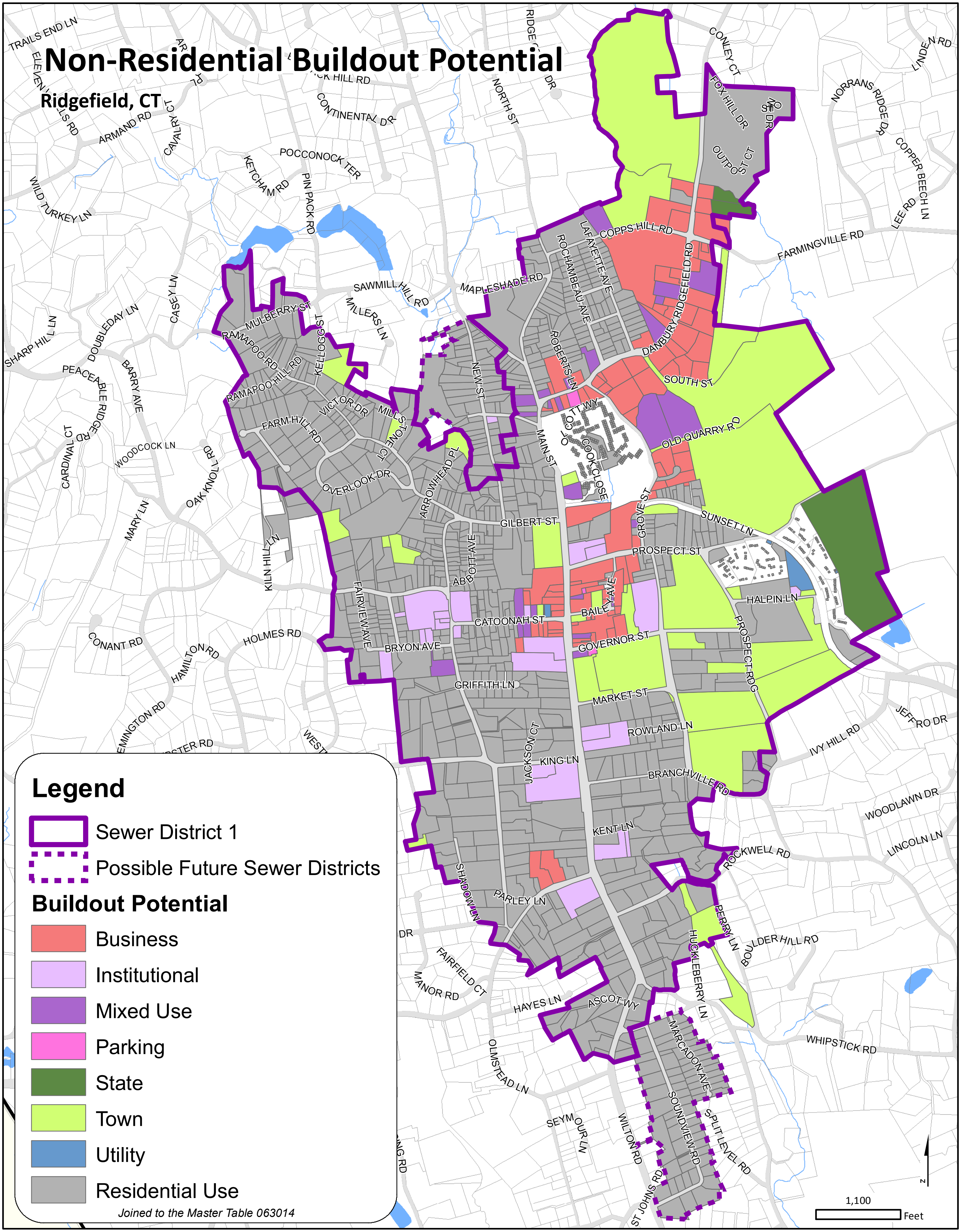
Town of Ridgefield - Land

Existing Residentia	Potential Future Reside	Non-Residential Flo	Buildout_Potential	Notes
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	Ballard Park
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	
0			Town	

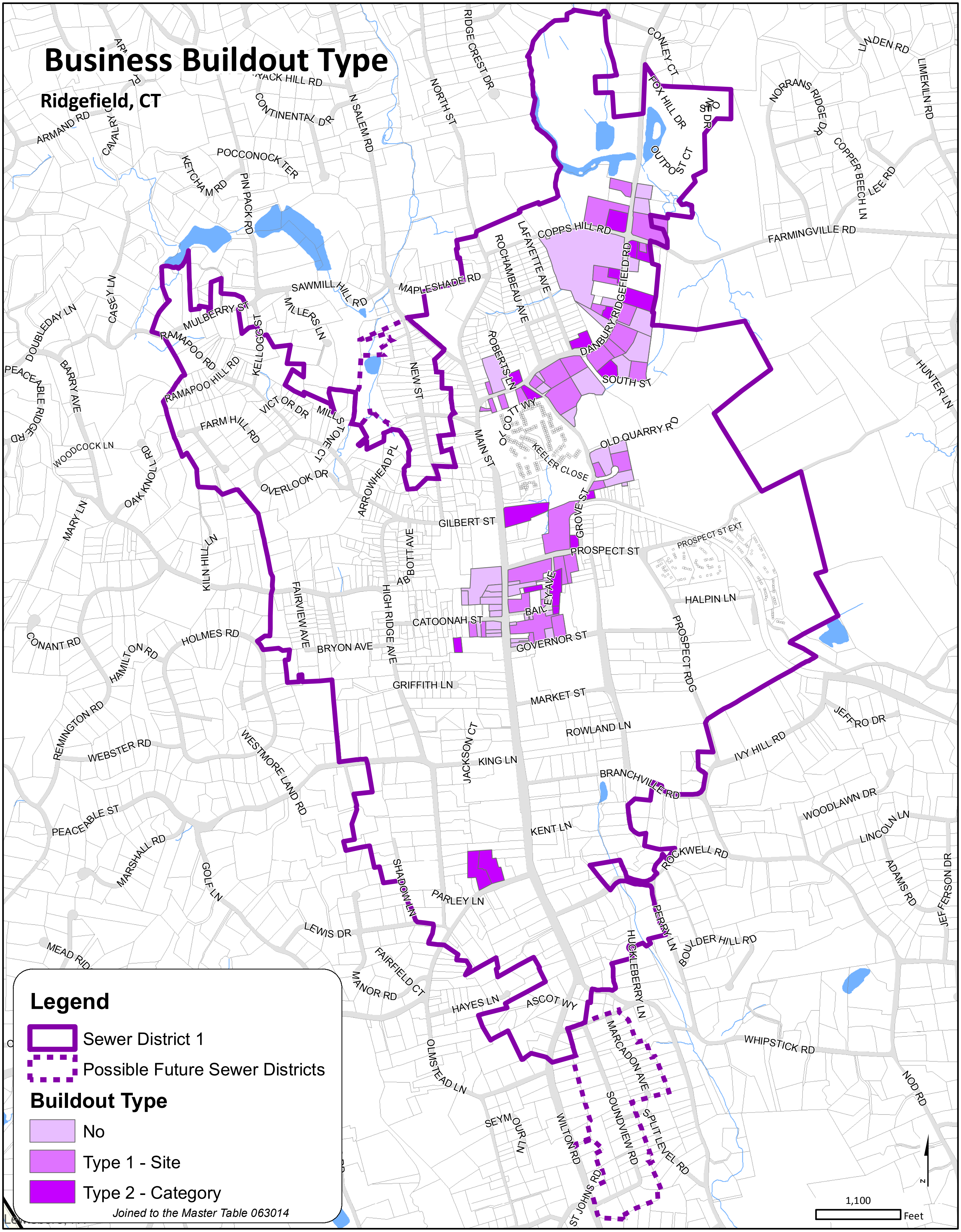

Land Use
Ridgefield, CT
Legend
Land Use
1-Family
2-4-Family
Multi-Family
Open Space
Town Facility
Other Institutional
Town Land
State Land
Utility
Parking
Mixed Use
Under Construction
Business
Sewer District 1
Possible Future Sewer Districts
Joined to the Master Table 063014
1,100
Feet
N
BARRACK HILL RD
CONTINENTAL DR
N SALEM RD
NORTH ST
RIDGE CREST DR
CONLEY CT
FOX HILL DR
OUTPOST CT
LINDEN RD
LIMEKILN RD
NORRANS RIDGE DR
COPPER BEECH LN
LEE RD
FARMINGVILLE RD
ARMAND RD
CAVALRY CT
POCCONOCK TER
KETCHAM RD
PIN PACK RD
SAWMILL HILL RD
MAPLESHADE RD
COPPS HILL RD
ROCHAMBEAU AVE
LAFAYETTE AVE
RIDGEFIELD RD
DANBURY RD
SOUTH ST
OLD QUARRY RD
MILLERS LN
MULBERRY ST
RAMAPOO RD
RAMAPOO HILL RD
KELLOGG ST
VICTOR DR
FARM HILL RD
MILLSTONE CT
OVERLOOK DR
ARROWHEAD PL
NEW ST
ROBERTS LN
MAIN ST
KEELER CLOSE
GILBERT ST
GROVE ST
PROSPECT ST
PROSPECT ST EXT
HALPIN LN
DOUBLEDAY LN
CASEY LN
BARRY AVE
PEACEABLE RIDGE RD
WOODCOCK LN
OAK KNOLL RD
MARY LN
KILN HILL LN
HUNTER LN
CONANT RD
HOLMES RD
HAMILTON RD
FAIRVIEW AVE
HIGH RIDGE AVE
BRYON AVE
CATOONAH ST
BAILEY AVE
GOVERNOR ST
GRIFFITH LN
PROSPECT RDG
MARKET ST
ROWLAND LN
JEFFRO DR
IVY HILL RD
JACKSON CT
KING LN
BRANCHVILLE RD
WOODLAWN DR
LINCOLN LN
KENT LN
ROCKWELL RD
ADAMS RD
JEFFERSON DR
SHADOW LN
PARLEY LN
PERRY LN
BOULDER HILL RD
FAIRFIELD CT
MANOR RD
HAYES LN
ASCOT WY
HUCKLEBERRY LN
WHIPSTICK RD
OLMSTEAD LN
MARCADON AVE
SOUNDVIEW RD
SPLIT LEVEL RD
SEYMOUR LN
WILTON RD
ST JOHNS RD
NOD RD

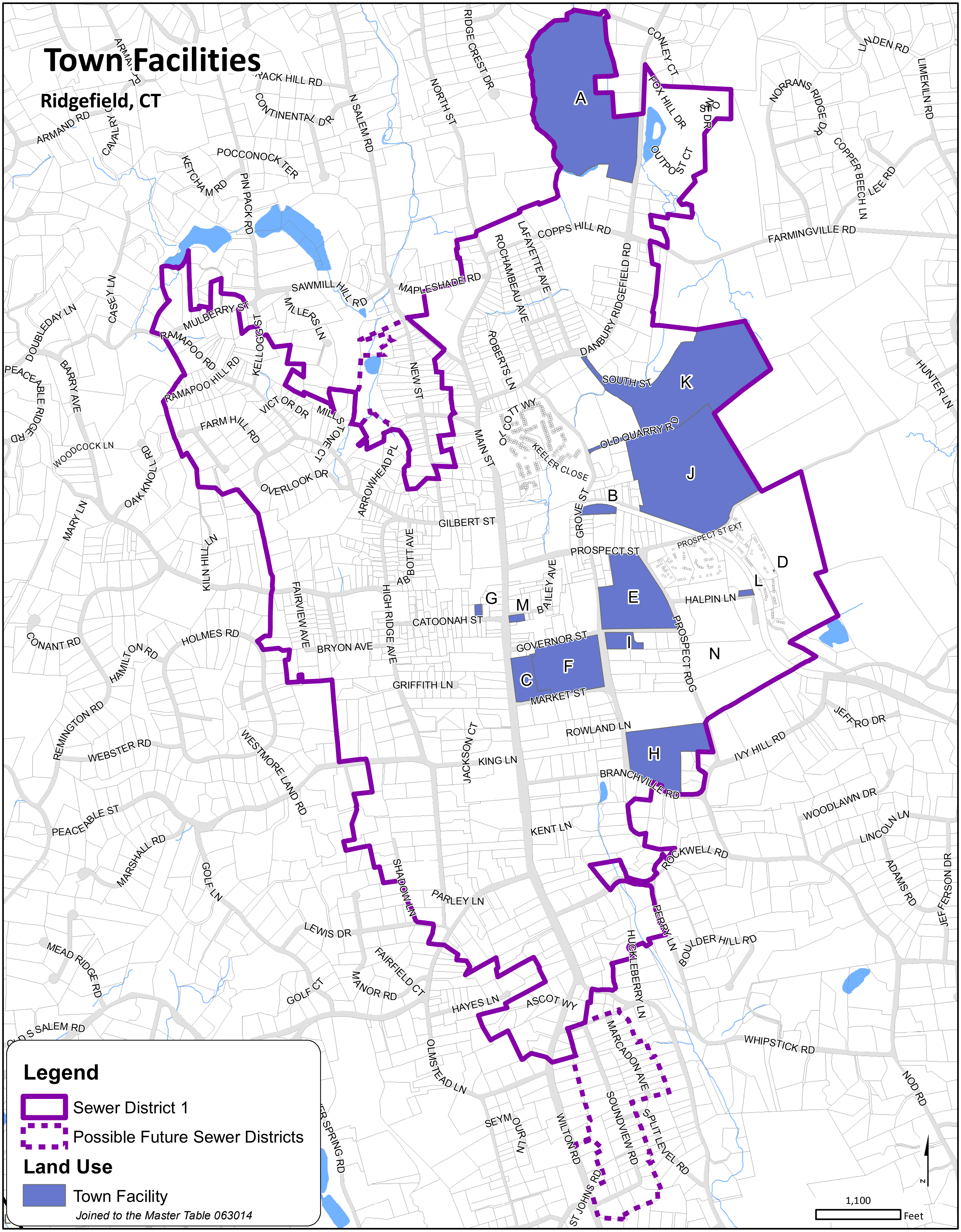

Natural Features
Ridgefield, CT
Legend
BASIN
Sewer District 1
Possible Future Sewer Districts
Slopes > 20%
Inland Wetland Soils
Floodzone (FEMA)
A - 1% Annual, 26% 30yr Mortgage (High Risk Area)
AE - Base Floodplain (High Risk Area)
X - Minimal Flood Hazard (Low Risk Area)
1,100
Feet
N
MAIN ST
NORTH ST
N SALEM RD
RIDGE CREST DR
CONLEY CT
FOX HILL DR
OUTPOST CT
NORRANS RIDGE DR
LINDEN RD
LIMEKILN RD
COPPER BEECH LN
LEE RD
FARMINGVILLE RD
COPPS HILL RD
ROCHAMBEAU AVE
LAFAYETTE AVE
DANBURY RIDGEFIELD RD
MAPLESHADE RD
SAWMILL HILL RD
MILLERS LN
CONTINENTAL DR
POCCONOCK TER
KETCHAM RD
PIN PACK RD
ARMAND RD
CAVALRY
MULBERRY ST
KELLOGG ST
RAMAPOO RD
RAMAPOO HILL RD
DOUBLEDAY LN
CASEY LN
BARRY AVE
PEACEABLE RIDGE RD
FARM HILL RD
VICTOR DR
MILLSTONE CT
NEW ST
ROBERTS LN
SOUTH ST
HUNTER LN
OLD QUARRY RD
KEELER CLOSE
WOODCOCK LN
OAK KNOLL RD
OVERLOOK DR
ARROWHEAD PL
GILBERT ST
GROVE ST
PROSPECT ST
PROSPECT ST EXT
MARY LN
KILN HILL LN
ABBOTT AVE
FAIRVIEW AVE
HIGH RIDGE AVE
CATOONAH ST
BAILEY AVE
HALPIN LN
CONANT RD
HOLMES RD
BRYON AVE
GOVERNOR ST
PROSPECT RDG
HAMILTON RD
GRIFFITH LN
MARKET ST
JEFFRO DR
REMINGTON RD
WESTMORE LAND RD
ROWLAND LN
WEBSTER RD
JACKSON CT
KING LN
IVY HILL RD
BRANCHVILLE RD
WOODLAWN DR
PEACEABLE ST
KENT LN
LINCOLN LN
MARSHALL RD
GOLF LN
ROCKWELL RD
ADAMS RD
JEFFERSON DR
SHADOW LN
PARLEY LN
PERRY LN
HUCKLEBERRY LN
BOULDER HILL RD
HAYES LN
ASCOT WY
MARCADON AVE
WHIPSTICK RD
SOUNDVIEW RD
SPLIT LEVEL RD
SEYMOUR LN
WILTON RD
ST JOHNS RD
NOD RD
Lewisboro, NY


Residential Buildout Potential
Ridgefield, CT
Legend
Sewer District 1
Possible Future Sewer Districts
Potential Future Residential Units
1
2
3+
Non-Residential Use
Joined to the Master Table 063014
1,100
Feet
N
ARMAND RD
CAVALRY
CONTINENTAL DR
N SALEM RD
NORTH ST
RIDGE CT
CONLEY CT
FOX HILL DR
OUTPOST CT
NORRANS RIDGE DR
LINDEN RD
LIMEKILN RD
COPPER BEECH LN
LEE RD
POCCONOCK TER
KETCHAM RD
PIN PACK RD
COPPS HILL RD
FARMINGVILLE RD
SAWMILL HILL RD
MAPLESHADE RD
ROCHAMBEAU AVE
LAFAYETTE AVE
RIDGEFIELD RD
DANBURY
CASEY LN
DOUBLEDAY LN
MULBERRY ST
MILLERS LN
KELLOGG ST
RAMAPOO RD
RAMAPOO HILL RD
PEACEABLE RIDGE RD
BARRY AVE
NEW ST
ROBERTS LN
SOUTH ST
HUNTER LN
FARM HILL RD
VICTOR DR
MILLSTONE CT
KEELER CLOSE
OLD QUARRY RD
WOODCOCK LN
OAK KNOLL RD
OVERLOOK DR
ARROWHEAD PL
MAIN ST
GILBERT ST
GROVE ST
MARY LN
KILN HILL LN
BOTT AVE
PROSPECT ST
PROSPECT ST EXT
HIGH RIDGE AVE
FAIRVIEW AVE
HALPIN LN
CATOONAH ST
BAILEY AVE
CONANT RD
HOLMES RD
BRYON AVE
GOVERNOR ST
PROSPECT RDG
HAMILTON RD
GRIFFITH LN
MARKET ST
REMINGTON RD
WESTMORE LAND RD
JACKSON CT
ROWLAND LN
JEFFRO DR
WEBSTER RD
KING LN
IVY HILL RD
BRANCHVILLE RD
PEACEABLE ST
KENT LN
WOODLAWN DR
LINCOLN LN
MARSHALL RD
ROCKWELL RD
ADAMS RD
JEFFERSON DR
GOLF LN
SHADOW LN
PARLEY LN
PERRY LN
LEWIS DR
BOULDER HILL RD
HUCKLEBERRY LN
HAYES LN
ASCOT WY
MARCADON AVE
WHIPSTICK RD
OLMSTEAD LN
NOD RD
SEYMOUR LN
WILTON RD
SOUNDVIEW RD
SPLIT LEVEL RD
ST JOHNS RD



Non-Residential Buildout Potential
Ridgefield, CT
Legend
Sewer District 1
Possible Future Sewer Districts
Buildout Potential
Business
Institutional
Mixed Use
Parking
State
Town
Utility
Residential Use
Joined to the Master Table 063014
1,100
Feet
N


Business Buildout Type
Ridgefield, CT
Legend
Sewer District 1
Possible Future Sewer Districts
Buildout Type
No
Type 1 - Site
Type 2 - Category
Joined to the Master Table 063014
1,100
Feet
N
RIDGE CREST DR
CONLEY CT
LINDEN RD
LIMEKILN RD
CONTINENTAL DR
N SALEM RD
NORTH ST
FOX HILL DR
NORRANS RIDGE DR
ARMAND RD
CAVALRY
POCCONOCK TER
KETCHAM RD
PIN PACK RD
OUTPOST CT
COPPER BEECH LN
LEE RD
COPPS HILL RD
FARMINGVILLE RD
ROCHAMBEAU AVE
LAFAYETTE AVE
RIDGEFIELD RD
SAWMILL HILL RD
MAPLESHADE RD
CASEY LN
DOUBLEDAY LN
MULBERRY ST
MILLERS LN
RAMAPOO RD
KELLOGG ST
ROBERTS LN
DANBURY RD
PEACEABLE RIDGE RD
BARRY AVE
RAMAPOO HILL RD
NEW ST
SOUTH ST
HUNTER LN
VICTOR DR
MILLSTONE CT
JOCOTT WY
FARM HILL RD
WOODCOCK LN
OAK KNOLL RD
MAIN ST
KEELER CLOSE
OLD QUARRY RD
ARROWHEAD PL
OVERLOOK DR
MARY LN
GILBERT ST
GROVE ST
ABBOTT AVE
PROSPECT ST
PROSPECT ST EXT
KILN HILL LN
FAIRVIEW AVE
HIGH RIDGE AVE
BAILEY AVE
HALPIN LN
CATOONAH ST
CONANT RD
HOLMES RD
BRYON AVE
GOVERNOR ST
PROSPECT RDG
HAMILTON RD
GRIFFITH LN
MARKET ST
JEFFRO DR
REMINGTON RD
WESTMORE LAND RD
ROWLAND LN
WEBSTER RD
JACKSON CT
KING LN
IVY HILL RD
BRANCHVILLE RD
PEACEABLE ST
KENT LN
WOODLAWN DR
LINCOLN LN
MARSHALL RD
ROCKWELL RD
GOLF LN
SHADOW LN
ADAMS RD
JEFFERSON DR
PARLEY LN
PERRY LN
LEWIS DR
BOULDER HILL RD
MEAD RIDGE
HUCKLEBERRY LN
FAIRFIELD CT
MANOR RD
HAYES LN
ASCOT WY
MARCADON AVE
WHIPSTICK RD
OLMSTEAD LN
NOD RD
SEYMOUR LN
WILTON RD
SOUNDVIEW RD
SPLIT LEVEL RD
ST JOHNS RD


Town Facilities
Ridgefield, CT
Legend
Sewer District 1
Possible Future Sewer Districts
Land Use
Town Facility
Joined to the Master Table 063014
1,100
Feet
N
A
B
C
D
E
F
G
H
I
J
K
L
M
N
CONLEY CT
LINDEN RD
LIMEKILN RD
NORRANS RIDGE DR
FOX HILL DR
OUTPOST CT
COPPER BEECH LN
LEE RD
FARMINGVILLE RD
COPPS HILL RD
RIDGE CREST DR
NORTH ST
N SALEM RD
CONTINENTAL DR
ARMAND RD
CAVALRY
POCCONOCK TER
KETCHAM RD
PIN PACK RD
SAWMILL HILL RD
MAPLESHADE RD
ROCHAMBEAU AVE
LAFAYETTE AVE
DANBURY RIDGEFIELD RD
SOUTH ST
OLD QUARRY RD
HUNTER LN
CASEY LN
DOUBLEDAY LN
MULBERRY ST
RAMAPOO RD
RAMAPOO HILL RD
KELLOGG ST
MILLERS LN
NEW ST
ROBERTS LN
MAIN ST
PEACEABLE RIDGE RD
BARRY AVE
VICTOR DR
MILL STONE CT
FARM HILL RD
WOODCOCK LN
OAK KNOLL RD
OVERLOOK DR
ARROWHEAD PL
KEELER CLOSE
GROVE ST
GILBERT ST
PROSPECT ST
PROSPECT ST EXT
MARY LN
KILN HILL LN
ABBOTT AVE
BAILEY AVE
HALPIN LN
CONANT RD
HOLMES RD
FAIRVIEW AVE
HIGH RIDGE AVE
CATOONAH ST
GOVERNOR ST
PROSPECT RDG
HAMILTON RD
BRYON AVE
GRIFFITH LN
MARKET ST
REMINGTON RD
WESTMORE LAND RD
JACKSON CT
ROWLAND LN
JEFFRO DR
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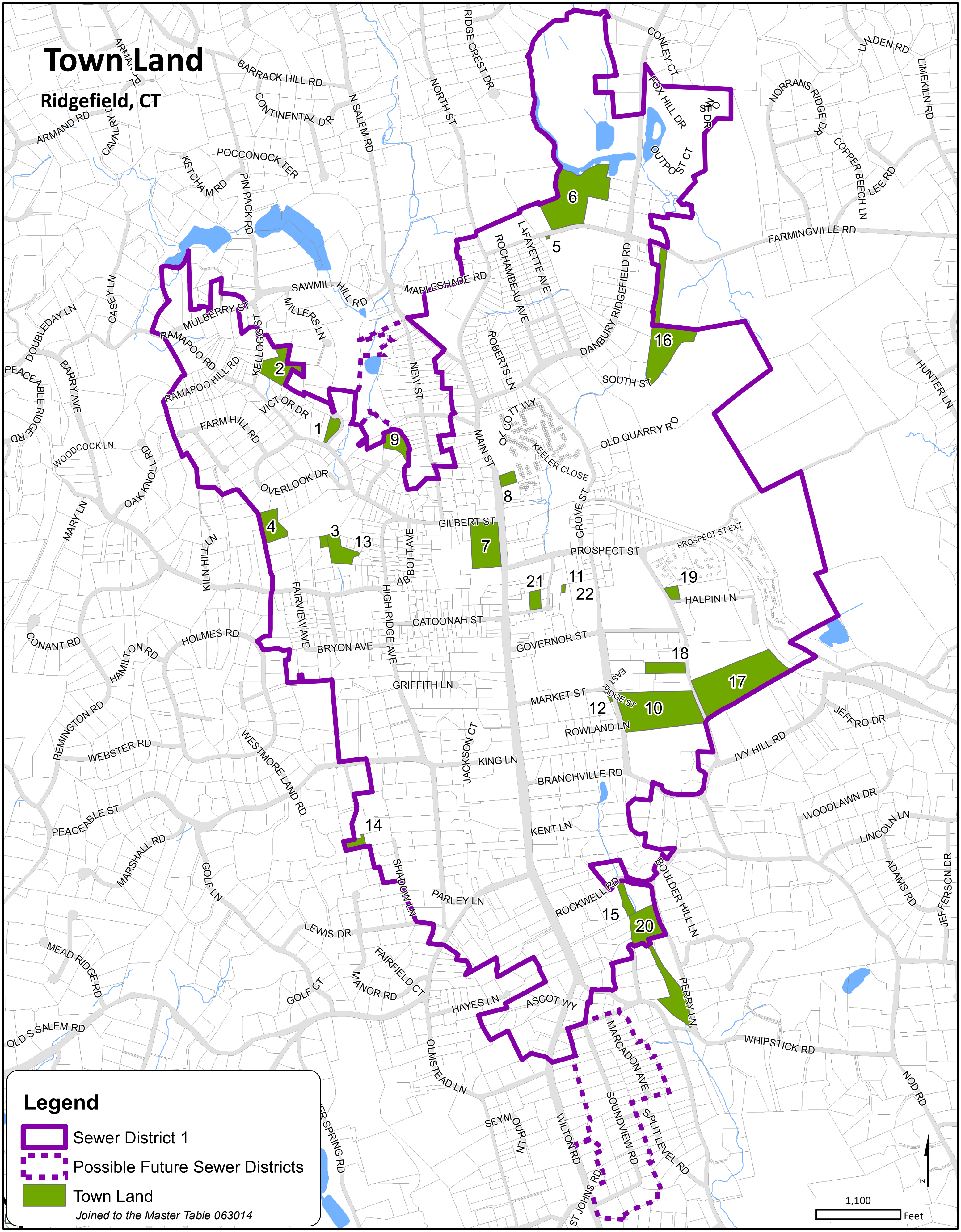
IVY HILL RD
BRANCHVILLE RD
PEACEABLE ST
WOODLAWN DR
KENT LN
LINCOLN LN
MARSHALL RD
GOLF LN
ROCKWELL RD
ADAMS RD
JEFFERSON DR
SHADOW LN
PARLEY LN
PERRY LN
LEWIS DR
BOULDER HILL RD
MEAD RIDGE RD
FAIRFIELD CT
HUCKLEBERRY LN
MANOR RD
GOLF CT
HAYES LN
ASCOT WY
OLD S SALEM RD
WHIPSTICK RD
MARCADON AVE
OLMSTEAD LN
NOD RD
SEYMOUR LN
WILTON RD
SOUNDVIEW RD
SPLIT LEVEL RD
ST JOHNS RD

Town Land

Ridgefield, CT



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APPENDIX B
MEMORANDA FROM RIDGEFIELD DIRECTOR OF HEALTH

AREAS OF TOWN THAT SHOULD RECEIVE PRIORITY CONSIDERATION FOR SEWER
JANUARY 16, 2014

PRIORITY AREAS FOR SEWER EXPANSION
JANUARY 29, 2015

TO: Rudy Marconi, First Selectman

FROM: Edward Briggs, Director of Health

RE: Areas of Town That Should Receive Priority Consideration for Sewer

DATE: January 16, 2014

The following is a list of properties that have had septic failures. They are in areas consisting of hardpan soils and high groundwater tables. Repairs required special designs by engineers, curtain drains, and/or large fill profiles. The streets listed should receive priority consideration for sewer extension.

SOUNDVIEW ROAD

House #	Date of Repair
17	1997
27	1979
39	2002
42	1998
46	2010
58	2002
61	2008
66	1998
81	2110
94	Active
96	1996
104	1989
105	1998
109	1979

MARCARDON

House #	Date of Repair
7	1993
11	1993
18	2002
19	Active
33	2003
35	1980
37	1981
38	1986

MARCARDON

House #	Date of Repair
39	2009
41	1998
42	1976

CREAMERY

House #	Date of Repair
40	2001

A couple of houses are on sewer force mains that have broken on several occasions.

TOWN OF RIDGEFIELD

Health Department

To: WPCA

From: Edward Briggs, Director of Health Edward L Briggs

Date: January 29, 2015

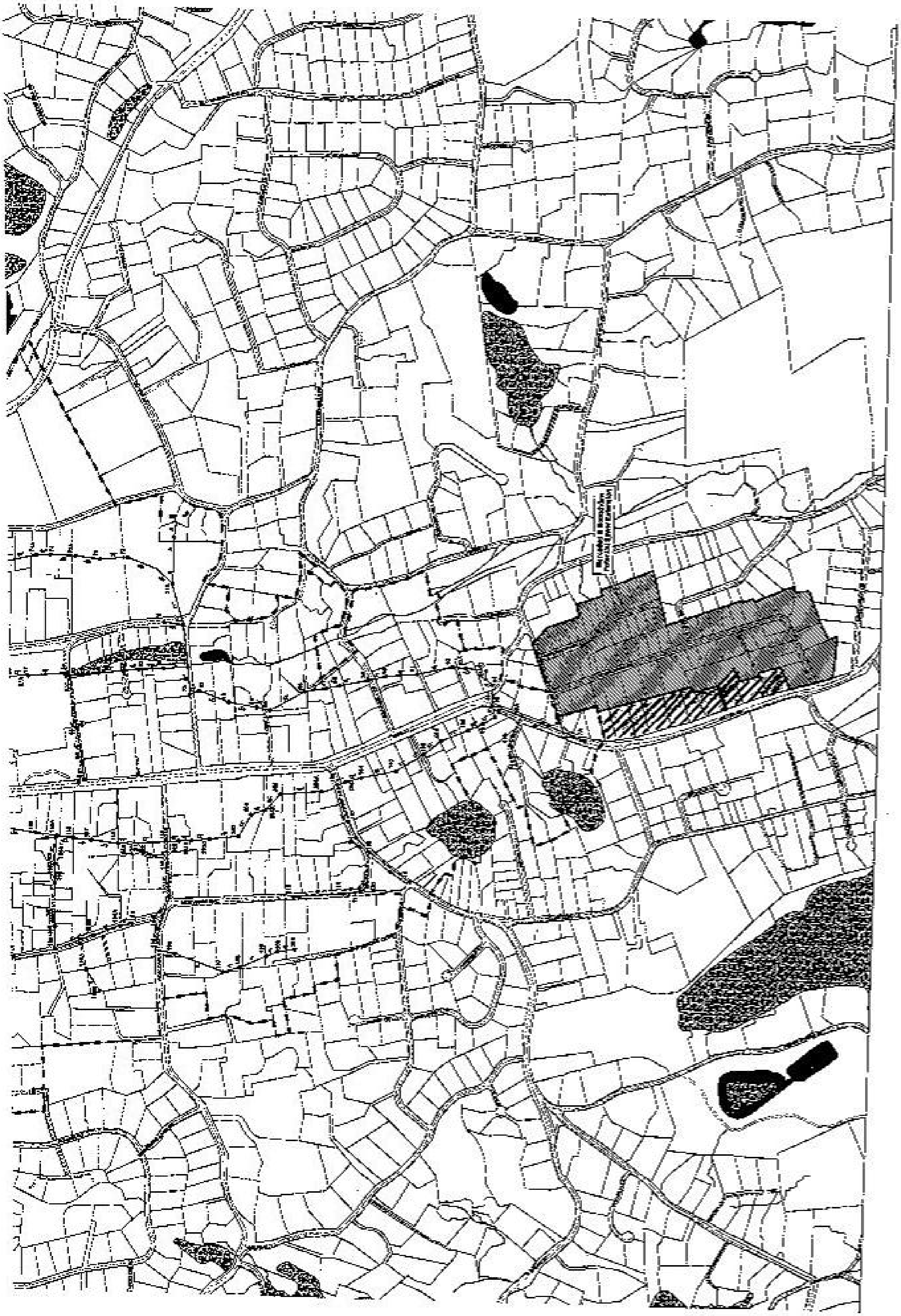
Subject: Priority areas for sewer expansion

I have reviewed the Health Department files for Wilton Road West and Olmstead Lane.. There are a few outliers that are close to an existing manhole that could be connected on Wilton Road West. However, I would suggest that the sewers be looped around Soundview and cover all the properties from 284 Wilton Road West back to the manhole in front of 378 Wilton Road West. As far as Olmstead Lane, a majority of the houses have engineered septic systems that were already installed and many have done building additions and have demonstrated suitable septic repair areas. I do not see anything in the files to justify the designation of this road as a priority area for sewers.

Since the late 1970's when I was on the Sewer Advisory Committee I have been stating that the priority areas should be New Street, Soundview, Marcadon and the outliers listed above. I have not changed my mind since then.

66 Prospect Street • Ridgefield, Connecticut 06877 • (203) 431-2745

www.ridgefieldct.org



APPENDIX C
WPCA PUBLIC MEETING
SEWER NEEDS FOR DISTRICT 1
DECEMBER 4, 2014 MINUTES

Unapproved
Town of Ridgefield
Water Pollution Control Authority

WPCA Public Meeting
Sewer Needs for District 1
December 4, 2014 7:00 p.m.
Town Hall Annex, 66 Prospect Street
Ridgefield, Connecticut 06877

WPCA Present:	Amy Siebert, Rudy Marconi, Ron Hill, Gary Zawacki, Kevin Briody
United Water:	Jason O'Brien, Jorge Periera
AECOM:	Jon Pearson, Don Chelton

These are not verbatim minutes of the proceeding but identification of general items and specific actions undertaken.

Public Meeting
A Special Meeting of the Water Pollution Control Authority was held on Thursday, December 4, 2014 in the BOE conference room at Town Hall Annex. The meeting was called to order at 7:00 p.m. by Ms. Siebert.

Introductions: Ms. Siebert introduced the WPCA and AECOM, Don Chelton and Jon Pearson

A copy of the presentation is attached to these minutes regarding Items 1 - 8:

1.Purpose of Meeting
2.Overview of Sewer District 1 existing wastewater facilities
3.Why is a facilities plan needed?
4.What is the facilities planning process
5.Work completed to date
6.South Street WWTF current flows and capacity
7.Identified sewer needs areas
8.Next steps in the planning process

9.Public comments and input

a. **Becky Mucchetti, Chairman, P&Z. Regarding the Ramapoo Road extension: Is connection mandatory for property owners?**
   i. No, they have an option to not connect.

   What percentage of property owners in District 1 have not connected to sewer?
   ii. Ms. Van Ness will investigate and report to Ms. Mucchetti.

b. **John Katz, P&Z member. Confused about the potential for expansion**
   i. Ms. Siebert. The WPCA is trying to comply with the 20 year plan for future sewer needs. The sewer plant is 20 years old and needs to be upgraded as it runs 24/7.
   ii. Mr. Pearson. Another option to provide treatment capacity is to reduce leakage into the sewer system from groundwater and stormwater.
   iii. Mr. Briody. The State has implemented very strict limits on phosphorous that must to be addressed in the upgrade.

c. **<u>Steve Cole, P&Z member</u>. Possibility of bringing flow from Route 7, District 2, to South Street, District 1 was discussed in presentation. Is there any possibility of bringing flow from South Street WWTF to Route 7 WWTF to free up capacity at South Street WWTF.**
    i. Ms.Siebert. Route 7 does not have additional capacity. It is fully allocated.
d. **<u>Peter Chipouras, P&Z member</u>. Where is sewer plant in regards to capacity?**
    i. Mr. Pearson. Depends on time of year and weather. Currently flows at 80% of plant capacity on average, but with rain events and inclement weather it has been over 120%.
e. **<u>Philp Mische, P&Z member.</u> Where in town are sewers below water table and are a problem?**
    i. Ms. Siebert. Sewers are frequently located below the water table, and the water table affects the majority of the sewer system.
f. **<u>Amy Siebert, WPCA Chairperson</u>**. We have to address the phosphorus limits set by the DEEP.
    i. Mr. Pearson. The DEEP will include a time limit on when we have to meet the phosphorus limits in the new permit that contains the limits.
g. **<u>Timothy Dunphy, ADDRESS?</u>. Regarding Flow Calculations: Is design to be based on full build-out?**
    i. Mr. Pearson. Based on input from Planning and Engineering Departments, projected flows are based on 70% of build out for next 20 years.

    How will 8-30g affect capacity allocated for others?
    ii. Mr. Chelton. This becomes a legal issue.
h. **<u>Diana Gaughran, 360 Wilton Road West</u>.** Property backs up to Soundview and is close to sewer on Wilton Road West. Would like to be considered for expanded district as her property has a very fragile septic system. High water table, poor soil and must pump her septic tank several times a year. When it rains there is a strong septic odor.
i. **<u>Tom O'Keefe, 12 Olmstead Lane</u>.** Property very close to sewer boundary. Has lots of ledge under surface. Surface water has effect on septic system.
j. **<u>Suzanne Gabriel, 351 Wilton Road West.</u>** Has the same issues as 360 Wilton Road West and 12 Olmstead.
k. **<u>Carrie Ertl, 48 Soundview Road.</u>** How does process work to expand district?
    i. Ms. Siebert. If a sewer project is to be implemented, process involves preliminary estimate of project cost, and estimate of assessment of cost for each parcel serviced would be calculated. If property owners to be serviced agree to fund the project, it would move forward and property owners would be assessed cost for the project once built.
l. **<u>Max Caldwell, 18 Downsbury Court</u>.** In 1990 two areas were identified for sewer expansion. Soundview, Marcadon and Ramapoo. Ramapoo area was selected because we had a grant.
m. **<u>Mack Reid, Ridgefield Press.</u>** How would an assessment be calculated?
    i. Ms. Siebert. The total cost of the project would be considered and each property assessed its share of the cost. There are different assessment approaches that could be used.

n. **<u>Gene Waradzin, 27 Marcadon Avenue</u>** Resident for 48 years. Had a problem with his septic for 9 years. Had it repaired and has his septic pumped once a year for preventive maintenance. Concerned that if areas that need sewer aren't hooked up now, there will be a problem in the future.

o. **<u>Robert Salvestrini, 74-78 New Street</u>** 78 New Street is pumped at least once a year. Has experienced significant increase in surface runoff in the past 15 years.

p. **<u>Diane Doyle, 1 Marcadon Avenue</u>**. What level of problem constitutes a health concern?

   i. Mr. Pearson. There a number of factors to consider but if there is breakout or ponding sewage from the system, it becomes a health issue.

q. **<u>Ed Briggs, Director of Health</u>** Has suggested the Soundview/Marcadon area be sewered 20 years ago. There are several failing septic systems in the Marcadon, Soundview area because of the high ground water, ledge, hard-pan and the repairs are very expensive because of the existing conditions.

r. **<u>Diana Gaughran, 360 Wilton Road West</u>** When would a plan actually be executed?

   i. Ms. Siebert. No set time – would depending on the desire of residents in an area to request sewer extension and agreeing to pay for cost of extension.

s. **<u>Don Sturges, 20 Olmstead Lane</u>** (His son Chris Sturges recently purchased this property) Property has a very high water table. Sewer service to the neighborhood would greatly improve soil quality and sanitation standards while minimizing potential public health concerns.

t. **<u>Tom O'Keefe, 12 Olmstead Lane</u>** Olmstead Lane has a higher elevation and would help drive flow.

u. **<u>Pete Gartland, 72 New Street</u>** Would like to be considered for sewer service.

v. **<u>Edward Shenkel, Gregory and Adams representing Eureka V LLC.</u>** Submitted letter requesting hearing to be left open for comment.

   i. <u>Ms. Siebert. This meeting is not a hearing and it pertains only to District 1. We will be happy to take your comments.</u>

w. **<u>Kitsy Snow, Conservation Commission</u>** Requested UV lights be left on all year.

   i. Ms. Siebert. Expensive and not required by permits.
   ii. Mr. Pearson. Not required to be left on year round.

x. **<u>John Katz, P&Z</u>** Conservation and Development POCD/CT reference. Takes conservative approach to expanded sewers.

   i. Mr. Pearson. New plan takes a more balanced approach

y. **<u>Phillip Mische, P&Z</u>** Re. Gregory and Adams request: This is a public meeting, not a public hearing. What factors are involved in a new sewer system?

   i. Mr. Chelton. Size, condition, flow and capacity of pipes in collection system. Cost and assessment of sewer. Value of property goes up with sewer system.

z. **<u>Merrill Brown, Acorn Place</u>**, has house on Creamery Lane. When you identify an area to be expanded, how long before it is connected?

   i. Mr. Chelton. Once an area is determined, design, public process, bids, etc., up to 3 years before it is connected.

aa. **Diana Gaughran, 360 Wilton Road West** How would the property owner pay for the sewer assessment?
   i. Don Chelton. Town takes out a 20 year bond and pay over time.

bb. **Gene Waradzin, 27 Marcadon Avenue**. Read summary of Information Given at June 4, 1993 Sewer Meeting. Stating that if the residents signed a petition that their septics were failing, the Town would extend the district. The property owners would then be assessed for the cost of the project.

cc. **Robert Salvestrini, 74, 78 New Street.** What's to prevent 8-30g developers demanding the sewer capacity?
   i. Mr. Marconi. The Town has a four year moratorium on 8-30g.

dd. **Rudy Marconi, WPCA** The DEEP Clean Water Fund has funds available on a first come, first served basis which consists of a 20% grant and 80% loan.

ee. **Becky Mucchetti, Chairman P&Z** Is it possible to allocate certain amount of capacity to each individual property?
   i. **Max Caldwell** Can only be allocated if it's a Benefit Assessment District. (like Route 7)
   ii. **Jon Pearson** Route 7 is different than South Street- Route 7 capacity was allocated and purchased by the property owners to fund the construction of the plant.

ff. **John Katz, P&Z** WPCA needs to put on its Health and Safety hat. How do we bring sewers to areas without more development.
   i. Don Chelton, AECOM This is a legal issue.
   ii. Rudy Marconi, WPCA 8-30 g developers cannot force the WPCA to expand sewer district.

gg. **Amale Hawi, New Street** Suggest dealing with 8-30g and sewer needs separately. Give the property owners with septic problems sewer and come up with a plan to control 8-30g.
   i. Rudy Marconi There are municipalities in Hartford working to change 8-30g. However, to date they have not been successful in controlling 8-30g.

hh. **Kevin Briody, WPCA** The WPCA is not designed to be a land use instrument. We should just be taking care of sewer needs.

4) Motion to adjourn public meeting at 8:25 p.m. by Mr. Zawacki, seconded by Mr. Hill, passing unanimously.

Submitted by Diana Van Ness

APPENDIX D

TECHNICAL MEMORANDUM NO. 6 – WWTF CAPACITY EVALUATIONS

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Technical Memorandum No. 6

To	Ridgefield WPCA	Page	1 of 61
CC	C. Fisher, J. O'Brien, J. Pereira, J. Pennell		
Subject	Town of Ridgefield, CT Phase 1 Wastewater Facilities Plan Draft Technical Memorandum No. 6 – WWTF Capacity Evaluations		
From	Jon Pearson and Matt Formica		
Date	February 26, 2014		

INTRODUCTION

The Town of Ridgefield owns two wastewater treatment facilities (WWTFs), the South Street WWTF which serves Sewer District No. 1 and the Route 7 WWTF which serves Sewer District No. 2. The Town owned WWTFs and the Sewer District 1 and Sewer District 2 collection systems are operated by United Water through an operations contract with the Town.

The South Street WWTF is the larger of the two WWTFs with a design average flow of 1.0 million gallons per day (mgd). The South Street WWTF provides advanced treatment using the activated sludge process to treat wastewater collected from Sewer District 1 which includes downtown Ridgefield.

Sewer District 2 is located in the northeast portion of the town in the area where Route 7 and Route 35 intersect. Wastewater collected in this area is treated by the Route 7 WWTF which has a design average flow of 0.12 mgd.

With the last major upgrade at the South Street WWTF in 1992, and the construction of the Route 7 WWTF in 1985, a significant portion of the Ridgefield wastewater treatment facility components have exceeded their anticipated 20 year service life.

Discharges from both plants are regulated by the Connecticut Department of Energy and Environmental Protection (DEEP) through permits issued under the National Pollutant Discharge Elimination System (NPDES) program. The existing permits for both plants expired in 2009 and the Town submitted renewal applications as required by the program rules. The DEEP deferred issuing new permits for both WWTFs until DEEP's Phosphorus Reduction Strategy for Inland Non-Tidal Waters could be developed and finalized. In the meantime, the expired NPDES permits were administratively continued and remained in effect.

DEEP's Phosphorus Reduction Strategy for Inland Non-Tidal Waters was developed in response to an EPA requirement to implement limitations on phosphorus in all wastewater NPDES permits where the potential exists for the discharge to contribute to eutrophication and impair designated uses in downstream non-tidal waters. As a result of the implementation of the strategy, in October 2014 DEEP issued a new NPDES permit for the Route 7 WWTF that includes an effluent phosphorus limit that the existing treatment facility cannot meet without modifications. The permit also includes a compliance schedule that defers the implementation of the new limit until August 2019 to allow time for the Town to complete the ongoing facilities planning effort and implement modifications to the Route 7 WWTF to meet the new phosphorus limit. It is anticipated that the South Street WWTF permit, once issued, will also contain a more stringent limit on effluent phosphorus as well as a compliance schedule to meet the new limit.

The purpose of this memorandum is to evaluate the capacity of each WWTF under current conditions, design conditions, and increased flow and loading conditions to determine which unit processes are limiting the WWTFs overall capacity. In order to assess the capacity of each WWTF, both the hydraulic capacity and the pollutant removal capacity were evaluated. At each plant, each unit process was evaluated and an opinion is offered on both its hydraulic capacity and pollutant removal capacity to identify which unit processes limit the overall WWTF capacity. After these capacity limitations were established potential modifications to relieve these limitations were then identified based on the current permit limits at both WWTFs and the potential future permit limits at the South Street WWTF. An opinion of the potential to “re-rate” the WWTFs to a higher capacity has been provided as part of the analyses. These analyses are presented below starting with the Route 7 WWTF followed by the South Street WWTF.

ROUTE 7 WWTF PLANT CAPACITY ANALYSIS

As noted above, Sewer District 2 is served by the Route 7 WWTF. The Route 7 WWTF was constructed in 1985 to serve the needs of Sewer District 2 that included flows from the Perkin Elmer facility. The Route 7 WWTF provides advanced wastewater treatment using rotating biological contactors, has an average daily design flow of 0.120 mgd, and discharges treated wastewater to the Norwalk River. Figure 1 provides a layout of the Route 7 WWTF. Figure 2 presents a process flow schematic of the existing Route 7 WWTF.

Existing and Design Flows and Loads

Influent Flow, Concentration and Load Data. The existing influent flows and concentrations of wastewater constituents for the WWTF for the period between July 1, 2010 and June 30, 2013 were evaluated. Based on this review, the current annual average daily flow is approximately 0.053 mgd with maximum month flow of 0.079 mgd, a maximum dally flow of 0.162 mgd and an instantaneous peak flow of 0.357 mgd. Figure 3 presents the WWTF influent flow data over the three year evaluation period.

In addition the WWTF influent constituent data for the Route 7 WWTF during the same period was evaluated. Figure 4 presents the influent total suspended solids (TSS) concentration data, Figure 5 presents the influent biochemical oxygen demand (BOD) concentration data, Figure 6 presents the influent total phosphorus (TP) data and Figure 7 presents the primary effluent ammonia (NH_3) data (Note: influent NH3 data is not reported on Route 7 WWTF Monthly Operating Reports (MORs)).

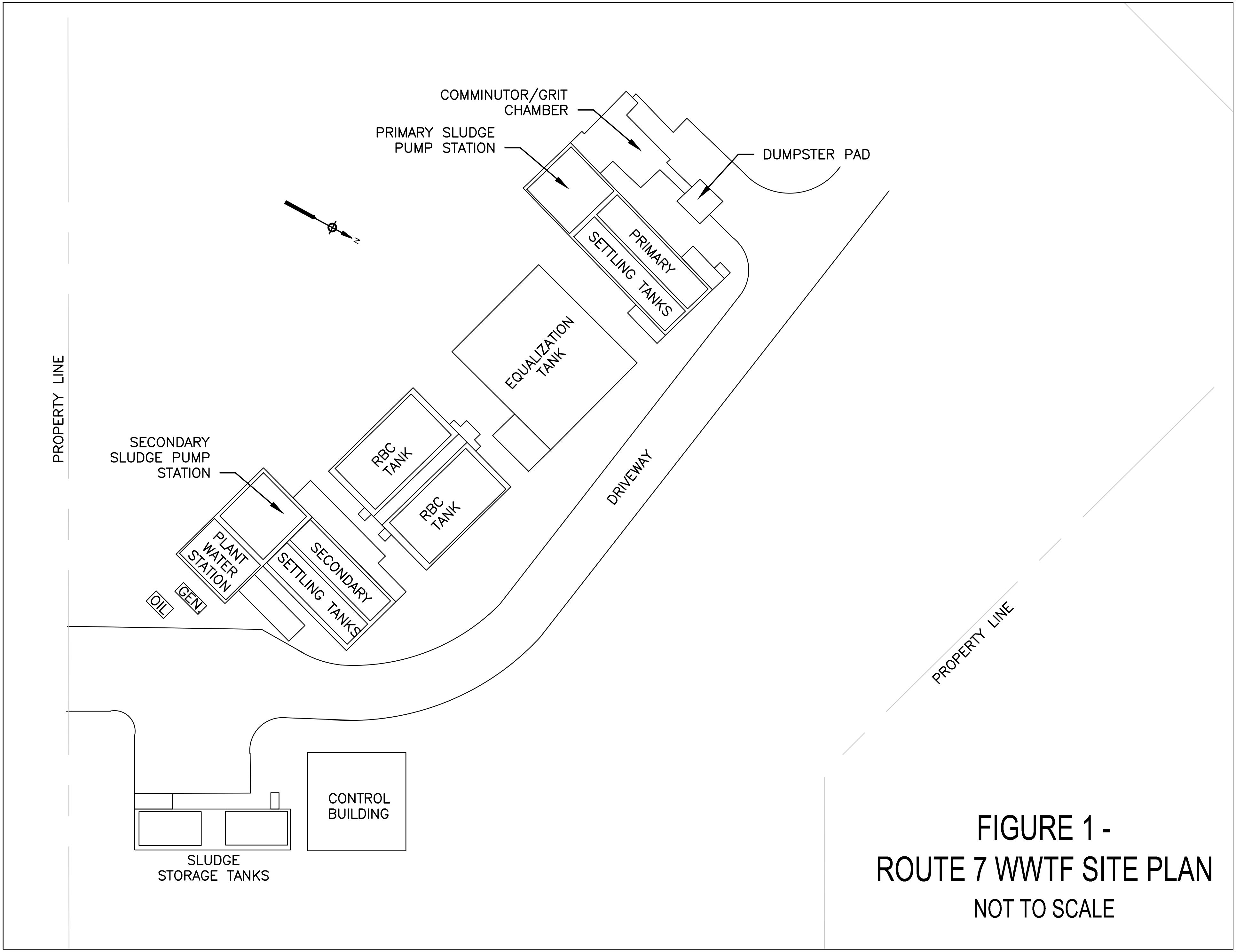

FIGURE 1 - ROUTE 7 WWTF SITE PLAN

NOT TO SCALE

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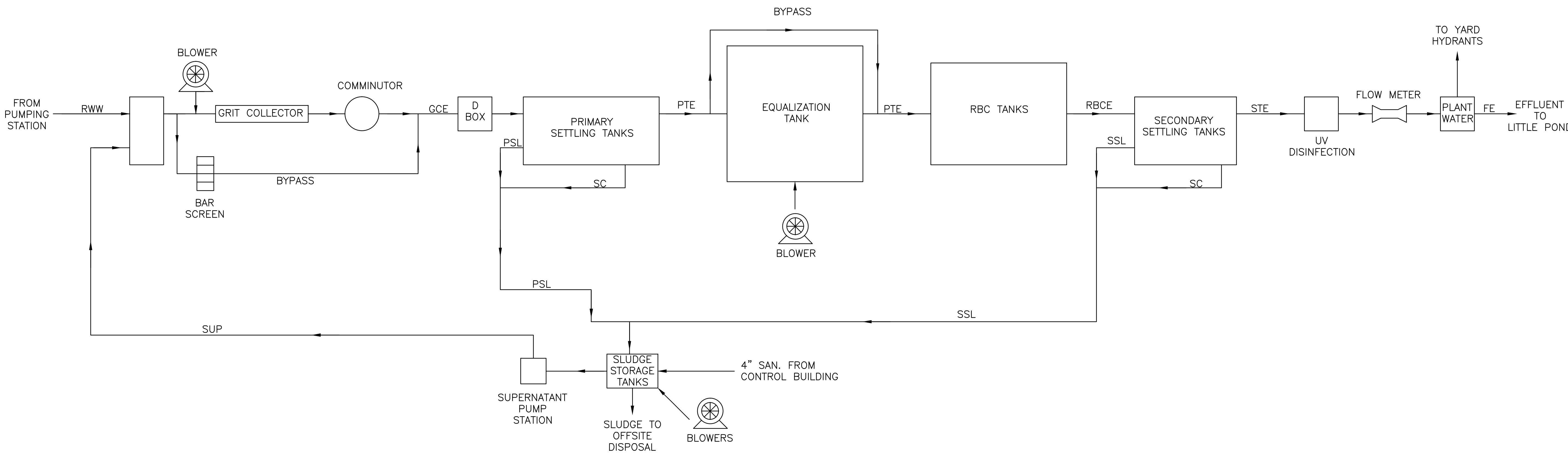
LAST UPDATE: Monday, January 19, 2015 2:53:49 PM PLOT DATE: Monday, February 16, 2015 9:36:32 AM

PATH/FILENAME: X:\RIDGEFIELD, CT\60299267 – RIDGEFIELD PHASE 1 FAC PLAN\500 DELIVERABLES\505 PLANT CAPACITIES MEMO\FIGURES\SHEETS\FIGURE 2.DWG
LAST UPDATE: Monday, January 19, 2015 2:36:11 PM PLOT DATE: Monday, January 19, 2015 2:54:59 PM



LEGEND	
RWW	RAW WASTEWATER
GCE	GRIT CHAMBER EFFLUENT
FE	FINAL EFFLUENT
PTE	PRIMARY SETTLING TANK EFFLUENT
STE	SECONDARY SETTLING TANK EFFLUENT
SC	SCUM
PSL	PRIMARY SLUDGE
SSL	SECONDARY SLUDGE
RBCE	ROTATING BIOLOGICAL CONTACTOR EFFLUENT
SUP	SUPERNATANT
SAN	SANITARY DRAIN

FIGURE 2 -
ROUTE 7 WWTF
PROCESS FLOW SCHEMATIC

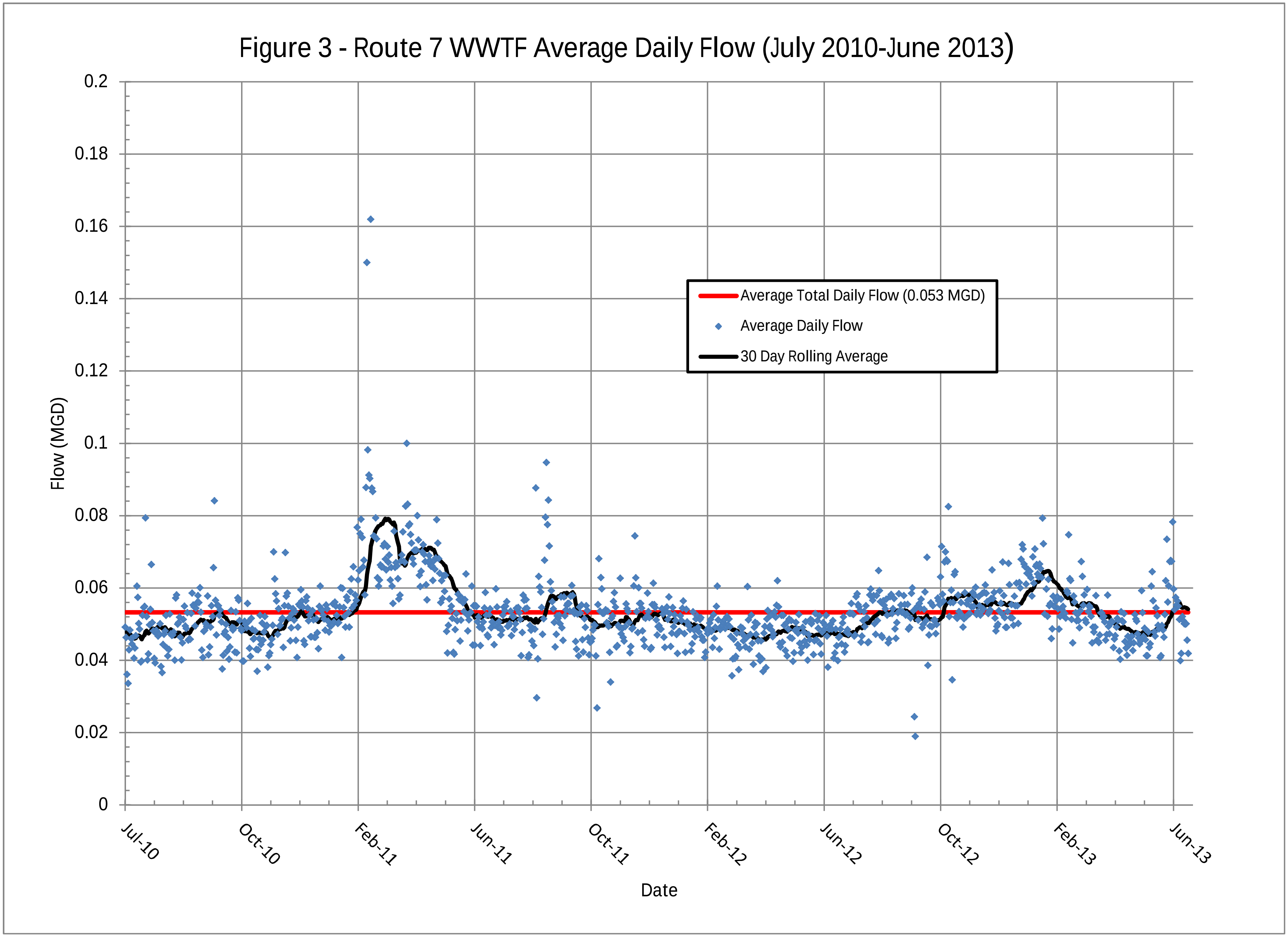
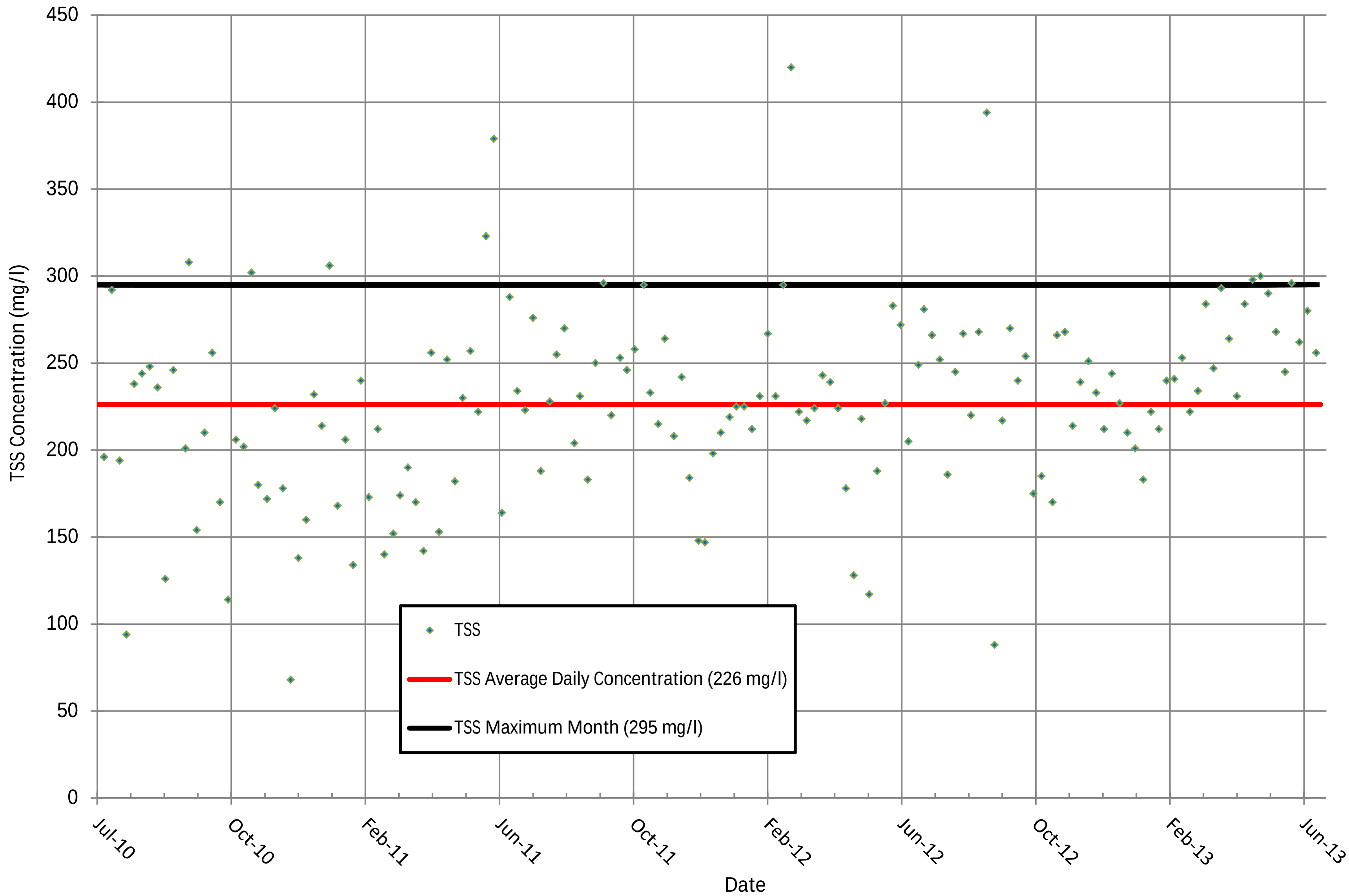


Figure 3 - Route 7 WWTF Average Daily Flow (July 2010-June 2013)
Average Total Daily Flow (0.053 MGD)
Average Daily Flow
30 Day Rolling Average
Flow (MGD)
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date

Figure 4 - Route 7 WWTF TSS Influent Concentration
(From July 2010 - June 2013)



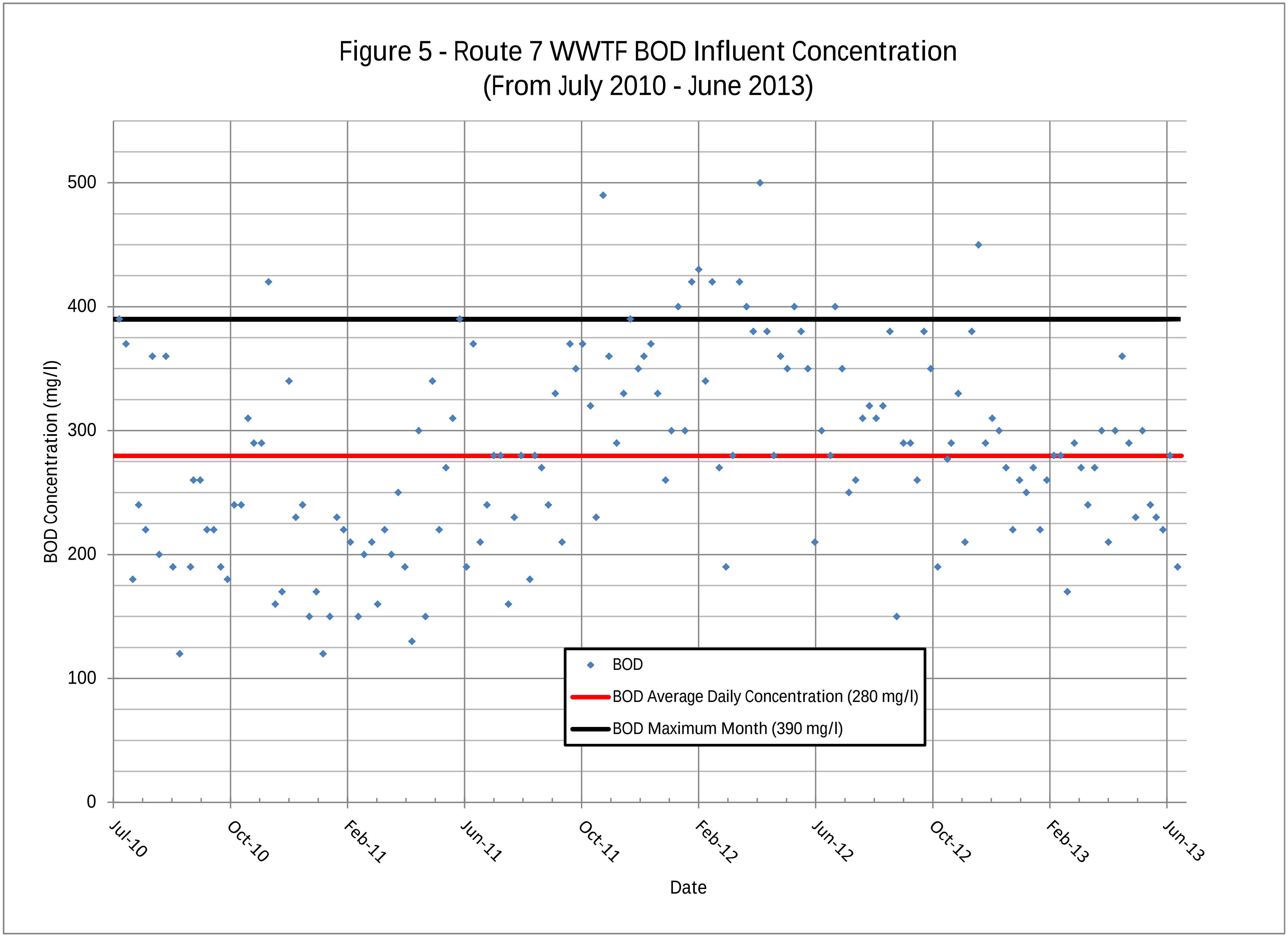
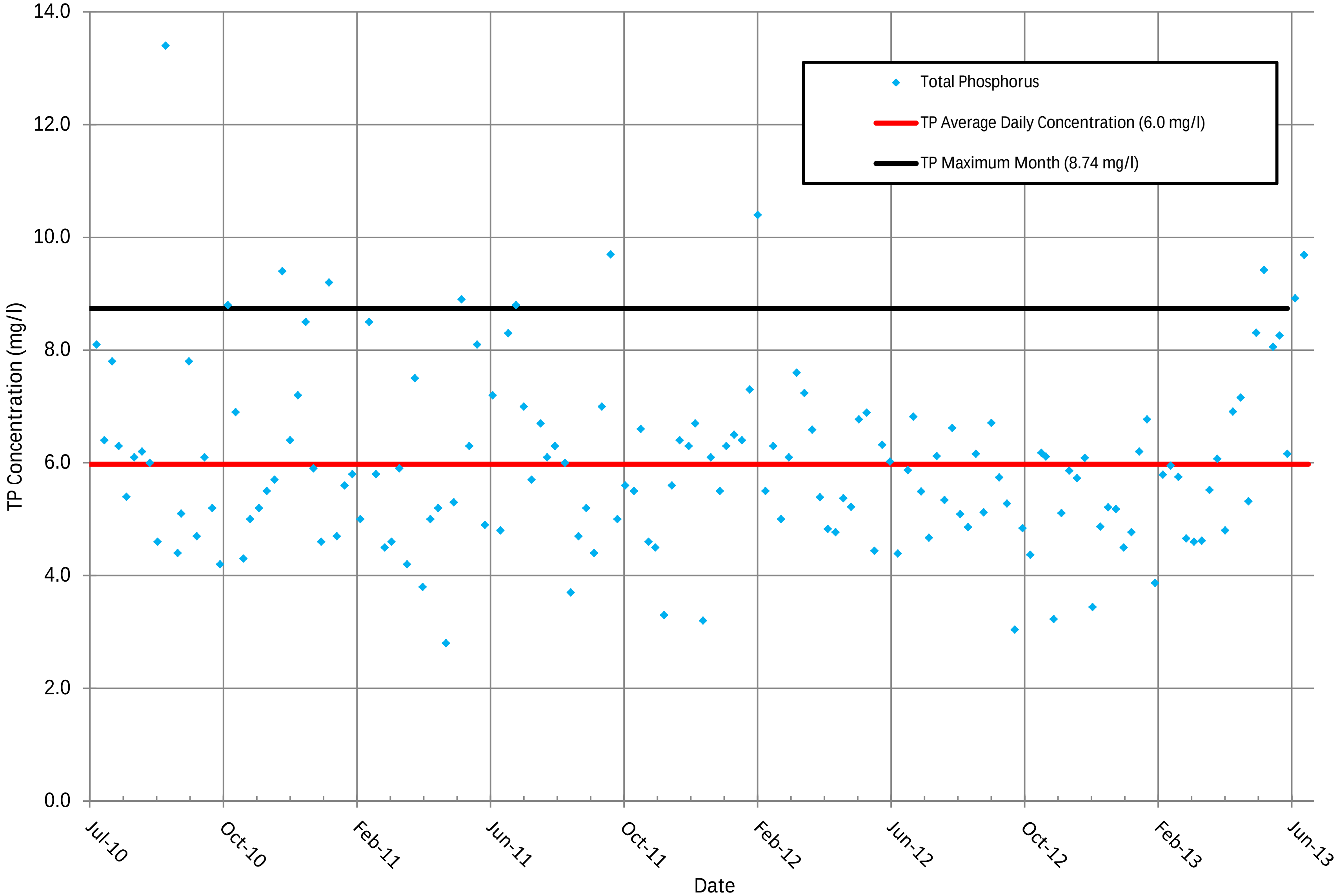


Figure 5 - Route 7 WWTF BOD Influent Concentration
(From July 2010 - June 2013)
BOD Concentration (mg/l)
0
100
200
300
400
500
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date
BOD
BOD Average Daily Concentration (280 mg/l)
BOD Maximum Month (390 mg/l)

Figure 6 - Route 7 WWTF Total Phosphorus Influent Concentration
(From June 2010 - June 2013)



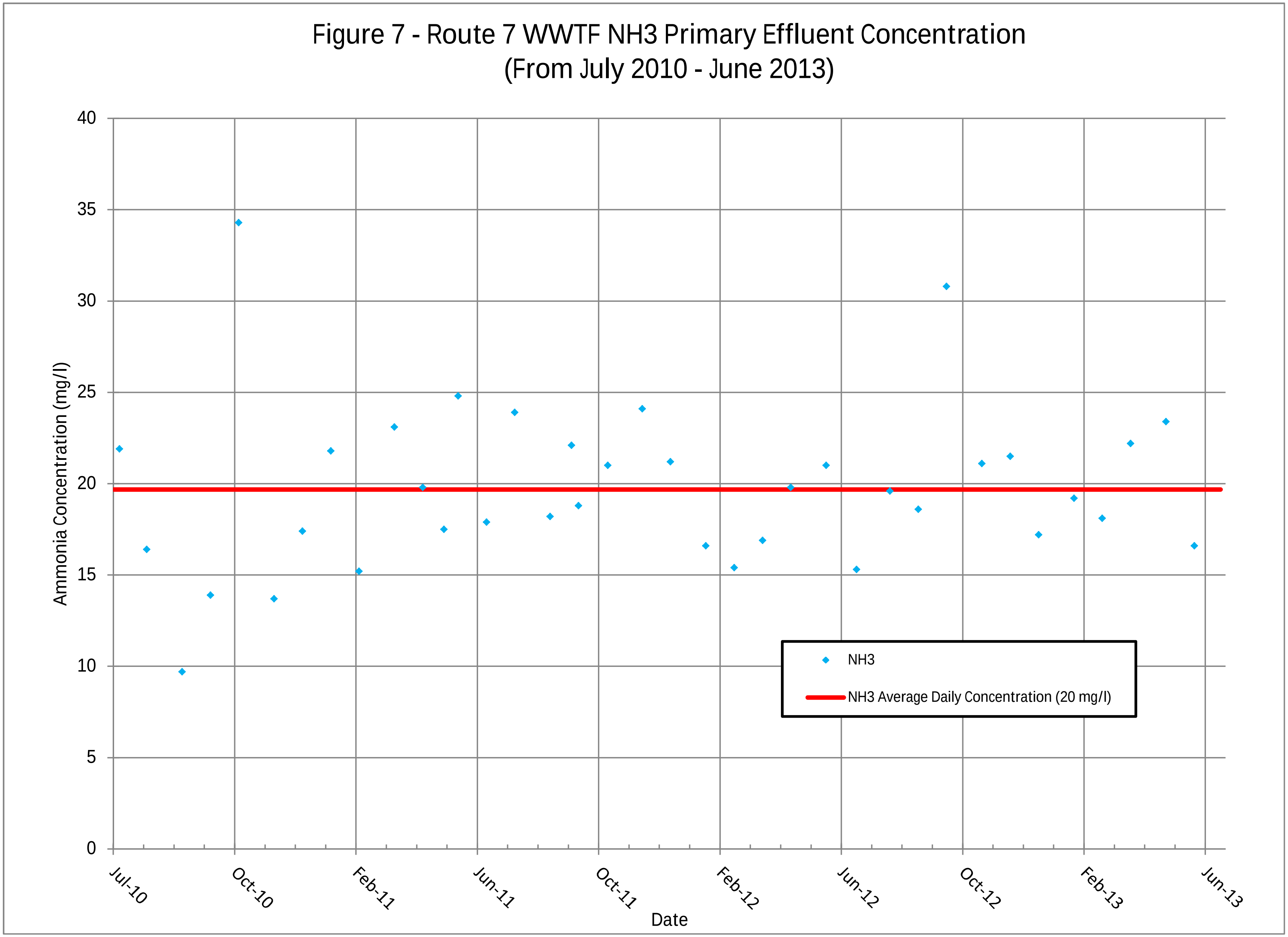


Figure 7 - Route 7 WWTF NH3 Primary Effluent Concentration
(From July 2010 - June 2013)
Ammonia Concentration (mg/l)
40
35
30
25
20
15
10
5
0
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date
NH3
NH3 Average Daily Concentration (20 mg/l)

Table 1 summarizes the Route 7 WWTF flow and wastewater constituent data for the period of July 1, 2010 to June 30, 2013 for the WWTF's influent, primary effluent and final effluent including maximum month conditions. Note due to the limited number of daily samples collected for analysis, the maximum month loading conditions were based on the 92nd percentile of all of the data while the maximum month concentration data was back calculated from the maximum month loading conditions and the maximum month flow.

TABLE 1 - ROUTE 7 WWTF FLOW AND LOADING SUMMARY (JULY 2010 TO JUNE 2013)

Parameter	Annual Average Day	Max Month Peaking Factor	Max Month
Influent			
Flow (mgd)	0.053	1.49	0.079
TSS (mg/l)	226		199
TSS (lb/d)	102	1.28	131
BOD_5 (mg/l)	280		263
BOD_5 (lb/d)	124	1.40	173
Total Phosphorus (mg/l)	5.98		5.84
Total Phosphorus (lb/d)	2.71	1.42	3.85
Ortho-Phosphate (mg/l)	3.28		2.94
Ortho-Phosphate (lb/d)	1.46	1.33	1.94
Primary Effluent			
TSS (mg/l)	109		139
TSS (lb/d)	49.3	1.86	91.5
BOD_5 (mg/l)	180		182
BOD_5 (lb/d)	81.8	1.47	120
Ammonia Nitrogen (mg/l)	19.7		17.8
Ammonia Nitrogen (lb/d)	8.91	1.31	11.7
Effluent Discharged			
TSS (mg/l)	2.62		4.46
TSS (lb/d)	1.17	2.51	2.94
BOD_5 (mg/l)	4.20		5.42
BOD_5 (lb/d)	1.89	1.89	3.57
Ammonia Nitrogen (mg/l)	0.52		0.90
Ammonia Nitrogen (lb/d)	0.24	2.46	0.59
Total Phosphorus (mg/l)	5.09		5.00
Total Phosphorus (lb/d)	2.29	1.44	3.29
Ortho-Phosphate (mg/l)	4.05		3.79
Ortho-Phosphate (lb/d)	1.82	1.37	2.50

Design Flow and Loading Comparison

Flows. The current flows to the WWTF are significantly lower than the design flows to the plant of 0.12 mgd and a peak hourly flow of 0.72 mgd for the processes upstream of the equalization tank and a peak flow of 0.30 mgd downstream of the equalization tank. It should be noted that the WWTF does not currently operate the equalization tank in an equalization mode but allows the flow to pass through the tank and exit the tank through an overflow pipe at the top of the tank.

Loads. Due to the limited records on the WWTF's design conditions, a direct comparison of the current influent loadings to the design influent loading could not be made. However the contract specifications from the 1984 upgrade include design criteria for the rotating biological contactors. Per the contract documents the RBCs were specified to treat 0.12 mgd of primary effluent with BOD and TSS concentrations of 275 mg/l. Comparing that design criteria to the current primary effluent concentrations as presented in Table 1, the RBCs are currently under loaded for both TSS and BOD. Also assuming the primary settling tanks were intended to remove approximately 35% of the influent BOD and 50% of the influent TSS, this would have meant the original influent wastewater constituent concentrations for BOD and TSS would have been approximately 425 mg/l and 550 mg/l. Again, comparing these back-calculated design concentrations with the current influent wastewater concentrations in Table 1 reinforces that the WWTF is currently underloaded.

ROUTE 7 WWTF HYDRAULIC CAPACITY

The hydraulic capacity of the Route 7 WWTF was evaluated and an opinion of the current hydraulic capacity of each unit process in the facility has been provided. The approach to evaluate the Route 7 WWTF and the results are summarized below.

Approach

Based on the existing WWTF drawings, a computer based hydraulic model was constructed to represent the physical conditions at the WWTF. The WWTF's current flows from July 2010 to June 2013 and the design year flows from the 1985 Route 7 WWTF construction were modeled.

The results from the current and design year model runs were compared to the hydraulic profiles included in the Route 7 WWTF 1985 contract drawings. The hydraulic model was adjusted as needed so the model output reflected the hydraulic profile within the contract drawings for the WWTF.

Once the model was calibrated, a number of hydraulic model runs with increasing flow rates were conducted to assess the hydraulic capacity of the existing WWTF. Specific unit process or WWTF components that limit the overall capacity at the WWTF were identified including quantification of their estimated hydraulic capacity. The hydraulic capacity was evaluated with one of the redundant units out of service for each of the applicable processes (for example one primary settling tank, one RBC, and one final settling tank) based on the requirements of "TR-16 Guides for the Design of Wastewater Treatment Works" published by the New England Interstate Water Pollution Control Commission in 2011.

For the purposes of this evaluation, unit processes or structures are considered to have limited hydraulic capacity if either of the following conditions occurs:

- There is less than 1 foot of difference (freeboard) between the top of the structure (top of concrete) and the hydraulic grade line (water elevation). It is desirable to have the water surface in a tank or structure about 1 foot below the tank wall to allow for foaming and splashing without overtopping the walls.
- There is less than 3 inches between a flow control weir in a structure and the water surface elevation downstream of the weir. It is desirable to have about 3 inches between a flow control weir and the downstream water surface so the flow control weir performance is not impacted by the downstream water surface.

Finally in order to increase the hydraulic capacity of the WWTF, potential modifications to the WWTF unit processes/components limiting capacity were identified. These modifications were identified for possible subsequent evaluation in the Phase 2 Facilities Plan. Some or all of these modifications will be evaluated and an assessment of the estimated costs of these modifications will also be performed as applicable as part of the Phase 2 Facilities Plan effort.

Route 7 WWTF Hydraulic Capacity Model Runs

The hydraulic model for the Route 7 WWTF was run at the following flow rates to evaluate the hydraulic capacity of the WWTF:

- 0.053 MGD - Current average daily flow to the WWTF (July 2010 to June 2013).
- 0.12 MGD - Design average daily flow to the WWTF.
- 0.20 MGD
- 0.30 MGD- Maximum flow for unit processes downstream of the equalization tanks.
- 0.60 MGD
- 0.72 MGD - WWTF peak flow (equal to the capacity of the Route 7 Pump Station Pumps).

As noted above, the maximum flow modeled was 0.72 mgd, which represents the maximum output of the Route 7 Pump Station which conveys flow to the WWTF. It should also be noted that all processes downstream of the equalization tank, the maximum flow modeled was 0.30 mgd as a result of the design of the equalization (EQ) tank to equalize the WWTF flows to 0.30 mgd or less.

The model runs for the Route 7 WWTF are summarized in Table 2, Table 3, and Table 4 below. These tables include the top of concrete or top of wall (TOC) elevation for the different process tanks/structures, weir elevations in the different structures, the model run hydraulic grade lines (HGL), and resulting differences in HGL elevation to the TOC and weirs in each process.

Based on the model runs performed, the hydraulic capacity of each of the unit process was evaluated. The summary of the hydraulic capacity of each of the unit process is summarized in Table 5. Please note that the hydraulic capacity of the rotating biological contactors and the UV disinfection system are indicated in the table as 0.0 mgd. This is the result of the weirs in both of these unit processes being located less than the recommended one foot below the top of the wall or structure. It should be noted that these structures have not been reported to have overtopped in the past and have been able to convey the flows recorded at the WWTF.

AECOM

TABLE 2 – ROUTE 7 WWTF HYDRAULIC PROFILE
AT EXISTING CONDITIONS (0.053 MGD) AND DESIGN FLOW (0.12 MGD) WITH LARGEST UNIT NOT IN SERVICE

	Field Conditions		Hydraulic Conditions at 0.053 MGD			Hydraulic Conditions at 0.120 MGD		
Treatment Unit	TOC Elevation [1]	Weir Elevation	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.
Plant Influent Chamber	530.0		528.10	1.90		528.16	1.84	
Grit Chamber	530.0		527.03	2.97		527.61	2.37	
Comminutor	529.0		527.00	2.00		527.44	1.56	
Primary Settling Tanks	529.0	526.83	526.90	2.10		526.92	2.08	
Primary Settling Tank Effluent Trough	529.0		526.35	2.65	0.48	526.45	2.55	0.38
Equalization Tank	527.0	525.58 [3]	525.66	1.34		525.72	1.28	
Rotating Biological Contactors	517.0	516.5	516.54	0.46		516.6	0.40	
Rotating Biological Contactors Effluent	517.0		513.5	3.50	3.00	513.51	3.49	2.99
Secondary Settling Tanks	511.0	508.63	508.69	2.31		508.72	2.28	
Secondary Settling Tank Effluent Trough	511.0		508.31	2.69	0.31	508.41	2.59	0.21
UV Disinfection Channel	499.31	498.31	498.84	0.47		498.88	0.43	
UV Disinfection Effluent Trough	498.96		497.82	1.14	0.49	497.93	1.03	0.38
Plant Water Station	511.0	497.48	497.73	13.27		497.83	13.17	
Plant Water Station Effluent	511.0		459.52	15.48	1.96	495.53	15.47	1.95

1. TOC - Top of concrete.
2. Highlighted items indicate that wall freeboard or weir submergence is out of desired range 1.0 ft and 0.25 ft, respectively.
3. Overflow pipe at top of tank modeled as a weir.

TABLE 3 – ROUTE 7 WWTF HYDRAULIC PROFILE
AT 0.2 MGD AND 0.30 MGD WITH LARGEST UNIT NOT IN SERVICE

	Field Conditions		Hydraulic Conditions at 0.20 MGD			Hydraulic Conditions at 0.30 MGD		
Treatment Unit	TOC Elevation [1]	Weir Elevation	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.
Plant Influent Chamber	530.0		528.2	1.77		528.3	1.70	
Grit Chamber	530.0		528.12	1.86		258.19	1.81	
Comminutor	529.0		527.49	1.51		527.57	1.43	
Primary Settling Tanks	529.0	526.83	526.94	2.06		526.96	2.04	
Primary Settling Tank Effluent Trough	529.0		526.53	2.47	0.30	526.62	2.38	0.21
Equalization Tank	527.0	525.58 [3]	525.78	1.22		525.84	1.16	
Rotating Biological Contactors	517.0	516.5	516.62	0.38		516.65	0.35	
Rotating Biological Contactors Effluent	517.0		513.51	3.49	2.99	513.51	3.49	2.99
Secondary Settling Tanks	511.0	508.63	508.74	2.26		508.76	2.24	
Secondary Settling Tank Effluent Trough	511.0		508.49	2.51	0.13	508.58	2.42	0.05
UV Disinfection Channel	499.31	498.31	498.92	0.39		498.96	0.35	
UV Disinfection Effluent Trough	498.96		498.04	0.92	0.27	498.2	0.76	0.11
Plant Water Station	511.0	497.48	497.91	13.09		497.99	13.01	
Plant Water Station Effluent	511.0		495.54	15.46	1.94	495.56	15.44	1.92

1. TOC - Top of concrete.
2. Highlighted items indicate that wall freeboard or weir submergence is out of desired range 1.0 ft and 0.25 ft, respectively.
3. Overflow pipe at top of tank modeled as a weir.

AECOM

TABLE 4 – ROUTE 7 WWTF HYDRAULIC PROFILE
AT 0.6 MGD AND 0.72 MGD WITH LARGEST UNIT NOT IN SERVICE

	Field Conditions		Hydraulic Conditions at 0.60 MGD			Hydraulic Conditions at 0.72 MGD		
Treatment Unit	TOC Elevation [1]	Weir Elevation	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.
Plant Influent Chamber	530.0		528.5	1.48		529.1	0.84	
Grit Chamber	530.0		528.4	1.63		529.1	0.87	
Comminutor	529.0		528.2	0.84		528.9	0.11	
Primary Settling Tanks	529.0	526.83	527.3	1.70		527.9	1.12	
Primary Settling Tank Effluent Trough	529.0		527.3	1.70	-0.47	527.9	1.15	-1.05
Equalization Tank	527.0	525.58 [3]	526.0	1.01		526.12	0.95	
Downstream Processes [4]	NA	NA	NA	NA	NA	NA	NA	NA

1. TOC - Top of concrete.
2. Highlighted items indicate that wall freeboard or weir submergence is out of desired range 1.0 ft and 0.25 ft, respectively.
3. Overflow pipe at top of tank modeled as a weir.
4. Processes downstream of the equalization tank were not modeled at the higher flow rates as it was assumed the equalization tank was operating and limiting the downstream flow to 0.30 mgd.

TABLE 5 – ROUTE 7 WWTF UNIT PROCESS HYDRAULIC CAPACITY WITH ONE UNIT OUT OF SERVICE

Treatment Unit	Unit Process Capacity	Comment
Plant Influent Chamber	Between 0.60 mgd and 0.72 mgd	
Grit Chamber	Between 0.60 mgd and 0.72 mgd	
Comminutor [1]	Between 0.50 mgd and 0.60 mgd	
Primary Settling Tanks	Greater than 0.72 mgd	Simulations capped at 0.72 mgd
Primary Settling Tank Effluent Trough	Less than 0.30 mgd	
Equalization Tank	Less than 0.72 mgd	
Rotating Biological Contactors	0.0 mgd [2]	RBC effluent weir is 6 in below the top of tank
Secondary Settling Tanks	Greater than 0.3 mgd	Simulations capped at 0.3 mgd downstream of EQ
Secondary Settling Tank Effluent Trough	Less than 0.12 mgd	
UV Disinfection	0.0 mgd [2]	UV effluent weir is less than 12 inches below UV channel top
Plant Water Station	Greater than 0.30 mgd	Simulations capped at 0.3 mgd downstream of EQ

1. Additional flows were evaluated that were not presented in Table 2, Table 3, and Table 4.
2. This structure has not been reported to have overtopped in the past and has been able to convey the flows recorded at the WWTF.

Hydraulic Relief Modifications

In order to increase the hydraulic capacity of each WWTF unit process, potential modifications to the WWTF unit processes/components that were limiting capacity were identified. The modifications are described below by unit process. These modifications will not be evaluated under the Phase 1 Facilities Planning efforts. An evaluation of these modifications and an assessment of the estimated costs will be performed as part of the Phase 2 Facilities Planning efforts as applicable.

Preliminary Treatment - Plant Influent Chamber, Grit Chamber and Comminutor. Based on the modeling, the hydraulic capacity of the plant influent chamber is slightly less than 0.72 mgd, the hydraulic capacity of the grit chamber is slightly less than 0.72 mgd, and the hydraulic capacity of the comminutor is between 0.50 mgd and 0.60 mgd. As noted in the tables, the capacity of the each unit process was evaluated with one unit out of service. For preliminary treatment, it was assumed that all of the flow was directed to the aerated grit chamber and no flow was directed to the channel containing a manual bar rack. In order to provide the desired recommend one foot of freeboard in the influent channel, the grit chamber and the comminutor at the 0.72 mgd peak flow the following modifications could be considered:

- Raising the slide gate in the bar rack channel so that the top of the gate is located slightly less than 1 foot below the top of the concrete. At this gate elevation, as flows increase to where the water elevation in the Plant Influent Chamber was approaching less than one foot of freeboard, additional flows would overtop the slide gate in the bar rack channel and provide hydraulic relief.
- Increasing the wall height of the influent chamber, grit chamber and around the comminutor.
- Replacement of the comminutor with a lower headloss type or other technology (channel grinder, fine screen, etc).
- Discussing the freeboard requirements with the Connecticut DEEP to see if this unit process could be grandfathered from the one foot freeboard guideline.

Primary Settling Tanks. Based on the modeling, the hydraulic capacity of the primary settling tanks is greater than 0.72 mgd. However the water surface elevation in the primary settling tank effluent trough is less than the desired 0.25 ft from the weir elevation in primary settling tanks at 0.30 mgd with the issue becoming more problematic at higher flows. In order to provide the desired 0.25 ft between the weir and the downstream water surface the following modification could be considered:

- Upsizing the 6 inch discharge pipe on the effluent trough to increase its hydraulic capacity.
- Adding a second discharge pipe to the effluent through to increase its hydraulic capacity.
- Increasing the width of the effluent trough.

Equalization Tank. Based on the modeling, the hydraulic capacity of the equalization tank is slightly less than 0.72 mgd. However, it should be noted that the model was configured with the all of the flow exiting the tank through the effluent overflow pipe at the top of the tank and not through the flow equalization piping and flow control valve at the bottom of the tank as intended by the original design. If only the overflow piping was to be used in the future, the following modifications could be considered to increase the hydraulic capacity to 0.72 and maintain a one foot freeboard:

- Lowering the overflow effluent pipe.
- Increasing the size of the 4 inch overflow pipe.
- Adding a second discharge pipe to the effluent through to increase its hydraulic capacity.

If flow equalization piping and flow control valve at the bottom of the tank is to be upgraded and used in the future, it is not believed that any further modification would be required to the tank to allow for one foot of freeboard at the WWTF peak flow of 0.72 mgd.

Rotating Biological Contactors (RBCs). Based on the modeling, the hydraulic capacity of the rotating biological contactors is 0.0 mgd. This is as a result of the weir in the RBCs being only 6 inches below tank walls therefore never allowing for one foot of freeboard regardless of the flow. At a flow of 0.30 mgd there was 0.35 feet of freeboard between the water surface and the top of the concrete. In order to provide the desired recommend one foot of freeboard at the 0.30 mgd flow, possible modifications include:

- Lowering of the RCB weir to allow for one foot of freeboard. This alternative would need to be evaluated in more detail, as lowering the water surface in the RBC reactors could have an impact of the treatment performance/capacity of the reactors.
- Increasing the wall height of the RBC tanks

- Discussing the freeboard requirements with the Connecticut DEEP to see if this unit process could be grandfathered from the one foot freeboard guideline.

Secondary Settling Tanks. Based on the modeling, the hydraulic capacity of the secondary settling tanks is greater than 0.30 mgd. As noted above the hydraulic capacity analysis of this unit process was limited to 0.30 mgd. However, the water elevation in the final settling tank effluent trough is less than the desired 0.25 ft from the weir elevation in secondary settling tanks at 0.12 mgd with the issue becoming more problematic at higher flows. In order to provide the desired 0.25 ft between the weir and the downstream water surface the following modification could be considered:

- Upsizing the 4 inch discharge pipe on the effluent trough to increase its hydraulic capacity.
- Adding a second discharge pipe to the effluent trough to increase its hydraulic capacity.
- Increasing the width of the effluent trough.

Ultraviolet (UV) Disinfection. Based on the modeling, the hydraulic capacity of the UV disinfection system is 0.0 mgd. This is as a result of the weir in the UV system being less than one foot below the UV channel walls therefore never allowing for one foot of freeboard regardless of the flow. At a flow of 0.30 mgd there was 0.35 feet of freeboard between the water surface and the top of the UV channel. It should be noted that this UV system is housed inside the plant water pump station so any splashing over the top of the channels would be contained within the structure. It should also be noted that this channel configuration is the manufacturer's standard configuration for a unit of this size. In order to provide the desired recommend one foot of freeboard at 0.30 mgd, possible modifications include:

- Increasing wall height of the UV channel (not recommended due the potential to submerge the UV lamp ballasts).
- Discussing the freeboard requirements with the Connecticut DEEP to see if this unit process could be grandfathered from the one foot guideline.
- It should also be noted if the WWTF was upgraded, that the UV system would likely be replaced to allow a system that would be able to provide reliable service for the next 20 years. As a result of that potential replacement, other UV system configurations could be examined at that time. These include a channel UV system with higher channel walls or the use of a pressurized (in pipe) UV system where freeboard is not an issue.

Plant Water Station Chamber. Based on the modeling, the hydraulic capacity of the plant water station is greater than 0.30 mgd. It should be noted that when the UV system and effluent flow meter were modified from the original system to the current system, the plant water system use was discontinued. As noted above the hydraulic capacity analysis of this unit process was limited to 0.30 mgd. No modifications to this system are required from a hydraulic stand point if the flows downstream of the equalization tank remain limited to 0.30 mgd

ROUTE 7 WWTF LOADING CAPACITY

General

The WWTF's unit processes and associated equipment were sized to treat the design loadings of the WWTF. The WWTF is able to produce an acceptable effluent today. However pollutant loadings in the future may increase and have the possibility to exceed the capacity of some of the unit processes. Upgrades to some unit processes and the associated equipment may to be required to accommodate the higher loadings. As a result, an evaluation of the WWTF's loading capacity was performed. The results of this evaluation are presented below.

Effluent Permit Changes

The Route 7 WWTF received a new NPDES permit effective October 1, 2014. One of the most significant changes to the permit was the inclusion of a seasonal mass based effluent total phosphorus limit of 1.0 lbs/day. At the current three year average daily flow of 0.053 mgd this mass limit of 1.0 lbs/day equates to an effluent total phosphorus concentration of 2.26 mg/l. At the 0.12 mgd design flow the Route 7 WWTF would be required to meet an effluent total phosphorus concentration of 1.0 mg/l. The permit also contains a maximum monthly average total phosphorus concentration limit of 1.55 mg/l and a maximum day concentration of 3.11 mg/l. Based on its existing performance with an average daily effluent total phosphorus concentration of 5.3 mg/l, the Route 7 WWTF will not be able to meet the new seasonal mass limit and the monthly total phosphorus limit and may not always be able to meet the maximum day total phosphorus limit. Chemical addition to precipitate phosphorus may be the least cost approach to meeting the proposed phosphorus limits, but the solids capture within the final settling tanks and the solids generation rates will need to be assessed.

It is also possible that the Route 7 WWTF might receive a total nitrogen permit limit in the future. If a total nitrogen effluent limit were to be is imposed on the WWTF, other modifications to the WWTF would likely be required. Unlike the effluent phosphorus limit which is contained in the Route 7 WWTF NPDES permit, nitrogen limits on WWTF's in the state are imposed through the Nitrogen General Permit issued by the DEEP. The current Nitrogen General Permit, which was issued in January 2010, did not include limits for municipal WWTF's with a capacity of less than 1.0 mgd. However, the current Nitrogen General Permit expires on December 31, 2015. The DEEP is currently reviewing the Nitrogen General Permit, and it is possible that the Route 7 WWTF could receive an effluent limit for total nitrogen when the permit is re-issued. This will need to be considered in the Phase 2 Wastewater Facilities Plan.

Unit Process Evaluations

Each unit process was examined and the design capacity was reviewed against standards provided in "Guides for the Design of Wastewater Treatment Works (TR-16)" prepared by the New England Interstate Water Pollution Control Commission, 2011 edition and other industry standards including Wastewater Engineering, Treatment and Reuse 4th edition (Metcalf and Eddy). Appendix A contains mass balances for the current average conditions as well as the current maximum month conditions based on the last three years of design data at the Route 7 WWTF. In addition Appendix B contains design data sheets for each unit process at current average day conditions, design conditions, and at the maximum allowable conditions for each unit process at the Route 7 WWTF. The results of the mass balances and design data sheet are the basis of the unit process capacity discussions below.

Aerated Grit Chamber. The WWTF has a single aerated grit chamber that is 2 feet 6 inches wide, 12 feet 8 inches long with varying depth (average depth of approximately 5 feet). In section, the chamber is triangular in shape to accommodate the screw auger grit removal system. Historical plant data has indicated that this unit was sized for a peak flow of 0.75 mgd. Typical hydraulic detention times for aerated grit chambers are 3 to 5 minutes. At the average and peak design flow of 0.12 mgd and 0.75

mgd the hydraulic detention times are 14.4 minutes and 2.3 minutes, respectively. Using a minimum 3 minute detention time, the maximum allowable flow to the grit chamber is 0.58 mgd.

Primary Settling Tanks. The WWTF has two rectangular primary settling tanks, each 32 feet long, 7 feet wide and 8 feet deep. Per TR-16, the hydraulic overflow rates for a WWTF less than 1.0 mgd should be 600 gpd/ft^2 at average day conditions and 2,500 gpd/ft^2 for peak flow conditions. At the average and peak design flows of 0.12 mgd and 0.72 mgd the hydraulic overflow rates are 268 gpd/ft^2 and 1,674 gpd/ft^2, respectively both of which are within recommended loading rates. The maximum average daily flow and peak hourly flow for the primary settling tanks were evaluated. With both settling tanks in service, the maximum allowable average daily flow is 0.27 mgd and maximum allowable peak hourly flow is 1.12 mgd. These flows are well with the WWTF influent design flows of 0.12 mgd average daily flow and 0.72 mgd peak hourly flow.

Equalization Tank. The equalization tank volume is approximately 60,000 gallons. A flow mass evaluation was performed using a typical WWTF influent diurnal flow curve to determine to capacity of the equalization tank. Based on the evaluation it appears that the equalization tank has sufficient capacity to equalize the influent diurnal flow for an average day influent flow of approximately 0.44 mgd. This volume is well within the equalization tank design basis provided to equalize the maximum flow to 0.30 mgd.

Rotating Biological Contactors. The WWTF has two Rotating Biological Contactors (RBCs), each with four stages. The media for each RBC is 12 feet in diameter, 25 feet long, and provides a surface area of 100,000 ft^2. With both units online the total media surface area is 200,000 ft^2. The original media was replaced in kind around the year 2000. These RBCs were intended to provide for both BOD removal as well as seasonal nitrification. The RBCs were evaluated against a number of design criteria including hydraulic loading rate (gpd/ft^2), BOD loading rate (lbs/d*1,000ft^2), soluble BOD loading (lbs/d*1,000ft^2) and NH_3 loading (lbs/d*1,000ft^2). In addition due to the multistage (4-stage) configuration of the system, the RBCs were also evaluated against design criteria for the first stage BOD loading (lbs/d*1,000ft^2) and the first stage soluble BOD loading (lbs/d*1,000ft^2). Based on the evaluation, the limiting design criteria for the RBCs was for the first stage BOD loading (lbs/d*1,000ft^2). Based on these criteria and the RBCs influent loading (based on current plant concentration data), the maximum allowable average daily flow is 0.18 mgd with both RBCs in service. This flow is well within the WWTF influent design flow of 0.12 mgd average daily flow. It should be noted that while the design criteria is based on a minimum wastewater temperature of 13^{o}C the actual wastewater temperature observed at the WWTF during the evaluation period was typically under 13^{o}C in the winter between early December and the end of March with a low temperature of 10^{o}C to 9^{o}C for the influent and the effluent respectively. The impact of these low temperatures should be evaluated in more detail in Phase 2 of the Faculties Plan if the flows and loads to the WWTF are projected to increase significantly.

Secondary Settling Tanks. The WWTF has two rectangular secondary settling tanks, each 28 feet long, 7 feet wide, and 7 feet deep. For the purposes of evaluating the secondary settling tanks, hydraulic overflow rates of 400 gpd/ft^2 at average day conditions and 800 gpd/ft^2 for peak flow conditions were used. At the average and peak design flow of 0.12 mgd and 0.30 mgd (downstream of the equalization tank) the hydraulic overflow rates are 306 gpd/ft^2 and 765 god/ft^2, respectively both of which are within recommended loading rates. With both secondary settling tanks in service, the maximum allowable average daily flow is 0.16 mgd and maximum allowable peak hourly flow is 0.32 mgd. These flows are well with the WWTF influent design flow of 0.12 mgd average daily flow and 0.30 mgd peak hourly flow (downstream of the equalization tanks).

Ultraviolet Disinfection. The ultraviolet (UV) disinfection system at the Route 7 WWTF consists of an open channel Trojan 3000 PTP system that has a single channel with 4 rows (modules) of 2 lamps (total of 8 lamps). Based on information received from the manufacturer, the installed unit is rated for a peak hourly flow of 0.20 mgd at 65% ultraviolet transmittance. It should be noted that the maximum total daily flow at the WWTF during the data period evaluated was 0.162 mgd which occurred outside of the disinfection season and that the maximum instantaneous daily flows reported at the WWTF for the three year reporting period only exceeded 0.20 mgd 6 times during the disinfection season with the maximum of 0.255 mgd (note the total flow reported that day was 0.056 mgd). It is believed that the disinfection system is properly sized for the current flows but would not be able to handle the design peak flow.

Solids Handling. The solids handling process at the Route 7 WWTF consists of pumping/decanting of the sludge and scum from the primary and secondary settling tanks once a day. These solids are discharged periodically to two solids holding tanks which can be aerated. When needed, the residuals collected in these tanks are trucked off-site to the South Street WWTF for further treatment. The volume of these two sludge tanks is believed to be sufficient due to the relative infrequency of the current off-site disposal. If the flows and loads or treatment process are anticipated to change significantly then the size and or configuration of these storage tanks should be evaluated in more detail.

Summary. Based on the existing wastewater constituent data, the estimated or calculated constituent removal by unit process, the design data sheets, the mass balances, and the unit process descriptions above, the loading capacity if the unit capacity of each unit process at the Route 7 WWTF was evaluated. Table 6 below presents process capacity of each unit process. As noted, the secondary settling tanks are the limiting process for average daily flow, and the UV system is the peak flow's limiting process.

TABLE 6 – ROUTE 7 WWTF UNIT PROCESS LOADING CAPACITY

Treatment Unit	Unit Process Capacity [1]	Limitation Comment
Grit Chamber	Peak Hour Flow - 0.58 mgd	Hydraulic Detention Time Limitation
Primary Settling Tanks	Ave Daily Flow - 0.27 mgd Peak Hour Flow – 1.12 mgd	Capacity in Excess of Maximum Influent Conditions
Rotating Biological Contactors	Ave Daily Flow - 0.18 mgd	Capacity in Excess of Maximum Influent Conditions
Secondary Settling Tanks	Ave Daily Flow - 0.16 mgd Peak Hour Flow - 0.32 mgd	Capacity in Excess of Maximum Influent Conditions and attenuated peak flow conditions
UV Disinfection	Peak Hour Flow - 0.20 mgd	Capacity per information from the manufacturer

1. The loading was based on increasing flows at current WWTF influent concentrations.

Unit Process Treatment Capacity Limitation Relief Modifications

Modifications to the WWTF unit processes/components were identified for unit processes/components that had capacities limited to less than the WWTF average day or peak hour capacities. These modifications are described by unit process below as applicable. An evaluation of these modifications and an assessment of the estimated costs will be performed as applicable as part of the Phase 2 Facilities Planning efforts.

Grit Chamber. The grit chamber was identified as having a peak hourly flow limitation of 0.62 mgd. This limitation is based on the hydraulic detention time in the grit chamber. Modifications to increase the capacity include:

- Replacing the grit chamber with one with a larger volume to increase the hydraulic detention time.

Another option would be to consider no change in the grit chamber configuration. This should be given consideration due to the limited duration of high flows to the WWTF coupled with the fact that the flow is generated from a small separated collection system with minimal grit served by the grit chamber. In addition, slightly more grit passing the grit chamber, while undesirable, should have little or no impact on the WWTF effluent and the financial impact associated with the additional wear that might occur in the primary sludge pumps will be significantly less than the installation cost of a new larger grit chamber.

UV Disinfection System. The UV disinfection system was identified as having a peak hourly flow limitation on 0.20 mgd. This limitation is based information obtained from the manufacturer. Modifications to increase the capacity include:

- Replacement of the UV system with a higher capacity system.
- The addition of a second UV system to operate in parallel or in series with the existing system.
- Modification of the existing system to increase the number of lamps or modules (need to confirm with manufacturer).
-

Phosphorus Removal. The new WWTF NPDES permit for the Route 7 WWTF contains total phosphorus limits to be achieved no later than September 2019. Based on the permit, the future effluent total phosphorus concentration limits are a monthly average of 1.55 mg/l, a maximum day of 3.11 mg/l and the seasonal average mass limit of 1.0 lbs/day between April 1st to October 31st. Based on the WWTF's current average effluent total phosphorus concentration is 5.3 mg/l potential modifications to meet the new total phosphorus limits include:

- Single or multi point chemical phosphorus removal (solids removal would occur in the existing settling tanks).
- Biological phosphorus removal (this would require the construction of an activated sludge process).

Opinion to Re-Rate the Route 7 WWTF

Based on results of the hydraulic and loading capacity analysis the potential to "re-rate" the Route 7 WWTF to a higher capacity was evaluated. Based on the evaluation it does not appear the Route 7 WWTF can be re-rated to a higher capacity as is. Based on current flow and loads there are hydraulic limitations in the RBCs, UV system, and secondary settling tank effluent troughs. In addition based on industry standards, the grit chamber does not have enough detention time at the design peak flow of 0.72

mgd. Finally, according to the manufacturer the UV system does not have the capacity to handle the design peak flow of 0.30 mgd.

SOUTH STREET WWTF PLANT CAPACITY ANALYSIS

As noted above, Sewer District 1 is served by the South Street WWTF, which discharges its treated wastewater to the Great Swamp. Until the early 1970's, the South Street WWTF consisted of primary treatment followed by continuously backwashing sand filtration. In 1973-74 the WWTF was upgraded to provide extended aeration with a design capacity of 0.72 mgd. The WWTF was subsequently upgraded/expanded in the early 1990's. This upgrade/expansion included the installation of a new influent headworks building, new aeration tanks to provide carbon oxidation as well as nitrification, new final settling tanks, continuously backwashing sand filters, post aeration, ultraviolet disinfection, sludge storage, and sludge thickening/dewatering. The 1990's upgrade/expansion design capacity is an average daily flow of 1.0 mgd and a peak hourly flow of 4.1 mgd. Figure 8 provides a layout of the South Street WWTF. Figure 9 presents a process flow schematic of the existing South Street WWTF.

Existing and Design Flows and Loads

Influent Flow, Concentration and Load Data. The existing influent flows and concentrations of wastewater constituents for the WWTF for the period between July 1, 2010 and June 30, 2013 were evaluated. Based on this review, the current annual average daily flow is approximately 0.85 mgd, the maximum month flow is 1.83 mgd, the maximum dally flow is 4.51 mgd, and the maximum instantaneous peak flow is 5.88 mgd. Figure 10 presents the WWTF influent flow data over the three year evaluation period.

A preliminary analysis of influent concentration and loading data for the primary pollutants (BOD, TSS, Total Kjeldahl Nitrogen (TKN), and TP) for the July 1, 2010 and June 30, 2013 period showed a great deal of variability, particularly with regards to TSS. As an example, some influent TSS data was greater than 1,000 mg/l with a maximum concentration of 2,420 mg/l and a minimum concentration of 11 mg/l. It is believed that some of the data may not be representative of the true WWTF influent.

During the July 1, 2010 and June 30, 2013 period, the WWTF received septage five to six days a week averaging 7,500 gal/day, with some days exceeding 20,000 gal/day. The septage is discharged to two septage holding tanks and then is pumped over the course of the day to a collection box just outside of the Influent Building. The WWTF's composite auto sampler withdraws its samples from this collection box. In addition to the septage being highly variable and concentrated, its discharge during low flow periods when mixing may be poor could bias the composite sample beyond what would be expected from the true weighted average of the influent and septage streams.

As a result, it was necessary to evaluate the data set for each parameter and to use a degree of engineering judgment, backed by experience at other local facilities as well as textbook references, to truncate data that appears to be unrepresentative. A description of this review process for each parameter is as follows.

Data Truncation. The data for the primary pollutants was plotted in histograms that provide a graphical representation of the distribution of data. Histograms are a convenient visual mechanism for reviewing a

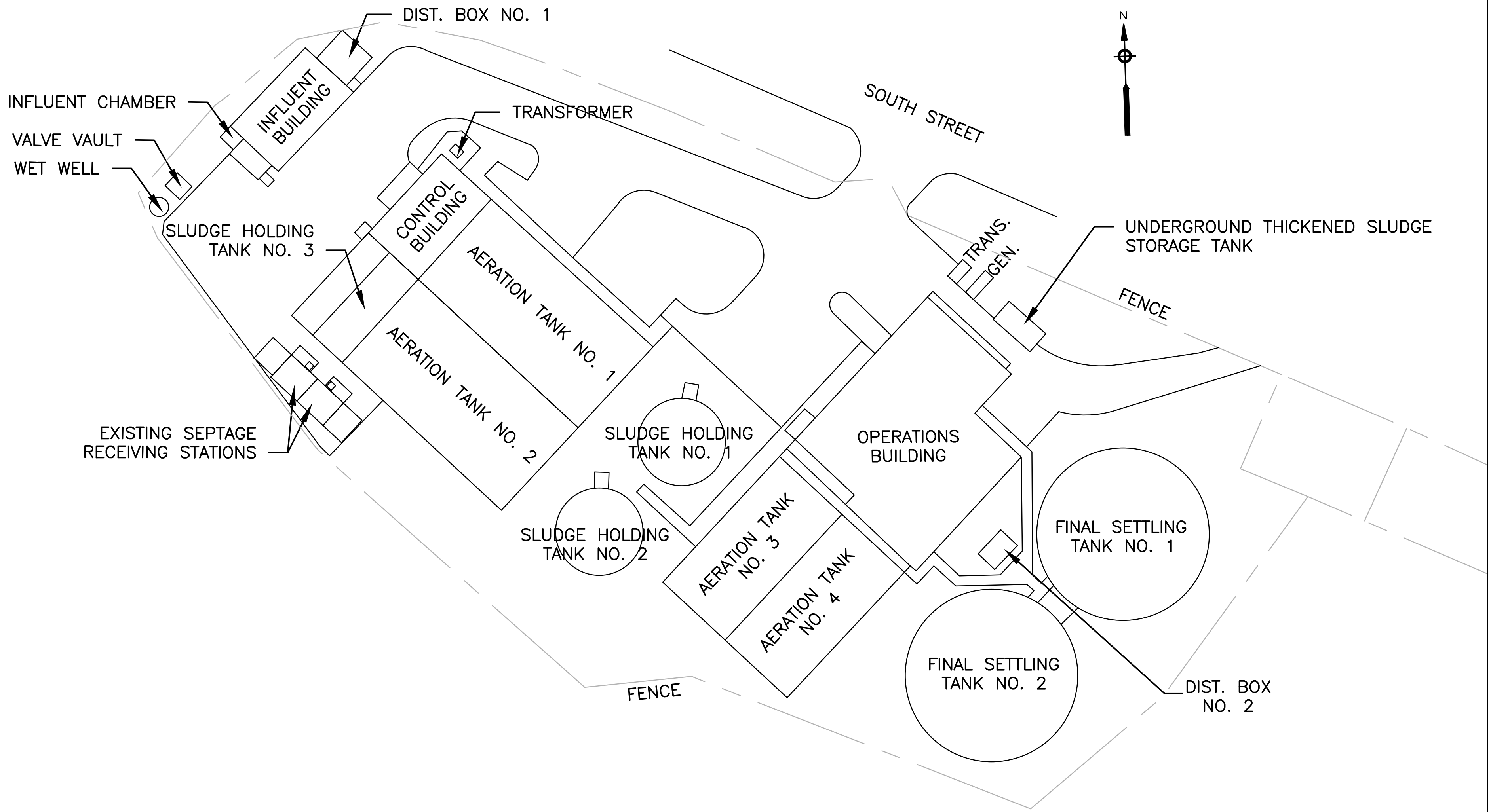


FIGURE 8 - SOUTH STREET WWTF SITE PLAN

NOT TO SCALE

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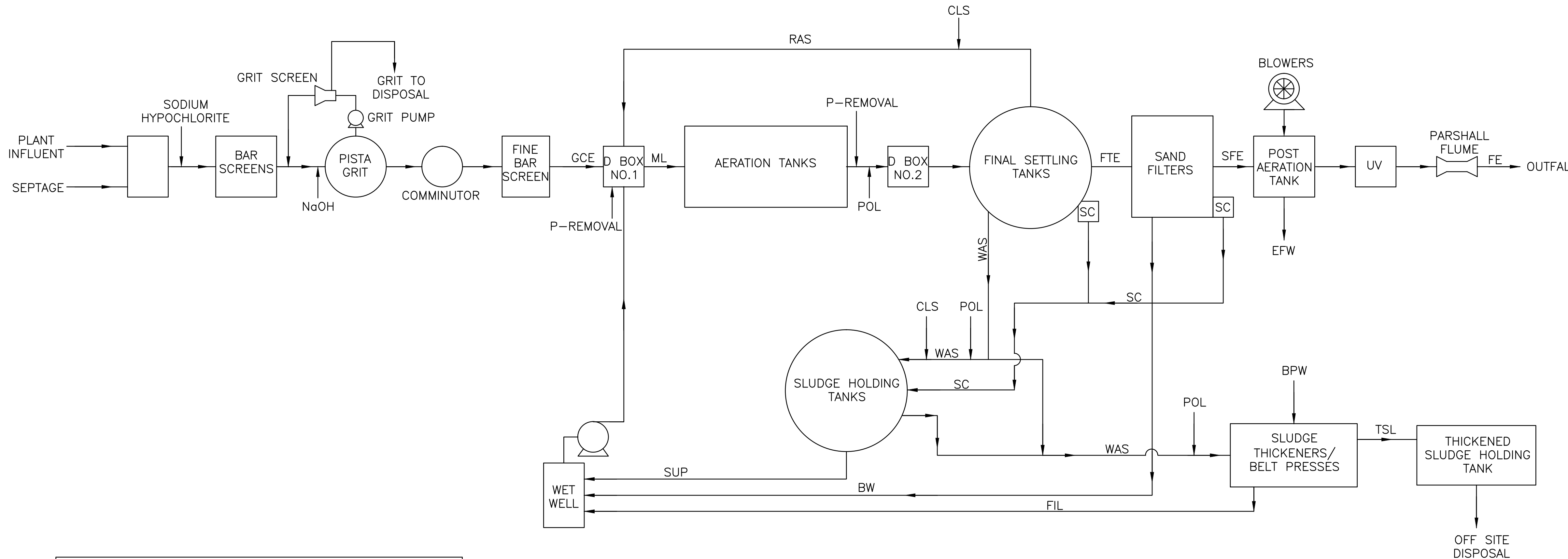

LEGEND

FE	FINAL EFFLUENT
BW	BACKWASH
SC	SCUM
FIL	FILTRATE
POL	POLYMER
RAS	RETURN ACTIVATED SLUDGE
WAS	WASTE ACTIVATED SLUDGE
SUP	SUPERNATANT
GCE	GRIT CHAMBER EFFLUENT
ML	MIXED LIQUOR
FTE	FINAL SETTLING TANK EFFLUENT
SFE	SAND FILTER EFFLUENT
BPW	BELT PRESS WASH WATER
EFW	EFFLUENT FLUSHING WATER
CLS	CHLORINE SOLUTION
TSL	THICKENED SLUDGE

FIGURE 9 -
SOUTH STREET WWTF
PROCESS FLOW SCHEMATIC

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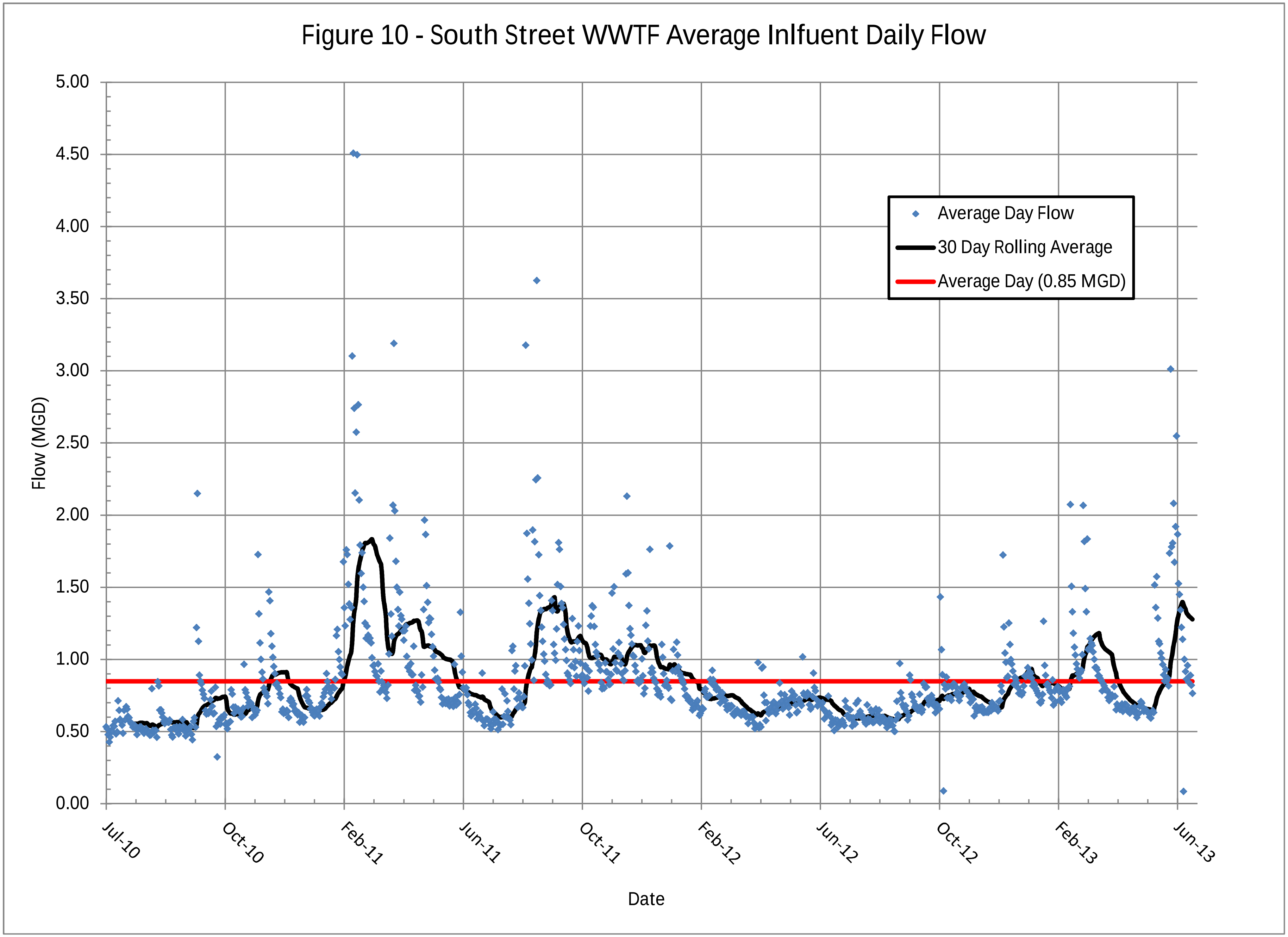
Figure 10 - South Street WWTF Average Inlfuent Daily Flow
Average Day Flow
30 Day Rolling Average
Average Day (0.85 MGD)
Flow (MGD)
5.00
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date

data set and for helping to identify "outliers" or values that should be disregarded due to suspect sampling and/or analysis. Table 7 presents the range of reported concentration data and the truncated concentration data set to eliminate potentially unrepresentative data.

TABLE 7 - SOUTH STREET WWTF PRIMARY POLLUTANT DATA TRUNCATION

Primary Pollutant	Data Range Reported	Truncated Data Set
BOD	53 mg/l - 480 mg/l	100 mg/l - 400 mg/l
TSS	11 mg/l - 2,420 mg/l	75 mg/l - 500 mg/l
TKN	10 mg/l - 73 mg/l	10 mg/l – 50 mg/l
TP [1]	1.2 mg/l - 9.5 mg/l	1.2 mg/ - 9.5 mg/l

1. The total phosphorus data was not truncated

Figure 11 presents the truncated influent TSS concentration data, Figure 12 presents the truncated influent concentration BOD data, Figure 13 presents the truncated influent TKN concentration data and Figure 14 presents the total phosphorus concentration data.

In addition, Table 8 summarizes the South Street WWTF flow and truncated wastewater constituent data for the period July 1, 2010 to June 30, 2013 for the WWTF influent and the final effluent including maximum month conditions. Note due to the limited number of daily samples collected for analysis, the maximum month loading conditions were based on the 92nd percentile of all of the data while the maximum month concentration data was back calculated from the maximum month loading conditions and the maximum month flow. These values were used to establish the baseline for Biowin model calibration as well as the baseline for a sensitivity analysis on the capacity of the biological process discussed later in this memorandum.

Design Flow and Loading Comparison

Flows. During the evaluation period of July 1, 2010 to June 30, 2013, the average annual flow was 0.85 mgd, or 15% below the design flow of 1.0 mgd. The maximum month flow was 1.83 mgd versus the design maximum month of 1.9 mgd. Finally, there were two instances in March 2011 where the total daily flow exceeded the 4.1 mgd peak design flow, and twenty-one instances where the maximum recorded daily flow exceeded 4.1 mgd peak design flow.

Loads. The loads from the three years evaluated (7/2010-6/2013) are presented below in Table 9 and compared to the design loadings of the WWTF. Based on the comparison, the plant is slightly under loaded organically and more significantly under loaded from a solids and nitrogen standpoint. Based on the current flows to the WWTF, the influent organic concentrations are similar to the design concentrations while the TKN and TSS concentrations are slightly less. This may have been the result of the data truncation as discussed above.

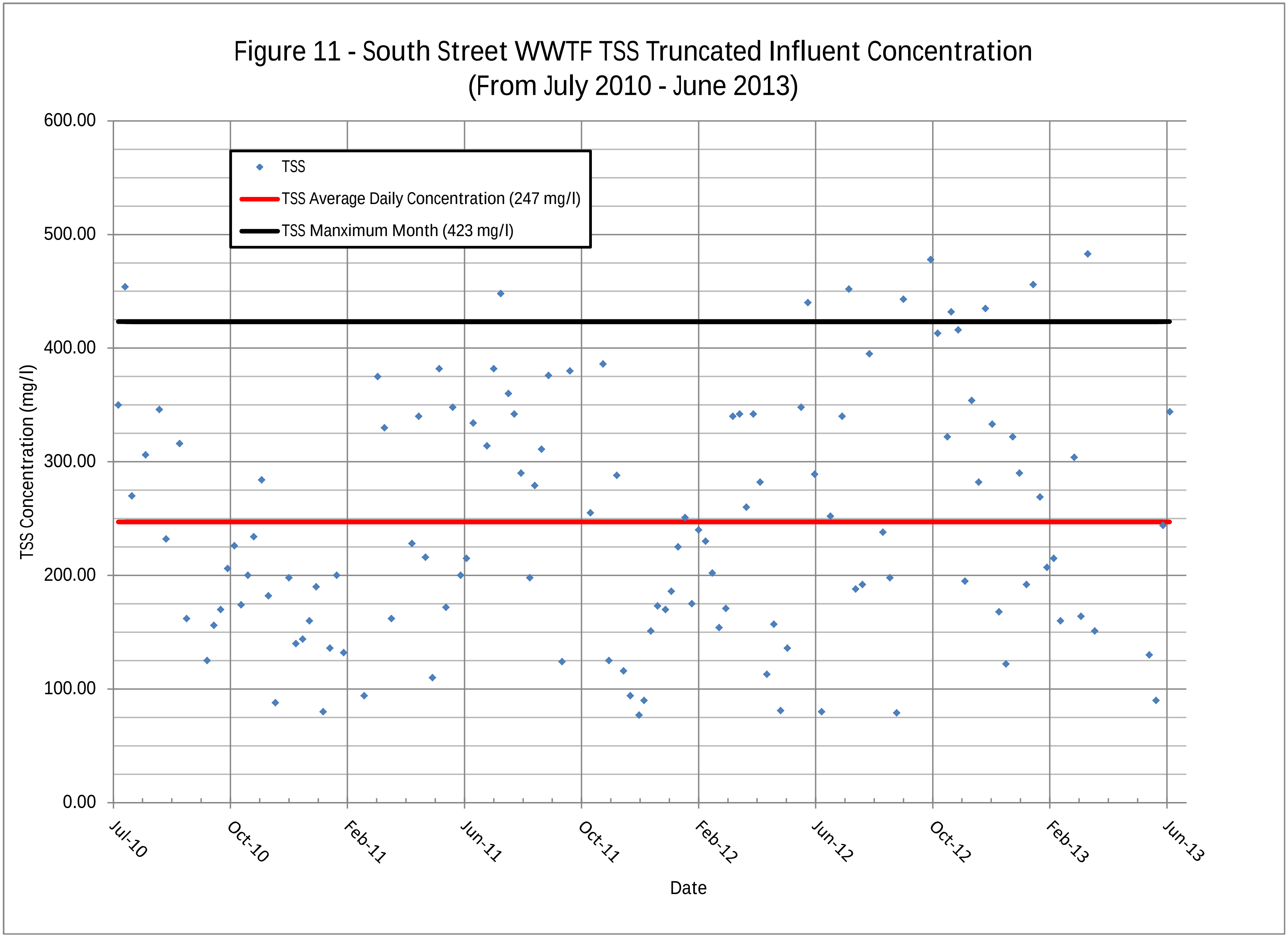
Figure 11 - South Street WWTF TSS Truncated Influent Concentration
(From July 2010 - June 2013)
TSS
TSS Average Daily Concentration (247 mg/l)
TSS Manximum Month (423 mg/l)
TSS Concentration (mg/l)
600.00
500.00
400.00
300.00
200.00
100.00
0.00
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date

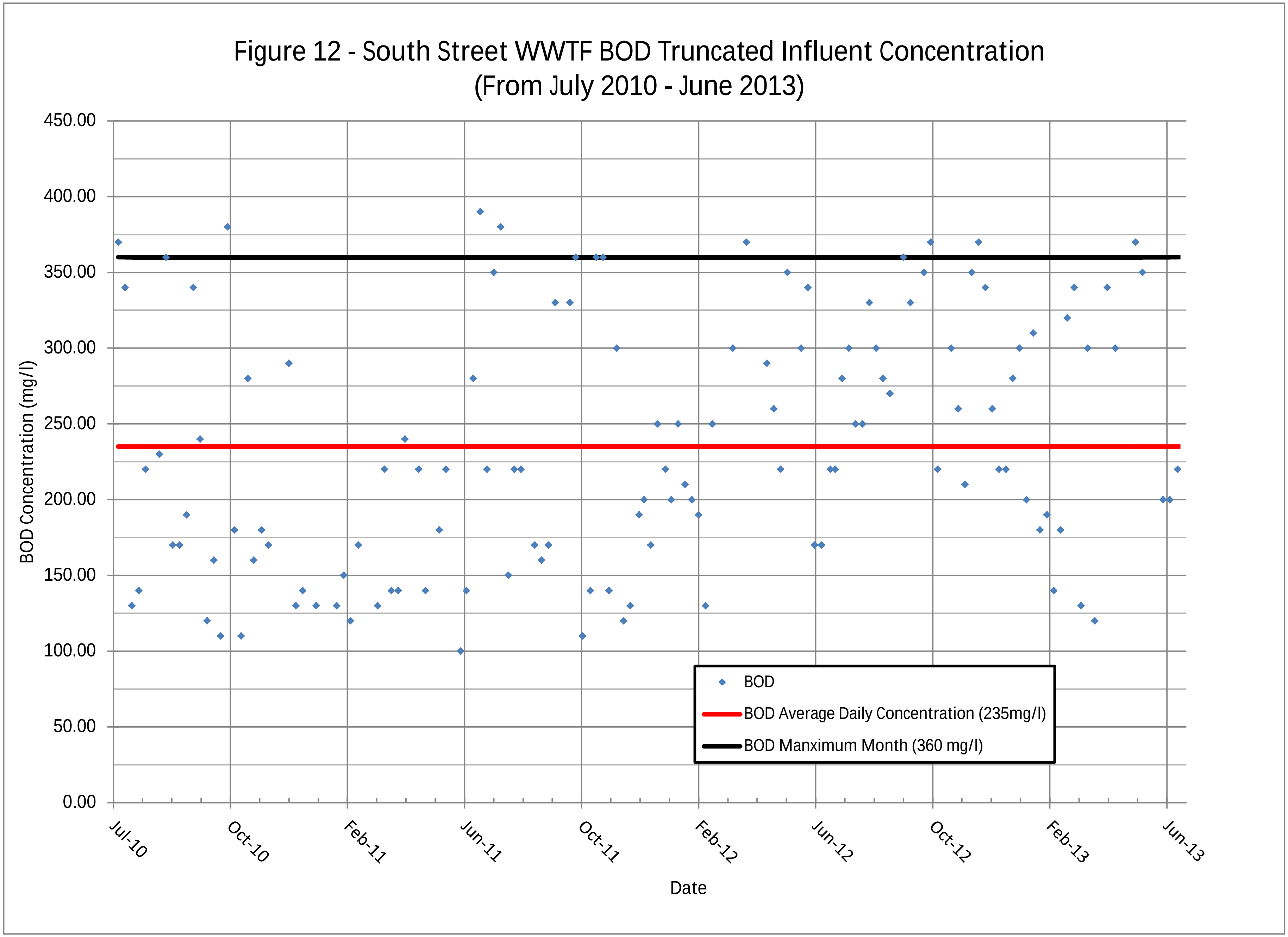

Figure 12 - South Street WWTF BOD Truncated Influent Concentration
(From July 2010 - June 2013)
450.00
400.00
350.00
300.00
250.00
200.00
150.00
100.00
50.00
0.00
BOD Concentration (mg/l)
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date
BOD
BOD Average Daily Concentration (235mg/l)
BOD Manximum Month (360 mg/l)

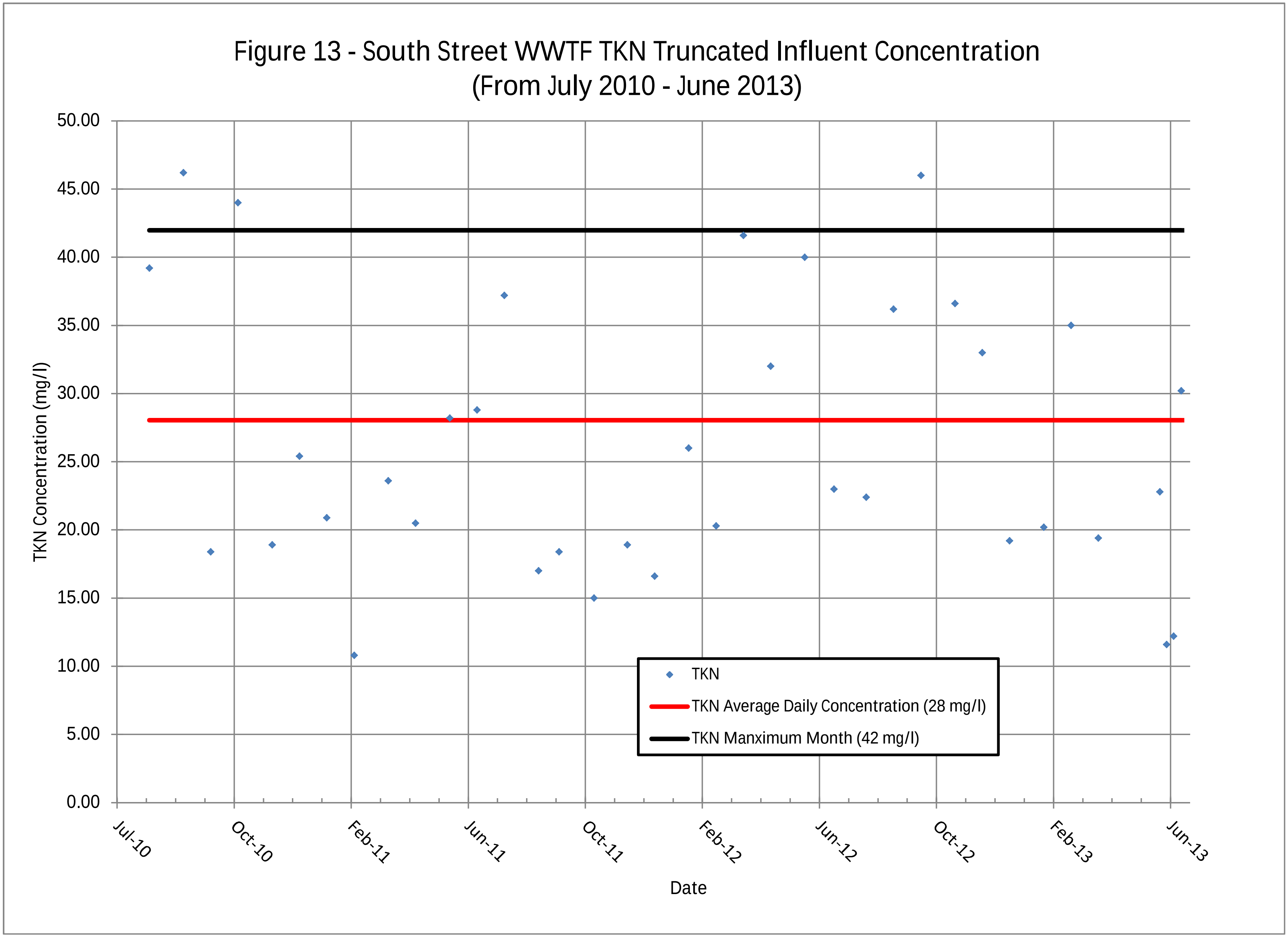
Figure 13 - South Street WWTF TKN Truncated Influent Concentration
(From July 2010 - June 2013)
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
TKN Concentration (mg/l)
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date
TKN
TKN Average Daily Concentration (28 mg/l)
TKN Manximum Month (42 mg/l)

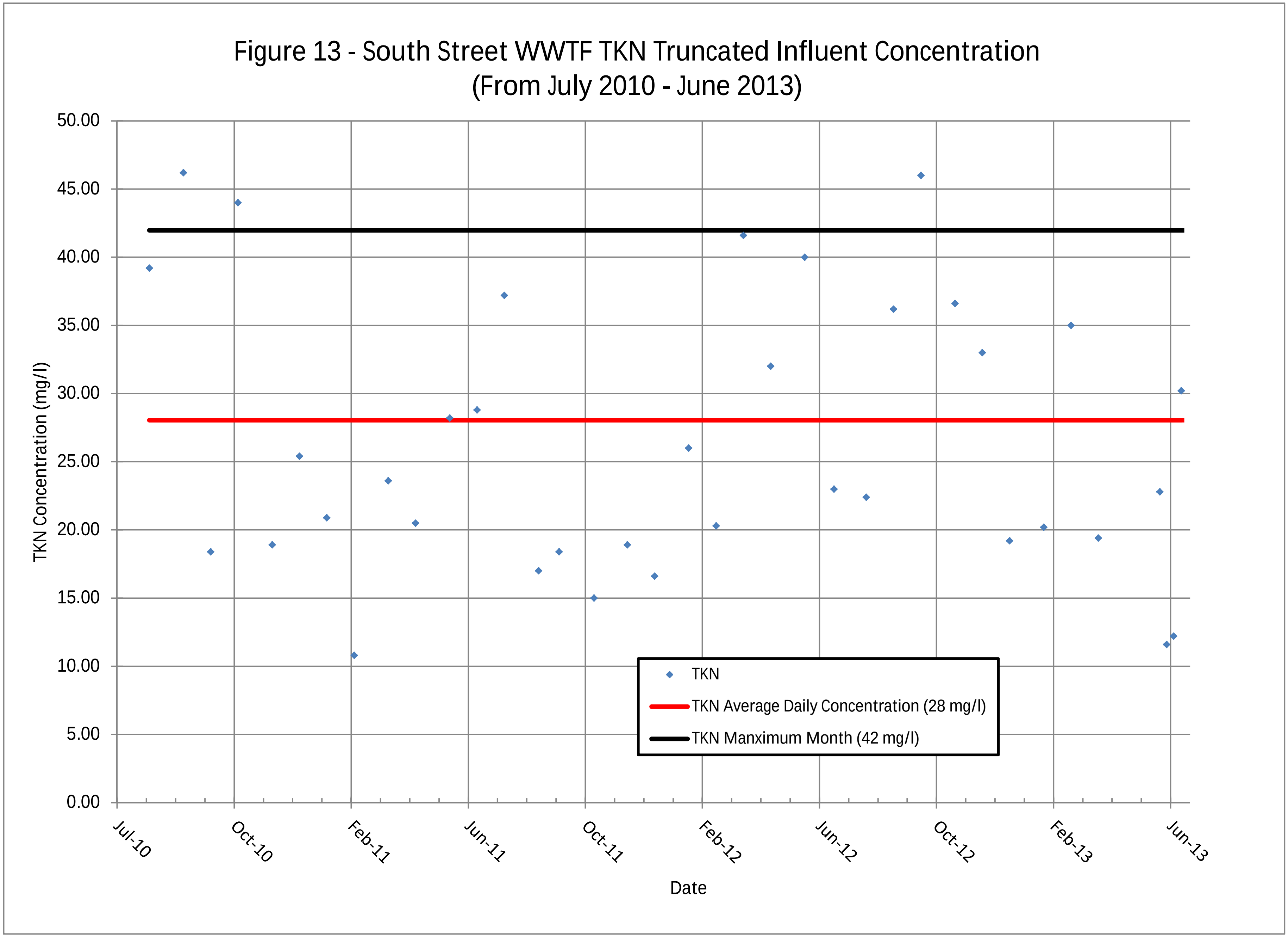

Figure 13 - South Street WWTF TKN Truncated Influent Concentration
(From July 2010 - June 2013)
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
TKN Concentration (mg/l)
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date
TKN
TKN Average Daily Concentration (28 mg/l)
TKN Manximum Month (42 mg/l)

TABLE 8 – SOUTH STREET WWTF FLOW AND LOADING SUMMARY (JULY 2010 TO JUNE 2013)

Parameter	Annual Average Day	Max Month Peaking Factor	Max Month
Influent			
Flow (mgd)	0.85	2.15	1.83
TSS (mg/l)	232		181
TSS (lb/d)	1,643	1.69	2,776
BOD_5 (mg/l)	219		158
BOD_5 (lb/d)	1,550	1.55	2,405
Total Kjeldahl Nitrogen (mg/l)	24.8		16.3
Total Kjeldahl Nitrogen (lb/d)	176	1.41	249
Total Phosphorus (mg/l)	4.0		3.1
Total Phosphorus (lb/d)	28.4	1.67	47.4
Zinc (kg/d) [1]	0.799	1.81	1.446
Effluent Discharged			
TSS (mg/l)	2.1		2.3
TSS (lb/d)	14.8	2.34	34.7
BOD_5 (mg/l)	2.2		2.1
BOD_5 (lb/d)	15.3	2.14	32.7
Ammonia Nitrogen (mg/l)	0.5		1.0
Ammonia Nitrogen (lb/d)	3.8	3.87	14.7
Total Nitrogen (mg/l)	5.9		4.2
Total Nitrogen (lb/d)	40.7	7.31	64.3
Total Phosphorus (mg/l)	0.2		0.3
Total Phosphorus (lb/d)	1.4	3.29	4.6
Zinc (kg/d) [1]	0.147	1.33	0.196

1. Zinc is reported by the WWTF in kg/day

TABLE 9 - SOUTH STREET WWTF DESIGN VERSUS CURRENT LOADING COMPARISON

Pollutant	Design Load [1]	Current Loads	Current Percent of Design Load
BOD			
Annual Average	2,000 lbs/day	1,550 lbs/day	78%
Maximum Month	3,000 lbs/day	2,405 lbs/day	80%
TSS			
Annual Average	2,900 lbs/day	1,643 lbs/day	57%
Maximum Month	4,300 lbs/day	2,776 lbs/day	65%
TKN			
Annual Average	360 lbs/day	176 lbs/day	49%
Maximum Month	500 lbs/day	249 lbs/day	50%

1. Design loads from the November 1987 Report on Wastewater Treatment and Sewer System Rehabilitation Needs prepared by Stearns and Wheler.

SOUTH STREET WWTF HYDRAULIC CAPACITY

The hydraulic capacity of the South Street WWTF was evaluated and an opinion of the current hydraulic capacity of each unit process in the facility has been provided. The approach to evaluate the South Street WWTF and a summary of the results is provided below.

Approach

Using the existing WWTF contract drawings, a computer based hydraulic model was constructed to represent the physical conditions at the WWTF. The WWTF's current flows from July 2010 to June 2013 and the design year flows from the South Street WWTF 1993 upgrade contract were modeled.

The results from the current and design year model runs were compared to the hydraulic profiles included in the 1993 South Street WWTF upgrade contract drawings. The hydraulic model was calibrated as needed so the model results reflected the hydraulic profiles in the contract drawings.

Once the model was calibrated, a number of hydraulic model runs with increasing flows were conducted for the WWTF to assess the hydraulic capacity of the existing WWTF. Specific unit process or WWTF components that limit the overall capacity at the WWTF were identified including quantification of their estimated hydraulic capacity. The capacity was evaluated with one of the redundant units out of service for applicable processes (for example the old aeration tanks (No.1 and No. 2), one final settling tank, and one sand filter) based on industry standards.

For the purposes of this evaluation, unit processes/structures are considered to have limited hydraulic capacity if either of the following conditions occur:

- There is less than 1 foot of difference (freeboard) between the top of the structure (top of concrete) and the hydraulic grade line (water elevation). It is desirable to have the water surface in a tank or structure about 1 foot below the tank wall to allow for foaming and splashing without overtopping the walls.
- There is less than 3 inches between a flow control weir in a structure and the water surface elevation downstream of the weir. It is desirable to have about 3 inches between a flow control weir and the downstream water surface so the flow control weir performance is not impacted by the downstream water surface.

Finally in order to increase the hydraulic capacity of the WWTF, potential modifications to the WWTF unit processes/components limiting capacity were identified. These modifications were identified for possible subsequent evaluation in the Phase 2 Facilities Plan. Some or all of these modification will be evaluated and an assessment of the estimated costs of these modifications will also be performed as applicable as part of the Phase 2 Facilities Plan.

South Street WWTF Hydraulic Capacity

The hydraulic model for the South Street WWTF was run at the following flow rates to evaluate the hydraulic capacity of the WWTF:

- 0.85 MGD - Current average daily flow to the WWTF (July 2010 to June 2013).
- 1.00 MGD - Design average daily flow to the WWTF.
- 4.10 MGD - Design Peak flow to the WWTF.
- 5.00 MGD.
- 6.00 MGD.
- 7.00 MGD.

The model runs for the South Street WWTF are summarized in Table 10, Table 11, and Table 12 below. These tables include the top of concrete or top of wall (TOC) elevation for the different process tanks or structures, weir elevations in the different structures, the model run hydraulic grade lines (HGL), and resulting differences in HGL elevation to the TOC and weirs in each process.

Based on the model runs performed, the hydraulic capacity of each of the unit process was evaluated. The summary of the hydraulic capacity of each of the unit process is summarized in Table 13.

Hydraulic Relief Modifications

In order to increase the hydraulic capacity of each WWTF unit process, potential modifications to the WWTF unit processes/components limiting capacity were identified. The modifications are described below by unit process. These modifications were identified for possible subsequent evaluation in the Phase 2 Facilities Plan. Some or all of these modifications will be evaluated and an assessment of the estimated costs of these modifications will also be performed as applicable as part of the Phase 2 Facilities plan.

Plant Influent Chamber. Based on the modeling, the hydraulic capacity of the plant influent chamber is slightly less than 4.50 mgd. In order to provide the desired recommend one foot of freeboard in the influent chamber at higher flows, the following modifications could be considered:

- Increasing the wall height of the chamber.
- Replacement of the comminutor with a channel grinder if one with a lower head can be identified.
- Replacement of the comminutor and manually cleaned fine screen with a mechanically cleaned fine screen.

Mechanical Screen. Based on the modeling, the hydraulic capacity of the mechanical screen is slightly less than 4.50 mgd**.** In order to provide the desired recommend one foot of freeboard on the upstream side of the screen at higher flows, the following modifications could be considered:

- Increasing the wall height of the channel on the upstream side of the mechanical screen.
- Replacement of the comminutor with a channel grinder if one with a lower head can be identified.
- Replacement of the comminutor and manually cleaned fine screen with a mechanically cleaned fine screen.

Grit Chamber. Based on the modeling, the hydraulic capacity of the grit chamber is between 6.0 mgd and 7.0 mgd**.** In order to provide the desired recommend one foot of freeboard on the upstream side of the grit chamber at higher flows, the following modifications could be considered:

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TABLE 10 – SOUTH STREET WWTF HYDRAULIC PROFILE
AT EXISTING CONDITIONS (0.85 MGD) AND DESIGN FLOW (1.00 MGD) WITH LARGEST UNIT NOT IN SERVICE

	Field Conditions		Hydraulic Conditions at 0.85 MGD			Hydraulic Conditions at 1.00 MGD		
Treatment Unit	TOC Elevation [1]	Weir Elevation	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.	Hydraulic Grade Line	Delta to TOC, ft.	Delta to Weir, ft.
Plant Influent Chamber	608.0		605.78	2.22		605.85	2.15	
Influent Screen[3]	608.0		605.77	2.23		605.84	2.16	
Grit Chamber [3]	608.0		605.08	2.92		605.12	2.88	
Comminutor [3]	608.0		604.98	3.02		605.02	2.98	
Fine Screen [3]	608.0		604.47	3.53		604.51	3.49	
Distribution Box No. 1	606.5	603.65	603.97	2.53		604.10	2.49	
Distribution Box No.1 Effluent Chamber	606.5		602.08	4.42	1.57	602.14	4.36	1.51
Aeration Tanks Influent Channel	604.33		601.86	2.47		601.88	2.45	
Aeration Tanks	604.33	600.75	601.09	3.24		601.11	3.22	
Aeration Tanks Effluent Channel	603.0		599.93	3.07	0.82	599.95	3.05	0.80
Distribution Box No. 2	598.92	596.5	596.86	2.06		596.88	2.04	
Distribution Box No.2 Effluent Chamber	598.92		595.45	3.47	1.05	595.48	3.44	1.02
Final Settling Tanks	600.5	595.25	595.33	5.17		595.34	5.16	
Final Settling Tank Launder	600.5		594.26	6.24	0.99	594.29	6.21	0.96
Final Settling Tank Effluent Box	597.0		592.85	4.15		592.88	4.12	
Sand Filters	593.5	591.67	591.72	2.28		591.72	2.28	
Sand Filter Effluent	593.5		591.50	2.50	0.17	591.50	2.50	0.17
UV	593.5		591.50	2.50		591.50	2.50	
UV Effluent	593.5		587.54	6.46		587.69	6.31	
Parshall Flume	593.5		587.33	6.67		587.37	6.63	

1. TOC – Top of concrete.
2. Highlighted items indicate that wall freeboard or weir submergence is out of desired range 1.0 ft and 0.25 ft, respectively.
3. Upstream side of equipment.

TABLE 11 - SOUTH STREET WWTF HYDRAULIC PROFILE
AT PEAK DESIGN FLOW (4.10 MGD) AND 5.00 MGD WITH LARGEST UNIT NOT IN SERVICE

	Field Conditions		Hydraulic Conditions at 4.10 MGD			Hydraulic Conditions at 5.00 MGD		
Treatment Unit	**TOC Elevation [1]**	**Weir Elevation**	**Hydraulic Grade Line**	**Delta to TOC, ft.**	**Delta to Weir, ft.**	**Hydraulic Grade Line**	**Delta to TOC, ft.**	**Delta to Weir, ft.**
Plant Influent Chamber	608.0		606.97	1.03		607.26	0.74	
Influent Screen[3]	608.0		606.91	1.09		607.19	0.81	
Grit Chamber [3]	608.0		605.83	2.17		606.10	1.90	
Comminutor [3]	608.0		605.65	2.35		605.87	2.13	
Fine Screen [3]	608.0		605.09	2.91		605.30	2.70	
Distribution Box No. 1	606.5	603.65	604.56	1.94		604.77	1.73	
Distribution Box No.1 Effluent Chamber	606.5		603.86	2.64	-0.21	604.27	1.93	-0.92
Aeration Tanks Influent Channel	604.33		602.20	2.13		602.27	2.06	
Aeration Tanks	604.33	600.75	601.43	2.90		601.51	2.82	
Aeration Tanks Effluent Channel	603.0		600.27	2.73	0.48	600.35	2.65	0.40
Distribution Box No. 2	598.92	596.5	597.22	1.70		597.33	1.59	
Distribution Box No.2 Effluent Chamber	598.92		596.30	2.62	0.20	596.66	2.26	-0.16
Final Settling Tanks	600.5	595.25	595.40	5.10		595.41	5.09	
Final Settling Tank Launder	600.5		594.86	5.64	0.39	594.99	5.51	0.26
Final Settling Tank Effluent Box	597.0		593.42	3.58		593.56	3.44	
Sand Filters	593.5	591.67	591.80	2.20		591.82	2.18	
Sand Filter Effluent	593.5		591.55	2.45	0.12	591.58	2.42	0.09
UV	593.5		591.50	2.50		591.50	2.50	
UV Effluent	593.5		588.53	5.47		588.72	5.28	
Parshall Flume	593.5		587.97	6.03		588.10	5.90	

1. TOC – Top of concrete.
2. Highlighted items indicate that wall freeboard or weir submergence is out of desired range 1.0 ft and 0.25 ft, respectively.
3. Upstream side of equipment.

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TABLE 12 - SOUTH STREET WWTF HYDRAULIC PROFILE AT 6.00 MGD AND 7.00 MGD WITH LARGEST UNIT NOT IN SERVICE

	Field Conditions		Hydraulic Conditions at 6.00 MGD			Hydraulic Conditions at 7.00 MGD		
Treatment Unit	**TOC Elevation [1]**	**Weir Elevation**	**Hydraulic Grade Line**	**Delta to TOC, ft.**	**Delta to Weir, ft.**	**Hydraulic Grade Line**	**Delta to TOC, ft.**	**Delta to Weir, ft.**
Plant Influent Chamber	608.0		607.86	0.14		608.82	-0.82	
Influent Screen[3]	608.0		607.78	0.22		608.72	-0.72	
Grit Chamber [3]	608.0		606.93	1.07		607.99	0.01	
Comminutor [3]	608.0		606.64	1.36		607.63	0.37	
Fine Screen [3]	608.0		606.07	1.93		607.07	0.93	
Distribution Box No. 1	606.5	603.65	605.55	0.95		606.56	-0.05	
Distribution Box No.1 Effluent Chamber	606.5		605.47	1.03	-1.82	606.51	-0.01	-2.86
Aeration Tanks Influent Channel	604.33		602.36	1.97		602.45	1.88	
Aeration Tanks	604.33	600.75	601.60	2.73		601.68	2.65	
Aeration Tanks Effluent Channel	603.0		600.43	2.57	0.32	600.51	2.49	0.24
Distribution Box No. 2	598.92	596.5	597.55	1.37		597.91	1.01	
Distribution Box No.2 Effluent Chamber	598.92		597.12	1.80	-0.62	597.65	1.27	-1.15
Final Settling Tanks	600.5	595.25	595.43	5.07		595.44	5.06	
Final Settling Tank Launder	600.5		595.08	5.42	0.17	595.11	5.39	0.14
Final Settling Tank Effluent Box	597.0		593.71	3.29		593.86	3.14	
Sand Filters	593.5	591.67	591.84	2.16		591.86	2.14	
Sand Filter Effluent	593.5		591.63	2.37	0.04	591.67	2.33	0.00
UV	593.5		591.50	2.50		591.50	2.50	
UV Effluent	593.5		588.93	5.07		589.12	4.88	
Parshall Flume	593.5		588.23	5.77		588.35	5.65	

1. TOC - Top of concrete.
2. Highlighted items indicate that wall freeboard or weir submergence is out of desired range 1.0 ft and 0.25 ft, respectively.
3. Upstream side of equipment.

TABLE 13 – SOUTH STREET WWTF UNIT PROCESS CAPACITY WITH ONE UNIT OUT OF SERVICE

Treatment Unit	Unit Process Capacity	Comment
Plant Influent Chamber	Between 4.1 mgd and 4.50 mgd [1]	
Influent Screen[1]	Between 4.1 mgd and 4.50 mgd [1]	
Grit Chamber [1]	Between 6.0 mgd and 7.0 mgd	
Comminutor [1]	Between 6.0 mgd and 7.0 mgd	
Fine Screen [1]	Between 6.0 mgd and 7.0 mgd	
Distribution Box No. 1 [1]	Between 5.75 mgd and 6.0 mgd	
Distribution Box No.1 Effluent Chamber	Between 6.0 mgd and 7.0 mgd the TOC[2] Slightly less than 4.1 mgd for the weir	Weir impact not significant for flow control since only one flow path remains to ATs No. 3 and No.4
Aeration Tanks Influent Channel	Greater than 7.0 mgd	
Aeration Tanks	Greater than 7.0 mgd	
Aeration Tanks Effluent Channel	Greater than 7.00 mgd for TOC[2] Slightly less than 7.0 mgd for the weir [3]	Weir impact not significant for flow control since only one flow path remains out of ATs No. 3 and No.4
Distribution Box No. 2	Greater than 7.0 mgd	
Distribution Box No.2 Effluent Chamber	Greater than 7.0 mgd for TOC[2] Slightly less than 4.1 mgd for the weir [3]	Weir impact not significant for flow control since only one flow path remains to one FST
Final Settling Tanks	Greater than 7.0 mgd	
Final Settling Tank Launder	Greater than 7.0 mgd for TOC[2] Slightly less than 4.1 mgd for the weir [3]	At 7.0 mgd there is 0.14 ft between the weir and the downstream water surface
Final Settling Tank Effluent Box	Greater than 7.0 mgd	
Sand Filters	Greater than 7.0 mgd	
Sand Filter Effluent	0.85 mgd	Conservative UV system model parameter indicated less than 3 inches between weir and downstream water surface.
UV	Greater than 7.0 mgd	Manufacturer indicated system can pass 7.0 mgd but is not recommended
UV Effluent	Greater than 7.0 mgd	
Parshall Flume	Greater than 7.0 mgd	

1. Additional flows were evaluated that were not presented in Tables 10, 11, and 12.
2. TOC- Top of concrete.
3. Limitation not consider significant due to the fact that only one flow path remains.

- Increasing the channel wall height and slide plate height on the grit upstream side of the grit chamber.
- Replacement of the comminutor with a channel grinder if one with a lower head can be identified.
- Replacement of the comminutor and manually cleaned fine screen with a mechanically cleaned fine screen.

Comminutor. Based on the modeling, the hydraulic capacity of the comminutor is between 6.0 mgd and 7.0 mgd. In order to provide the desired recommend one foot of freeboard on the upstream side of the comminutor at higher flows, the following modifications could be considered:

- Increasing the channel wall heights and slide plate heights on the upstream side of the comminutor.
- Replacement of the comminutor with a channel grinder if one with a lower head can be identified.
- Replacement of the comminutor and manually cleaned fine screen with a mechanically cleaned fine screen.

Fine Screen. Based on the modeling, the hydraulic capacity of the fine screen is slightly less than 7.0 mgd**.** In order to provide the desired recommend one foot of freeboard on the upstream side of the fine screen at higher flows, the following modifications could be considered:

- Increasing the channel wall heights and slide plate heights on the upstream side of the fine screen.
- Replacement of the comminutor with a channel grinder if one with a lower head can be identified.
- Replacement of the comminutor and manually cleaned fine screen with a mechanically cleaned fine screen.

Distribution Box No.1 - Upstream of Weirs*.* Based on the modeling, the hydraulic capacity of the upstream side of the distribution box, based on providing one foot of freeboard, is between 5.75 mgd and 6.0 mgd. In order to provide the desired recommend one foot of freeboard on the upstream side of the weirs at higher flows, the following modifications could be considered:

- Increasing the distribution box wall height and slide plate heights on the upstream side of the weirs.
- Lowering the weirs in the box (which would require upsizing of the piping between the distribution box and the aeration tanks).
- Revising the discharge location of the return sludge downstream of the current location. Note this would have an impact on mixing, RAS distribution, and process control which would need to be evaluated further.

Distribution Box No.1 - Downstream of Weirs. Based on the modeling, the hydraulic capacity of the downstream side of the distribution box based on providing one foot of freeboard is between 6.0 mgd and 7.0 mgd. In order to provide the desired recommend one foot of freeboard on the downstream side of the weirs at higher flows, the following modifications could be considered:

- Increasing the distribution box wall height and slide plate heights on the downstream side of the weirs.
- Upsizing of the piping between the distribution box and the aeration tanks or providing parallel pipes
- Revising the discharge location of the return sludge downstream of the distribution box. Note this would have an impact on mixing, RAS distribution, and process control which would need to be evaluated further.
- Operating the aeration tanks as two sets of two tanks in series versus one set of four tanks in series. Note this would have an impact on the biological treatment and would most likely reduce the ability of the aeration tanks to remove nitrogen. This impact on treatment performance would need to be evaluated further.

Based on the modeling, the hydraulic capacity of the distribution box based on providing three inches between the weirs and the downstream water surface is slightly less than 4.1 mgd with the issue becoming more problematic at higher flows. The distribution box can distribute flows to either the new aeration tanks or the old aeration tanks. As noted in the tables, the capacity was evaluated with one unit out (set of aeration tanks out of service). With one set of aeration tanks out of service, the need to provide a reliable flow split to both sets of aeration tanks with the weirs is not required as there is only one flow path. Therefore the inability to provide three inches between the weirs and the downstream water elevation is not an issue with one set of aeration tanks out of service.

However, if it is still desired to provide the desired 0.25 ft between the weir and the downstream water surface at higher flows, with one set of aeration tanks out of service the following modifications could be considered:

- Increasing the weir height in the distribution box (this will have an impact on the water surfaces upstream and would need to be addressed).
- Upsizing of the piping between the distribution box and the aeration tanks or add a parallel pipes.
- Revising the discharge location of the return sludge downstream of the distribution box. Note this would have an impact on mixing, RAS distribution, and process control which would need to be evaluated further.
- Operating the aeration tanks as two sets of two tanks in series versus one set of four tanks in series. Note this would have an impact on the biological treatment and like reduce the ability of the aeration tanks to remove nitrogen. This impact on treatment performance would need to be evaluated further.

Aeration Tanks. Based on the modeling, the hydraulic capacity of the aeration influent channel, the tanks, and the effluent channel is greater than 7.0 mgd. However, based on the modeling, the hydraulic capacity of the effluent channel, based on providing three inches between the weirs and the downstream water surface, is slightly less than 7.0 mgd. As noted in the tables, the capacity was evaluated with one unit out (set of aeration tanks out of service). In addition, the model was run in worst case scenario with all four of the on-line aeration tanks run in series. With all four tanks run in series (versus two sets of two tanks run in series) the need to control the aeration tank flow split to both sets of online aeration tanks with the weirs is not required as there is only one flow path. Therefore the inability to provide three inches between the weirs and the downstream water elevation is not an issue with one set of aeration tanks out of service and all four of the online aeration tanks run in series.

However, if it is still desired to provide the desired 0.25 ft between the weir and the downstream water surface at higher flows, with one set of aeration tanks out of service the following modifications could be considered:

- Increasing the weir height in the aeration tanks (this will have an impact on the water surfaces upstream and would need to be addressed).
- Upsizing of the piping between aeration tank effluent and Distribution Box No 2.
- Operating the aeration tanks as two sets of two tanks in series versus one set of four tanks in series. Note this would have an impact on the biological treatment and like reduce the ability of the aeration tanks to remove nitrogen. This impact on treatment performance would need to be evaluated further.

Distribution Box No. 2. Based on the modeling, the hydraulic capacity of both the upstream side and downstream side of the distribution box, based on providing one foot of freeboard is greater than 7.0 mgd. However, the hydraulic capacity of the distribution box based on providing three inches between the weirs and the downstream water surface is slightly less than 4.1 mgd. This Distribution Box can distribute flows to one or both final settling tanks. As noted in the tables, the capacity was evaluated with one unit (final settling tank) out of service. With one final setting tank out of service the need to provide a reliable flow split to both final settling tanks with the weirs is not required. Therefore the inability to provide three inches between the weirs and the downstream water elevation is not an issue with one final settling tank out of service.

If it is still desired to provide the desired 0.25 ft between the weir and the downstream water surface at higher flows, with one set of aeration tanks out of service the following modifications could be considered:

- Increasing the weir height in the distribution box (this will have an impact on the water surfaces upstream and would need to be addressed).
- Upsizing of the piping between Distribution Box No. 1and the Final Settling Tanks (not practical as they are under the final settling tanks).

Final Settling Tanks. Based on the modeling, the hydraulic capacity of one final settling tank, based on providing one foot of freeboard is greater than 7.0 mgd. However, the hydraulic capacity of one final settling tank, based on providing three inches between the weirs and the downstream water surface is slightly less than 4.1 mgd. Note that at 7.0 mgd there is 0.14 feet between the weir and the downstream water surface. As noted in the tables, the capacity was evaluated with one unit (final settling tank) out of service. With one final setting tank out of service the need to provide a reliable flow split to both final settling tanks with the weirs is not required. Therefore the inability to provide three inches between the weirs and the downstream water elevation is not an issue with one final settling tank out of service as there is only one flow path.

However, if it is still desired to provide the desired 0.25 ft between the weir and the downstream water surface at higher flows, with one set of aeration tanks out of service the following modifications could be considered:

- Increasing the weir height in the final settling tanks (this will have an impact on the water surfaces upstream and would need to be addressed).
- Upsizing the wall penetration between the effluent lauders and the downstream effluent box.

Sand Filters. Based on the modeling, the hydraulic capacity of the sand filters, based on providing one foot of freeboard is greater than 7.0 mgd. However, the hydraulic capacity of the sand filters, based on providing three inches between the weirs and the downstream water surface is less than 0.85 mgd. The model is overly conservative for the headloss through the downstream UV flow control gate at low flows. Due to the limited hydraulic information on the UV system flow control gate, the water surface upstream of the UV systems (downstream of the sand filter weirs) was assumed to always be the UV manufacture's reported maximum water surface (591.50) from there shop drawing records. Based on this elevation, there will always be less than 3 inches of freeboard on the sand filter weir.

It should be noted that the flow split to the on-line sand filters will be impacted more by the headloss through the filters and less by the filter weirs. At 7.0 mgd, the model predicted that there is no difference between the downstream water surface elevation and the weirs. This can be significant if one set of sand filters is out of service as there is the possibility of the water in the downstream channel back filling the offline sand filter. It is anticipated that the UV system installed over 20 years ago will be replaced in any future upgrade or expansion. The limitations on the sand filter weir freeboard should be addressed with the selection and layout of a new UV system and its headloss.

In order to provide the desired recommend 0.25 feet of freeboard on the downstream side of the weirs at higher flows, the following modifications could be considered:

- Increasing the weir height of the sand filters. This would require modifications to the sand filter distribution and backwashing system components.
- Providing a new UV system with reduced system headloss. Options to reduce the UV system headloss include:
    - Use of different UV system level controllers (ex. actuated weir gates).
    - Increased UV channel width.

Ultraviolet (UV) Disinfection. Based on the modeling and information from the UV system vendor the hydraulic capacity of the UV disinfection system is 7.0 mgd. As noted above the UV system is impacting the weir freeboard on the sand filters. As this system will likely be replaced during the next major WWTF upgrade investigation into mean to reduce the headloss through a new UV system should be conducted.

Parshall Flume. Based on the modeling, the hydraulic capacity of the parshall flume is greater than 7.0 mgd. However is should be noted that this 9 inch flume is rated for a maximum flow of 5.8 mgd. While the flume can pass more than the rated 5.8 mgd its accuracy above the 5.8 mgd rating is reduced.

SOUTH STREET WWTF LOADING CAPACITY

General

The WWTF's unit processes and associated equipment were sized to treat the design loadings of the WWTF. The WWTF is able to produce an acceptable effluent today. However pollutant loadings in the future may increase and have the possibility to exceed the capacity of some of the unit processes. Upgrades to some unit processes and the associated equipment may to be required to accommodate the higher loadings. As a result, an evaluation of the WWTF's loading capacity was performed. The results of this evaluation are presented below.

Effluent Permit Changes

The South Street WWTF's current NPDES expired on September 29, 2009. The Town reapplied for the permit, and the existing permit remains in effect until a new permit is issued. It is anticipated that the WWTF will receive a new draft permit sometime in the near future. Based on the current information available from the DEEP, it is anticipated that the South Street WWTF will receive a seasonal mass based effluent total phosphorus limit of 1.04 lbs/day. At the current design flow of 1.0 mgd this equates to an effluent total phosphorus concentration of 0.12 mg/l. If the future flow to the South Street WWTF is projected to be greater than 1.0 mgd, the resulting required effluent total phosphorus concentration would be less than 0.12 mg/l. It is not anticipated that the existing unit processes at the WWTF would be able to meet a total phosphorus effluent limit of 0.12 mg/l. As a result, modification or additions to the WWTF would be required. In addition, there is the potential that the South Street WWTF may receive a more stringent total nitrogen limit when the Nitrogen General Permit is reissued and that the CT DEEP Nitrogen General Permit program that allows for purchasing of nitrogen credits may be modified or discontinued. Changes or elimination of the Nitrogen General Permit will have an impact on the WWTF and may require modification to allow the WWTF to improve nitrogen removal. Finally, there is also the potential for new or stricter metal limits to be included in the new South Street WWTF NPDES permit. It is anticipated that the South Street WWTF would not be able to meet new or stricter metals limits and as a result WWTF modifications would be required.

Of particular concern is the effluent limit on zinc in light of the past issues at the South Street WWTF with meeting the monthly average and daily maximum limits in the existing NPDES permit. In 2009, the US EPA, in conjunction with the CT DEEP, issued an Administrative Enforcement Order (AO) to the Town which required the Town undertake actions to address the levels of total zinc in the plant effluent that had periodically exceeded the permit limits. The Town complied with the requirements of the AO, and in March 2011 submitted a report entitled "Draft Report on the Investigation and Recommended Implementation Program to Achieve Total Zinc Limits of the South Street WWTF" prepared by AECOM to the EPA and the DEP. The report concluded that the largest source of zinc in the plant influent was from the water supply system, recommended that the Aquarion Water Company be asked to reduce or eliminate the use of a zinc based corrosion inhibitor in the water supply, and if that was not successful at addressing the zinc levels, then a zinc removal upgrade at the WWTF be considered. The zinc removal upgrade would involve construction of chemical storage and feed system for alum and sodium hydroxide as well as a flocculation chamber. Since the 2011 zinc report, the Aquarion Water Company has changed the corrosion inhibitor they have been using, and violations of the effluent zinc limits at the WWTF have become very infrequent. During the initial steps of the Phase 2 Facilities Plan, feedback from the DEEP will be necessary as to whether the zinc limits are to be revised, and if chemical precipitation for zinc will be necessary in the plant upgrade to meet the future effluent limit.

Unit Process Evaluations

Each unit process was examined and the design capacity was reviewed against standards provided in "Guides for the Design of Wastewater Treatment Works (TR-16)" prepared by the New England Interstate Water Pollution Control Commission, 2011 edition and other industry standards including Wastewater Engineering, Treatment and Reuse 4th edition (Metcalf and Eddy). Appendix C contains mass balances for the current average conditions as well as the current maximum month conditions based on the last three years of design data at the South Street WWTF. In addition Appendix D contains design data sheets for each unit process (with the exception of the Grit Chamber and Aeration Tanks) at current average day conditions, design conditions and at the maximum allowable conditions for each unit process at the South Street WWTF. The results of the mass balances and design data sheets are the basis of the unit process capacity discussions below. The vortex style grit chamber does not have a design data sheet as their sizing is based on manufacturer's recommendations and not established industry standards. In addition the aeration tanks are not presented with design data sheets as their analysis was performed via a Biowin model, which is discussed below.

Process Model for South Street. In order to assist in the evaluation of the biological process capacity at the South Street WWTF, a BioWin wastewater process model was developed specific to the South Street WWTF. The BioWin model was developed based on the existing process flow diagrams and the physical characteristics such as tank volumes, depths, aerator sizes, etc. This model will also be used in the Phase 2 Facilities Plan to assess biological upgrades to the South Street WWTF.

The South Street WWTF model results were compared to existing operating data, and mathematical variables in the model were adjusted to "calibrate" the model. The data used to calibrate the model was the normal WWTF operating/reporting data as well as the plant sampling data collected in fall of 2013. The model performance was compared to plant operating data and operating conditions (temperature, loadings, etc.) from various times during the year to provide a good fit of the respective model results and the respective WWTF field reported results.

Based on the mass balances, design data sheets, and the calibrated South Street biological model, each unit process at the WWTF was evaluated at different loading conditions to assess the loading capacity of the existing WWTF. These evaluations are summarized below.

Vortex Grit Chamber. The WWTF has a single vortex grit chamber that is rated for an influent flow of 4.1 mgd, and has a grit pumping capacity of 175 gpm. Based on discussions with the manufacturer, the system's maximum flow is 4.1 mgd. At flows in excess of the maximum rated flow the grit removal efficiency is anticipated to decrease. This decrease in capture efficiency is significant as the WWTF does not have primary clarifiers, and any grit that is not captured in the grit removal system will end up accumulating in the aeration tanks which do not have a means to remove the grit. Over time, grit entering the aeration tanks can accumulate to the point where it effectively reduces the tank volume available for biological treatment.

Aeration Tanks. The WWTF has four aeration tanks. Aeration Tank No. 1 and No. 2 are configured with two equally sized zones each, of dimensions 42 ft. x 42 ft. with a depth of 10.4 ft., providing a total volume of 137,000 gallon each. Aeration Tank No. 3 and No. 4 are similarly configured, with each zone having dimensions of 32 ft. x 32 ft. with a depth of 14.8 ft. providing a total volume of 113,000 gallons each.

Aeration Tank Loading - The design maximum month BOD loading of 3,000 lbs/d is approximately 20% higher than the actual maximum month BOD loading of 2,405 lbs/d suggesting some additional unused capacity. It is important to note however that the design was based on using all four aeration tanks in service and a mixed liquor suspended solids (MLSS) concentration of 3,500 mg/l. During the three years evaluated, the plant operated with only Aeration Tanks No. 3 and No. 4 in operation, essentially reducing the design reactor volume by more than half. This reduction in reactor volume however was offset in part by increasing the MLSS concentration in the two online aeration tanks. The annual average and maximum month concentration of MLSS within the aeration tanks during the timeframe evaluated was 5,300 and 6,500 mg/l, respectively.

A common metric used to quantify the loading on the biological process is the food to mass (F:M) ratio, which is the daily mass of BOD applied per mass of MLSS in the reactors. The design MLSS value and use of all four tanks resulted in a design F:M ratio of 0.09 lbs BOD/lb MLSS-d. The resultant design solids retention time (SRT) was 17 days. Assuming the maximum month BOD loading is coincident with maximum month MLSS concentrations, the actual F:M the plant has been operating at under maximum month conditions is 0.10 lbs BOD/lb MLSS-d, or just slightly higher than design.

Another consideration in comparing how the plant is operating relative to design is the portion of the total tank volume operated under aerobic conditions. The basis of design assumed that all of the aeration tank volume is operated under aerobic (aerated) conditions. During the three year evaluation period, the common practice was to operate the first zone of Aeration Tank No.3 without aeration to provide some denitrification in the process. The aerator in this zone, was cycled on periodically (10-15 minutes a day when sampling) to suspend solids. This operation results in anoxic conditions in 25% of operational reactor volume which effectively reduces the aerobic SRT by 25%. This reduction in SRT has a direct influence on nitrification stability (see the discussion on process sensitivity analysis below).

In summary, while maximum month BOD loadings are approximately 20% below design, the WWTF as currently operated is at capacity for organic loading. It should be noted that this is based on the current operating configuration with only the two aeration tanks in service and operating one zone in an anoxic mode (see the discussion on process sensitivity analysis below for further analysis).

Aeration Tank Mixer/Aerators. The function of the mixer/aerators is to keep the tank contents well suspended, and to provide sufficient aeration to maintain dissolved oxygen levels high enough to sustain aerobic biological activity, typically around 2 mg/l. Aeration Tank No. 1 and No. 2 are provided with two 15-HP constant speed units each (one per zone).

Aeration Tank No. 3 and No. 4 are provided with two speed mixer/aerators (one per zone) that are two speed and are capable of running at either 15-HP or 20-HP. Table 14 presets the flow path through the current online aeration tank zones and average DO profile in each zone for the three year reporting period.

TABLE 14 AERATION TANK ZONES DISSOLVED OXYGEN PROFILE

Aeration Tank Zone	Average Dissolved Oxygen, mg/l
Zone 3-B	0.3
Zone 3-A	0.4
Zone 4-A	3.7
Zone 4-B	5.1

As previously indicated, the WWTF cycles the aerator in Zone 3-B infrequently in order to suspend solids. It is therefore not surprising that the DO in the Basin is depressed below the 2.0 mg/l level typically desired. The depressed DO in Zone 3-A however suggests that the oxygen demand in this zone is higher than what the single 20-HP aerator is capable of supporting. Again, this should not be surprising as this aerator is seeing close to the full design load when it was intended to only the portion of the flow and load (roughly 45%) with the balance to be conveyed to Aeration Tanks No. 1 and No. 2. Additionally, the WWTF operates the Aeration Tanks No. 3 and 4 as four zones in series which results in the initial aerobic zone in service having the majority of the oxygen demand. The aerators in Zones 4-A and 4-B appear to keep up sufficiently. In summary, the aeration in Tank 3-A is currently inadequate and could possibly contribute to some degree of nitrification instability (see the discussion on process sensitivity analysis below).

Model Configurations Examined. The next portion of the evaluation was to evaluate the sensitivity of the biological process analysis to increases in load. The calibrated Biowin model was used to evaluate the response of a biological treatment process to changes in process flows and loads. The two model configurations that were evaluated are as follows:

- Configuration No. 1 - Only Aeration Tanks No. 3 and No. 4 in operation (run in series with the 1st zone anoxic) – Current operating conditions

- Configuration No. 2- All Aeration Tanks in operation. Tanks No. 1 and 2 run in series with the 1st zone anoxic and Tanks No. 3 and 4 run in series with the 1st zone anoxic.

Configuration No. 1 and Configuration No. 2 are shown below in Figures 15 and 16, respectively.

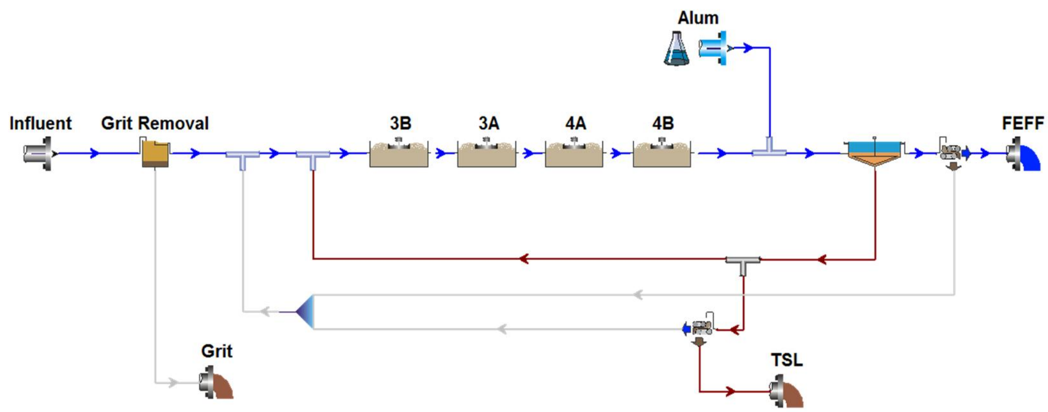


FIGURE 15 - CONFIGURATION NO. 1
ONLY AERATION TANKS NO. 3 AND NO. 4 IN OPERATION

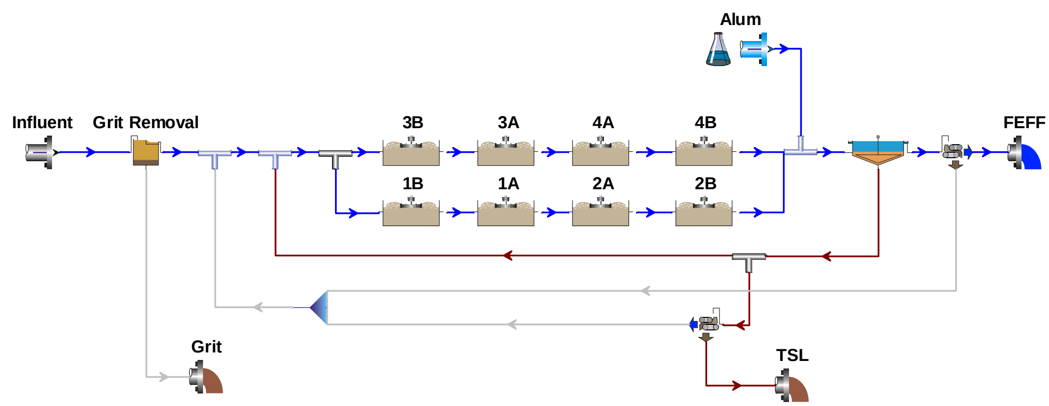


FIGURE 16 - CONFIGURATION NO. 2
ALL AERATION TANKS IN OPERATION

Sensitivity Analysis General. In order to evaluate the loading capacity of the South Street biological process, the flows to the model were incrementally increased at the current maximum month influent concentrations (resulting in increased loads) and the resulting model response in terms of predicted effluent quality was observed. The model runs were run at minimum month temperatures. While the South Street WWTF's current NPDES permit has a seasonal ammonia requirement, the basis of this analysis is that stable nitrification is desired year round. This need for stable nitrification is based on a number of factors including the economic impact of losing nitrification (i.e. – N credit costs), the possibility that there will be a numerical limit for TN in the future, and the fact that processes drifting in/out of full nitrification are often subject to other process problems (i.e. – settling, whole effluent toxicity, etc).

Existing maximum month loadings, as summarized in Table 8, were used as the baseline condition. It was noted that the minimum month temperature was different depending on whether it was measured at the influent or effluent location, differing by more than 2.5^{o}C under minimum month conditions. Surface aerators, such as those employed by the WWTF, are known to cause evaporative cooling of the mixed liquor relative to influent temperatures. As a result, a minimum month temperature of 8.0 degrees, reflective of the minimum month effluent temperature, was used as the assumed temperature of the aeration tanks for performing the load sensitivity analysis.

Another important factor when performing sensitivity analyses on an existing plant is what target MLSS value is assumed. The final settling tank capacity which is discussed below, allows the WWTF to run at MLSS values considerably higher than the design MLSS. During the evaluation period, MLSS levels averaged 5,300 mg/l, however operation over 6,000 mg/l was not uncommon. The assumed MLSS concentrations are described below.

Configuration 1 (Aeration Tanks No. 3 and 4 Only) - Sensitivity Analysis. Based on the available final settling tank solids loading capacity and to maximize the estimated capacity using Aeration Tanks No. 3 and No. 4, the loading sensitivity analysis was performed assuming an operating MLSS concentration between of 6,000-6,250 mg/l. The load was then increased incrementally within the models influent element by assuming constant concentrations of all parameters, and increasing influent flow. A table of the modeled conditions and predicted results is shown in Table 15.

As indicated in Table 15, as the flow/load increases, there is an increase in predicted solids production and corresponding decrease in SRT. This is to be expected when a fixed aeration tank volume and MLSS are subject to increased organic loading. Typically, the effluent NH_4-N is used as an indicator of the stability of the biological process. The response of a biological process, as indicated by effluent NH_4-N, tends to be exponential in nature with respect to decreasing SRT. As shown in Figure 17, the curve is relatively flat at NH_4-N levels below 0.5 mg/l, then increases rapidly as the SRT is decreased and NH_4-N levels begin to exceed 1.0 mg/l.

The exact SRT where this transition occurs varies with a variety of factors, most notably temperature and should be evaluated on a case by case basis. However as seen in the figure, effluent NH_4-N above 1.0 mg/l indicate that the process is in a region of potential instability and slight changes effecting the SRT can easily result in a full loss of nitrification. As a result, it can be concluded that the biological process is already operating at or slightly in excess of capacity with an effluent NH_4-N of 1.2 mg/l, increasing the probability of a process upset and loss of nitrification.

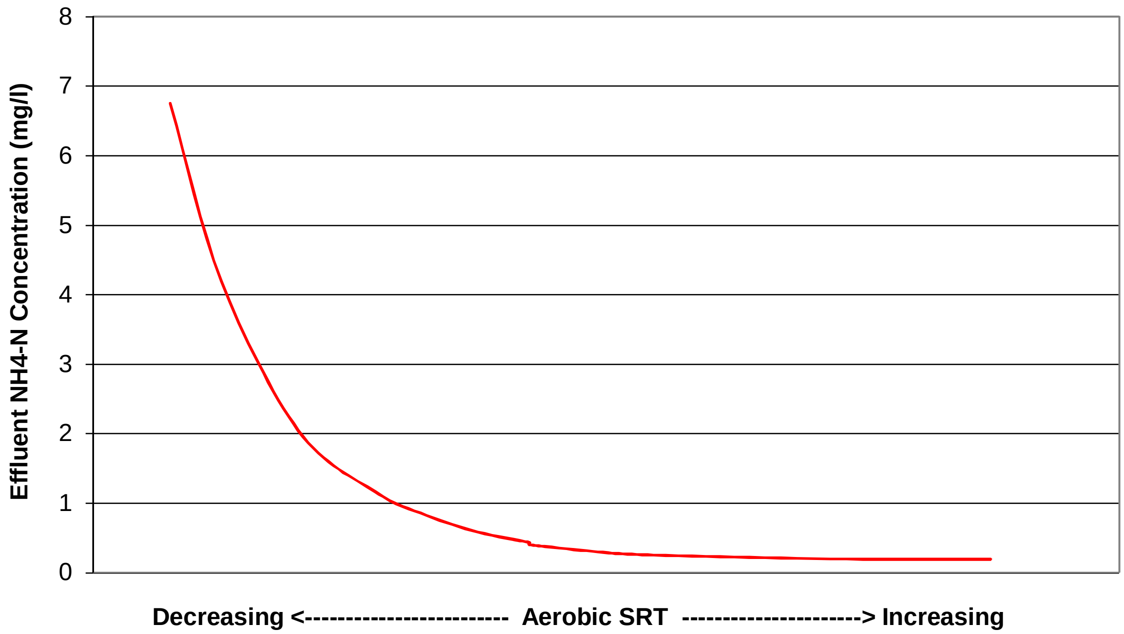
One other observation is that the predicted nitrite nitrogen (NO_2-N) is higher than would be expected. Under normal circumstances, most of the inorganic nitrogen should be in the form of nitrate, or NO_3-N. The presence of significant NO_2-N (NO_2-N of 2.0 mg/l) suggests inadequate oxygen supply. Table 15 shows that the model predicts the first aerobic zone struggling to achieve any measureable DO, and Zone 4-A typically not achieving a value of 2.0 mg/l which would be a typical aerobic target for a nitrifying process. In summary, the biological and aeration capacity of Aeration Tanks No. 3 and No. 4 appear to be inadequate to accommodate any increases in future flows/loads, and under the right combination of conditions, may be inadequate for current flows/loads.

TABLE 15 – CONFIGURATION NO. 1 AERATION TANKS NO. 3 AND NO. 4 IN OPERATION SENSITIVITY ANALYSIS RESULTS

	Baseline (Current Max Month)	10% Increase	15% Increase	20% Increase
Influent [1]				
Q, mgd	1.83	2.02	2.11	2.20
BOD, mg/l	158.0	158.0	158.0	158.0
cBOD, mg/l	132.0	132.0	132.0	132.0
TSS, mg/l	182.0	182.0	182.0	182.0
VSS, mg/l	128.0	128.0	128.0	128.0
TKN, mg/l	16.3	16.3	16.3	16.3
NH4-N, mg/l	10.6	10.6	10.6	10.6
TP, mg/l	3.10	3.10	3.10	3.1
OP, mg/l	1.36	1.36	1.36	1.36
Operating Conditions				
Temp, ^{o}C	8	8	8	8
Flow to Tanks 3 & 4, %	100	100	100	100
Zone 3B DO, mg/l	0	0	0	0
Zone 3A DO, mg/l	0.06	0.04	0.04	0.03
Zone 4A DO, mg/l	1.67	1.28	1.21	1.11
Zone 4B DO, mg/l	2.71	2.08	1.93	1.78
Zone 1B DO, mg/l	-	-	-	-
Zone 1A DO, mg/l	-	-	-	-
Zone 2A DO, mg/l	-	-	-	-
Zone 2B DO, mg/l	-	-	-	-
Final Zone MLSS, mg/l	6,157	6,146	6,030	6,076
RAS Flow, mgd	0.99	0.99	0.99	0.99
RAS/WAS TSS, mg/l	17,948	19,141	19,348	20,070
Solids Loading Rate, lbs/sf-d	22.6 22.6	24.0	24.3	25.2
WAS Flow, mgd	0.00820	0.00881	0.00931	0.00951
WAS TSS, lbs/d	1,228	1,406	1,502	1,591
Aerobic SRT	15.0	13.1	12	11.4
Final Effluent				
cBOD, mg/l	1.0	1.5	1.5	1.5
TSS, mg/l	2.0	2.0	2.0	2.0
NH4-N, mg/l	1.0	2.0	3.0	4.0
NO3-N, mg/l	2.0	1.0	0.5	34
NO2-N, mg/l	2.0	3.0	3.0	2.0
TN, mg/l	7.0	7.5	8.0	8.0
TP, mg/l	0.2	0.2	0.2	0.2

1. Influent concentrations used were calculated current maximum month loading and flow of 1.83 mgd.
2. DO – Dissolved Oxygen, RAS- Return Activated Sludge, WAS- Waste Activated Sludge.

FIGURE 17 - TYPICAL RESPONSE OF EFFLUENT NH_4-N TO AEROBIC SRT



Configuration No. 2 - All Aeration Tanks Online - Sensitivity Analysis. While the WWTF has been operating with only Aeration Tanks No. 3 and No. 4 online during recent years, the original design intent was for all four aeration tanks to be in operation. A second sensitivity analysis was performed with all four aeration tanks in service. This assumes that all installed systems are operable. It should be noted that Aeration Tanks No. 1 and No. 2 are currently not operable and that some of their mechanical and electrical systems require upgrade or repair to allow for use of the tanks. An assessment of the mechanical, electrical and structural condition of the facility will be addressed in the Phase 2 Facilities Plan.

The addition of Aeration Tanks No. 1 and No. 2 to Aeration Tanks No. 3 and No. 4 more than doubles the available aeration tank volume and increases the aeration capacity. The previous model from the Configuration No.1 sensitivity analysis was used and all calibration and wastewater characterization values were left unchanged. Influent flow and RAS were split in between the two process trains (Aeration Tanks No. 3 and 4 and Aeration Tanks No. 1 and 2) in proportion to tank volume. The other major model difference is that the assumed MLSS concentration was reduced to the historical average of approximately 5,300 mg/l. A table of the modeled conditions and predicted results is shown in Table 16.

As can be seen from Table 16, the increased tank volume significantly improves the treatment capacity of the biological process. Even at the reduced MLSS, increases in loading of up to 45% were achievable without exceeding an effluent NH_4-N of 1.0 mg/l. There is some increase in NO_2-N predicted, which suggests that while the biological capacity is sufficient at the 45% increase, the capacity of the aeration system to maintain adequate DO levels is limited.

TABLE 16 – CONFIGURATION NO. 2 ALL FOUR AERATION TANKS IN OPERATION SENSITIVITY ANALYSIS RESULTS

Influent [1]	**Baseline (Current Max Month)**	**15% Increase**	**30% Increase**	**45% Increase**	**60% Increase**
Q, mgd	1.83	2.11	2.38	2.66	2.93
BOD, mg/l	158.0	158.0	158.0	158.0	158.0
cBOD, mg/l	131.8	131.8	131.8	131.8	131.8
TSS, mg/l	182	182	182	182	182
VSS, mg/l	128	128	128	128	128
TKN, mg/l	16.3	16.3	16.3	16.3	16.3
NH4-N, mg/l	10.6	10.6	10.6	10.6	10.6
TP, mg/l	3.1	3.1	3.1	3.1	3.1
OP, mg/l	1.36	1.36	1.36	1.36	1.36
Plant Operating Conditions					
Temp, ^oC	8	8	8	8	8
Flow to Tanks 3 & 4, %	45	45	45	45	45
Zone 3B DO, mg/l	0	0	0	0	0
Zone 3A DO, mg/l	1.16	0.56	0.32	0.2	0.14
Zone 4A DO, mg/l	6.48	5.64	4.88	4.15	3.61
Zone 4B DO, mg/l	7.82	7.2	6.56	5.81	5.13
Zone 1B DO, mg/l	0	0	0	0	0
Zone 1A DO, mg/l	0.14	0.08	0.06	0.04	0.03
Zone 2A DO, mg/l	3.19	2.08	1.29	0.78	0.53
Zone 2B DO, mg/l	5.2	4	2.85	1.81	1.15
Final Stage MLSS, mg/l	5,218	5,261	5,223	5,230	5,183
RAS Flow, mgd	0.99	0.99	0.99	0.99	0.99
RAS/WAS TSS, mg/l	15,204	16,869	18,224	19,781	21,066
Solids Loading Rate, lbs/sf-d	19.1	21.2	23.0	24.9	26.5
WAS Flow, mgd	0.008	0.009	0.010	0.010	0.011
WAS TSS, lbs/d	1,053	1,253	1,460	1,684	1,916
Aerobic SRT	32.7	27.8	23.7	20.5	17.9
Final Effluent					
cBOD, mg/l	1.0	1.0	1.0	1.0	1.0
TSS, mg/l	1.5	1.5	1.5	1.5	1.5
NH4-N, mg/l	0.5	0.5	0.5	0.5	1.0
NO3-N, mg/l	5.0	5.0	5.0	4.0	2.5
NO2-N, mg/l	0.0	0.0	0.0	1.0	2.0
TN, mg/l	7.0	7.0	7.0	7.0	7.0
TP, mg/l	0.2	0.2	0.2	0.2	0.2

1. Influent concentrations used were calculated current maximum month loading and flow of 1.83 mgd.
2. DO – Dissolved Oxygen, RAS- Return Activated Sludge, WAS- Waste Activated Sludge.

Biological Treatment Capacity Summary. Based on the loading sensitivity analyses performed, the South Street WWTF is at or already exceeding its biological capacity at the maximum month loading conditions at 1.83 mgd as it is currently operated with only two Aeration Tanks in operation. If a period of maximum month loadings occurs at the same time as low temperatures, there is a very real risk of a nitrification upset. While it is understood that the WWTF currently has a seasonal nitrification requirement, it is desirable to maintain stable nitrification year round to meet the effluent annual total nitrogen limit included in the Nitrogen General Permit. With all four Aeration Tanks in service, the WWTF appears to have the biological capacity at a maximum month loading conditions between 2.7 and 2.9 mgd assuming similar influent concentrations.

A separate but related limitation appears to be with the aeration system. Both the model and actual plant data under the two aeration tank configuration suggest that the first aerobic zone (Zone 3A) is underaerated. The average DO in this zone ranged from 0.2 to 0.6 during the evaluation period, far less than the 1.5 to 2.0 generally considered optimal for supporting carbon oxidation and nitrification. This can further lead to issues with nitrification stability. Additionally, the plant runs the first zone (3B) in an anoxic state to achieve some degree of denitrification by cycling the aerator in this zone on/off. While there has been a reasonable degree of success with this strategy, allowing solids to settle and stratify in the reactor can lead to problems with the biology by allowing portions of the reactor to go anaerobic in an uncontrolled manner. If the plant wishes to continue operating this zone in an anoxic state, some form of mechanical mixing should be provided. In addition without internal recycle between the aerobic zone and the first anoxic zone the denitrification performance is limited by the return activated sludge rate.

Final Settling Tanks. The plant is configured with two final settling tanks, each 65-feet in diameter and having a sidewater depth of 13 feet. Combined, these two clarifiers provide an effective surface area of 6,640 ft^2. Typically, there are two metrics applied to assess the capacity of secondary clarifiers, the surface overflow rate (SOR) and the solids loading rate (SLR). Because the SLR is measuring the upward velocity as the water column rises and goes over the weirs, it does not include return activated sludge (RAS) flow while the SLR does include the solid associated with both the forward flow and RAS streams.

For the purposes of evaluating the final settling tanks, hydraulic overflow rates of 700 gpd/ft^2 at average day conditions and 1,600 gpd/ft^2 for peak flow conditions were used. Under current conditions the 0.85 mgd average day and 5.88 mgd peak flows the hydraulic overflow rates are 128 gpd/ft^2 and 886 gpd/ft^2, respectively both of which are well within recommended loading rates. At the 1.0 mgd average and 4.1 peak design flows the hydraulic overflow rates are 151 gpd/ft^2 and 618 god/ft^2, respectively both of which are well within recommended loading rates.

For the purposes of evaluating the final settling tanks, solids loading rates of 1.0 lbs/ft^2*hr at average day conditions and 1.6 lbs/ft^2*hr for peak flow conditions were used. Under current conditions for average and peak flows and the MLSS concentration of 5,300 mg/l, the solids loading rates are 0.47 lbs/ft^2*hr and 1.98 lbs/ft^2*hr. Under the peak flow conditions the solids loading rate is higher than the guidelines. However it is not known if the 5.88 mgd peak flow recorded at the WWTF was sustained for the full two hours the guideline is based upon. It is also important to note that these guidelines do not reflect the presence of tertiary filtration (unit process at the WWTF) which provides an added measure of safety against effluent TSS excursions.

At the average and peak design flow design conditions of 1.0 mgd and 4.1 mgd and with a MLSS of 3,500 mg/l, the solids loading rates are 0.37 lbs/ft^2*hr and 0.98 lbs/ft^2*hr, respectively both of which are well within recommended loading rates.

With both final settling tanks in service considering both surface overflow rate and solids loading rates and a MLSS concentration of 5,300 mg/l, the maximum allowable average daily flow is 2.35 mgd and maximum allowable peak hourly flow is 4.50 mgd. The peak flow is below the current 5.88 mgd peak flow recorded. However as noted above it is unclear if this peak flow was sustained for the full two hours that the guideline is based upon and the fact that the guidelines do not reflect the presence of tertiary filtration.

RAS Pumping System. There are three variable frequency drive (VFD) driven return activated sludge (RAS) pumps, each rated for up to 0.95 mgd each. With one assumed to be a standby, there is approximately 1.9 mgd of RAS pumping capacity. The plant currently operates with an average RAS rate of approximately 1.0 mgd. It's not clear if there is one pump running at full capacity to two running throttled, but in either case, there is significant excess RAS capacity for current operations. Most biological nutrient removal processes run at a RAS rate of between 50-100% of average influent flow, so there is sufficient RAS pumping capacity to handle any biological process.

Sand Filters. The plant is configured with six cells of two sand filters each. Each cell has a surface area of 116 ft^2 with a total surface area of 694 ft^2. For the purposes of evaluating the sand filters, loading rates of 1.5 gpm/ft^2 at average day conditions and 5.3 gpm/ft^2 for peak flow conditions were used (WWTF O&M manual loading rates). Under current flow conditions of 0.85 mgd average day and 5.88 peak hour, the loading rates are 0.9 gpm/ft^2 and 5.9 gpm/t^2, respectively. Under the peak flow conditions the loading rate is higher than the sand filter O&M guidelines. However as noted before in the final settling tank section it's not known if the 5.88 mgd peak flow recorded was sustained. Under design conditions for average and peak flows the loading rates are 1.0 gpm/ft^2 and 4.1 gpm/t^2, respectively which are well within the O&M guidelines.

With all six sand filter cells in service, the maximum allowable average daily flow is 1.5 mgd and maximum allowable peak hourly flow is 5.30 mgd. This peak flow is below the current 5.88 mgd peak flow recorded. However as noted above it is unclear if this peak flow was sustained.

Ultraviolet Disinfection. The ultraviolet (UV) disinfection system at the South Street WWTF consists of an open channel Trojan 3000 system that has a single channel with two banks of lamps with 11 rows (modules) of 8 lamps (total of 176 lamps). Based information received from the manufacturer the installed units is rated to provide the design disinfection dose of 30,000 uWs/cm^2 at a maximum flow of 6.2 mgd at 65% ultraviolet transmittance. Based on this information there is sufficient disinfection capacity to handle the current and design average day and peak hourly flows.

Solids Handling.

Waste Sludge Pumps. There are two waste activated sludge (WAS) pumps, each rated for 100 gpm (0.144 mgd). A review of plant operating data indicates that wasting of sludge from the final settling tanks occurs on a discontinuous basis, typically 4-5 days/week (4.2 days/week between 7/1/10 and 6/30/13). The average daily WAS quantity during the evaluation period was approximately 16,200 gal/d, which would require approximately 2-1/2 hours of operation with one pump at 100 gpm. There was one instance (2/11/11) where the recorded wasting for the day was 85,253 gals. It's not known what the circumstances of this instance were, but even at this quantity, this represents only seven hours of operation with two pumps running. Given that many plants waste sludge on a continuous basis, there is sufficient WAS pumping capacity to accommodate current and design conditions.

Sludge Storage. Waste sludge can be pumped to holding tanks for storage prior to thickening/dewatering. There are three storage tanks, with two sized at 42,300 gals/each, and one at 65,000 gals. As previously stated, average WAS flows on days during which wasting occurred were 16,200 gals/d, however when adjusted for a full seven days a week timeframe, this equates to approximately 10,000 gals/d. Based on this, at average current conditions, there are 15-days of storage under current average conditions. Maximum month wasting rates appear to be roughly double average rates, so there is more than seven days of storage under current maximum month conditions. Based on the WWTF's ability to process sludge on a real time basis and with the contingency to settle/decant and ship partially thickened sludge, there is significant excess sludge storage capacity.

Sludge Thickening. For sludge processing, the WWTF is equipped with a combination gravity belt thickener (GBT)/belt filter press (BFP). The plant currently operates in thickening mode only. In thickening mode, this unit has the capability of processing 600 lbs/hr and has a hydraulic capacity of 160 gpm at a WAS concentration of 0.75%. Based on the last three years of data, the WWTF wastes and thickens sludge approximately 4.2 days/week with a feeds solids concentration of 0.93%, a loading rate of 1,250 lbs/day, and a WAS flow rate of 16,200 gal/day. At a solids loading rate of 600 lbs/hr this sludge could be thickened in 2.1 hrs. The design maximum month loading to the thickeners is 15,400 lbs/week at 0.75% solids. If thickening 5 days a week, this results in a loading rate of 3,080 lbs/day, a flow rate of 49,200 gal/day and results in the need to operate the thickener 5.1 hrs/day. The maximum operating time is typically considered 6 hours a day to allow for some start up and shut down time. The 5.1 days is within this guideline. Finally the maximum allowable solids loading to the thickeners was developed based on operating 5 days a week, 6 hours a day and at the current sludge feed concentration of 0.93% solids. Under these conditions the thickener would be able to handle 3,600 lbs/day in 46,400 gallons. 3,600 lbs/day is almost three times as much as is being processed today and 15% more than the design maximum month condition.

Sludge Dewatering. As previously noted, the WWTF is equipped with a combination gravity belt thickener (GBT)/belt filter press (BFP). The plant currently operates in thickening mode only. In dewatering mode, this unit has the capability of processing 750 lbs/hr and has a hydraulic capacity of 50 gpm at a thickened WAS concentration of 3.0%. Assuming a 3.0% feed, dewatering sludge approximately 4.2 days/week, a loading rate of 1,250 lbs/day, a flow rate of 5,000 gal/day and a solids loading rate of 750 lbs/hr this sludge could be dewatered in 1.7 hrs. The design maximum month loading to the dewatering unit is 15,400 lbs/week at 3.0% solids. If dewatering 5 days this results in a loading rate of 3,080 lbs/day, a flow rate of 12,300 gals/day and results in the need to operate the thickener 4.1 hrs/day. The maximum operating time is typically considered 6 hours a day to allow for some start up and shut down time. The 4.1 hours is within this guideline. Finally the maximum allowable solids loading to the dewatering unit was developed based on operating 5 days a week, 6 hours a day and at the a sludge feed concentration of 3.0% solids. Under these conditions the dewatering unit would be able to handle 4,500 lbs/day in 18,000 gallons. 4,500 lbs/day is almost four times as much as is being processed today and 45% more than the design maximum month condition.

Summary. Based on the existing wastewater constituent data, the estimated or calculated constituent removal by unit process, the design data sheets, the mass balances, the Biowin model, and the unit process descriptions above the loading capacity of each unit process at the South Street WWTF was evaluated. Table 17 below presents a summary of the capacity of each unit process.

TABLE 17 – SOUTH STREET WWTF UNIT PROCESS LOADING CAPACITY

Treatment Unit	Unit Process Capacity	Limitation Comment
Grit Chamber	4.1 mgd	Based on vendor information. Grit capture reduced above 4.1 mgd
Aeration Tanks - Two Tanks in Service	Current Max Month Loading 1.83 mgd	Capacity at or in excess of maximum month conditions. All zones in ATs No.3 and No. 4 run in series
Aeration Tanks - Four Tanks in Service	2.7 – 2.9 mgd at current maximum month loading influent concentrations	All zones in ATs No. 3 and No. 4 run in series and all zones in ATs No. 1 and No. 2 run in series
Aerators - Two Tanks in Service	Insufficient aeration capacity in 1st aerobic zone under current average day conditions	All zones in ATs No. 3 and No. 4 run in series
Aerators - Four Tanks in Service	Insufficient aeration capacity in 1st aerobic zone under current maximum month conditions	All zones in ATs No. 3 and No. 4 run in series and all zones in ATs No. 1 and No. 2 run in series
Finial Settling Tanks	Ave Day - 2.35 mgd Peak Hour - 4.5 mgd	MLSS assumed to be 5,300 mg/l similar to current operation
Sand Filters	Ave Day – 1.5 mgd Peak Hour - 5.3 mgd	Based on vendor loading rates
UV Disinfection	6.2 mgd	Based on vendor information.
Solids Handling - Thickening	3 x existing conditions 15% greater than design maximum month conditions	
Solids Handling - Thickening	4 x existing conditions 45% greater than design maximum month conditions	

Unit Process Treatment Capacity Limitation Relief Modifications

Modifications to the WWTF unit processes/components that were identified as limiting capacity to less than the WWTF design average day or peak hour capacities were identified. These modifications are described by unit process below as applicable. An evaluation of these modifications and an assessment of the estimated costs will be performed as applicable as part of the Phase 2 Facilities Planning efforts.

Grit Chamber. The grit chamber was identified as having a peak hourly flow limitation on 4.1 mgd. This limitation is based on the loss of grit removal efficiency at flows greater than 4.1 mgd. This decrease in capture efficiency is significant since the WWTF does not have primary settling tanks and any grit that is not captured in the grit removal system will end up in the aeration tanks that do not have a means to remove the grit. Over time grit entering the aeration tanks can accumulate to the point where it effectively reduces the tank volume available for biological treatment. Modifications to increase the capacity include:

- Replacing the grit chamber removal system with a system with higher capacity.
- Providing a second grit removal system in series with the existing system to capture additional grit that is not captured in the existing system.
- Investigate mechanical system modifications with the manufacture to increase system capacity without structural change (may not be feasible).

Aeration Tanks. Under the current two Aeration Tank operating condition the evaluation identified the unit process as having a capacity limitation at the current maximum month loading conditions. This limitation is based on the potential loss of nitrification and predicting increase in NO_2-N in the effluent. Under the four Aeration Tank operating condition there was sufficient capacity. Modification to increase the capacity of the aeration tanks include:

- Operating the unit process in a four Aeration Tank configuration.
- Modify Aeration Tanks No. 3 and No. 4 process to increase the aeration tank biomass without requiring additional final settling tank capacity such as:
    - Integrated Fixed Film Activated Sludge (IFAS) Process
    - BioMag /BioActiflo Processes (Ballasted activated sludge).
    - Membrane Bioreactor (MBR)
    - Providing separate stage denitrification and run all zones aerobically.

Aerators. The existing aerators currently have insufficient capacity at current loading conditions with two or four aeration tanks on line operating in the current four zones in series configuration. Modifications to increase capacity include:

- Provide larger surface aerators preferably with VFDs and DO monitoring to control aeration.
- Provide fine bubble aeration and blowers with VFDs and DO monitoring to control aeration.
- Provide Invent mixer style mixer/aerators with blowers, VFDs and CO monitoring to control aeration.
- Providing an internal recycle stream to increase denitrification in the first stage and decrease the oxygen demand to the subsequent stages (other process impacts would need to be evaluated).

Final Settling Tanks. The final settling tanks were identified as having a peak hourly flow limitation of 4.5 mgd with the assumed current MLSS concentration of 5,300 mg/l. This 4.5 mgd limitation is based on not exceeding the industry guidelines for solids loading rate. It should be noted that the guidelines do not reflect the presence of tertiary filtration. Modifications to increase the capacity include:

- Construction of an additional final settling tank.
- Reducing the aeration tank MLSS concentration (would need to operate more than two aeration tanks).
- Modify the Aeration Tanks process to maintain or increase the aerobic aeration tank biomass while reducing the MLSS to the final settling tanks such:
    - Use of the Integrated Fixed Film Activated Sludge (IFAS) process
    - Providing separate stage denitrification and run all zones aerobically.
- Modifying the activated sludge process to allow for an increase in the final settling tank solids loading rate such as the use of the BioMag/BioActiflo Processes (ballasted activated sludge).

Sand Filter. The sand filters were identified as having a peak hourly flow limitation on 5.3 mgd based the vendor loading rates. Potential modifications to increase the capacity of the sand filters are noted below. However, consideration of increasing the capacity of the sand filters needs to be considered in conjunction with anticipated new effluent phosphorus limit which will require a tertiary solids removal system such as sand flites (see below for an additional discussion on phosphorus removal). Modifications to increase sand filter capacity include:

- Construction of additional sand filters cells (it is not anticipated that the existing single stage of sand filter will be able to achieve the new total phosphorus limits).
- If the existing system is used in conjunction with another downstream solids separation process then increasing the filter loading rate by either increasing the media size or using the same media size with the recognition that treatment performance would be slightly reduced should be discussed with the sand filter vendor.

Phosphorus Removal. As noted above it is anticipated that the new WWTF NPDES permit for the South Street WWTF when issued will contain a seasonal mass based effluent total phosphorus limit of 1.04 lbs/day. At the current design flow of 1.0 mgd this equates to an effluent total phosphorus concentration of 0.12 mg/l. Any design flow increases to the WWTF would result in a lower effluent concentration. It is not anticipated that the existing unit processes at the WWTF would be able to meet a total phosphorus effluent limit of 0.12 mg/l. The WWTFs current average effluent total phosphorus concentration is 0.21 mg/l. Potential modifications to meet the new total phosphorus limits include:

- Biological Phosphorus Removal Options which would include/require:
    - Aeration Tank Modifications (which would impact current process and capacity)
    - Tertiary chemical phosphorus removal and solids separation processes.
    - Potential solids handling process modifications to prevent anaerobic conditions

- Chemical Phosphorus Removal Options which would include/require:
    - Single or multipoint chemical addition.

- Use of existing final settling tanks for partial phosphorus removal for multipoint chemical addition (tertiary treatment would still be required).
- Membrane Bioreactor for secondary treatment with chemical addition (see the MBR option for increasing the aeration tank capacity above).
- Use of the sand filters as noted in the sand filter section above followed by a second set of sand filters (Dynasand D2 Process) or another tertiary solid process.
- Use of a tertiary solids separation process with or without the sand filters including:
  - Disc Filters
  - Membrane Filters
  - Ballasted flocculation systems (Comag, actiflo)
  - BluePro

It should be noted that any additional chemical added for phosphorus removal will impact the aeration tank capacity. If the chemical area added either upstream of the aeration tanks or upstream of the final settling tanks the additional solids generated will have a direct impact on the aeration tank capacity. If the additional chemicals are added upstream of the sand filters or to another tertiary solids removal process, the additional solids generated will indirectly impact the aeration tanks as the WWTF recycle streams are returned just upstream of the aeration tanks. The impact of any additional chemical phosphorus removal solids will need to be evaluated further as part of the Phase 2 Facilities Plan.

UV Disinfection System. The UV disinfection system was identified as having a peak hourly flow limitation of 6.2 mgd. This limitation is based information obtained for the manufacturer. Modifications to increase the capacity include:

- Replacement of the UV system with a new larger system.
- The addition of a UV system to operate in parallel on in series with the existing system.
- Modification of the existing system to increase the number of lamps or modules (need to confirm with manufacturer).

Nitrogen Removal. As previously noted there is the potential that the South Street WWTF may receive a more stringent total nitrogen limit when the Nitrogen General Permit is reissued and that the CT DEEP Nitrogen General Limit program that allows for purchasing of nitrogen credits may be modified or discontinued. Changes or elimination of the Nitrogen General Permit will have an impact on the WWTF and may require modification to allow the WWTF to improve nitrogen removal. To improve the WWTFs nitrogen removal performance the following modifications could be considered:

- The addition of separate stage denitrification process (would increase the nitrification capacity of the existing aeration tanks but may require the use of supplemental carbon).
- Modifications to the existing aeration tanks to improved provide nitrogen removal (which would impact the loading capacity and hydraulic capacity of the aeration tanks) including:
  - Providing internal recycle pumps for the current aeration tank to operate in a Modified Ludzack-Ettinger (MLE) scenario
  - Modifications to provide a 4 stage Bardenpho process
  - Use of other processes (ex. MBRs, IFAS, etc)

Metals Removal. There is the potential for new or stricter metal limits to be included in the new South Street WWTF NPDES permit. It is anticipated that the South Street WWTF would not be able to meet new or stricter metals limits and as a result WWTF modifications would be required. The alternatives to alternative to evaluate would be dependent upon the numerical limits and metals included in the permit. This evaluation should be performed in conjunction with the chemical phosphorus removal analysis as many of the technologies to improve phosphorus removal have the potential to increase metals removal.

Of particular concern is the effluent limit on zinc in light of the past issues at the South Street WWTF with meeting the monthly average and daily maximum limits in the existing NPDES permit. As noted above in 2009 the US EPA, in conjunction with the CT DEEP, issued an Administrative Enforcement Order (AO) to the Town which required the Town undertake actions to address the levels of total zinc in the plant effluent that had periodically exceeded the permit limits. The Town complied with the requirements of the AO, and in March 2011 submitted a report entitled “Draft Report on the Investigation and Recommended Implementation Program to Achieve Total Zinc Limits of the South Street WWTF” prepared by AECOM and submitted to the EPA and the DEP. The report concluded that the largest source of zinc in the plant influent was from the water supply system, recommended that the Aquarion Water Company be asked to reduce or eliminate the use of a zinc based corrosion inhibitor in the water supply, and if that was not successful at addressing the zinc levels, then a zinc removal upgrade at the WWTF be considered. The zinc removal upgrade would involve construction of chemical storage and feed system for alum and sodium hydroxide as well as a flocculation chamber. Since the 2011 zinc report, the Aquarion Water Company has changed the corrosion inhibitor they have been using, and violations of the effluent zinc limits at the WWTF have become very infrequent. Figure 18 presents the effluent zinc loading and the average monthly zinc effluent loading relative to the maximum day permit limit and the average monthly permit limit showing the infrequent violations. During the initial steps, of the Phase 2 Facilities Plan, feedback from the DEEP will be necessary as to determine whether the zinc limits are to be revised, and if chemical precipitation for zinc will be necessary in the plant upgrade to meet the future effluent limit.

Opinion to Re-Rate the South Street WWTF

Based on results of the hydraulic and loading capacity analysis the potential to “re-rate” the South Street WWTF to a higher capacity was evaluated. Based on the evaluation it does not appear the Route 7 WWTF can be re-rated to a higher capacity.

Hydraulics Limitations. Based on the evaluation there are potentially hydraulic limitations in the sand filter effluent channel (UV influent channel) under current average daily flows. As noted previously, the hydraulic capacity of the sand filters, based on providing three inches between the weirs and the downstream water surface is less than 0.85 mgd. It was noted that the model is overly conservative for the headloss through the downstream UV flow control gate at low flows. Due to the limited hydraulic information on the UV system flow control gate, the water surface upstream of the UV systems (downstream of the sand filter weirs) was assumed to always be the UV manufacturer’s reported maximum water surface. Based on this elevation, there will always be less than 3 inches of freeboard on the sand filter weir. It is anticipated that the UV system installed over 20 years ago will be replaced in any future upgrade or expansion. The limitations on the sand filter weir freeboard should be addressed with the selection and layout of a new UV system and its headloss.

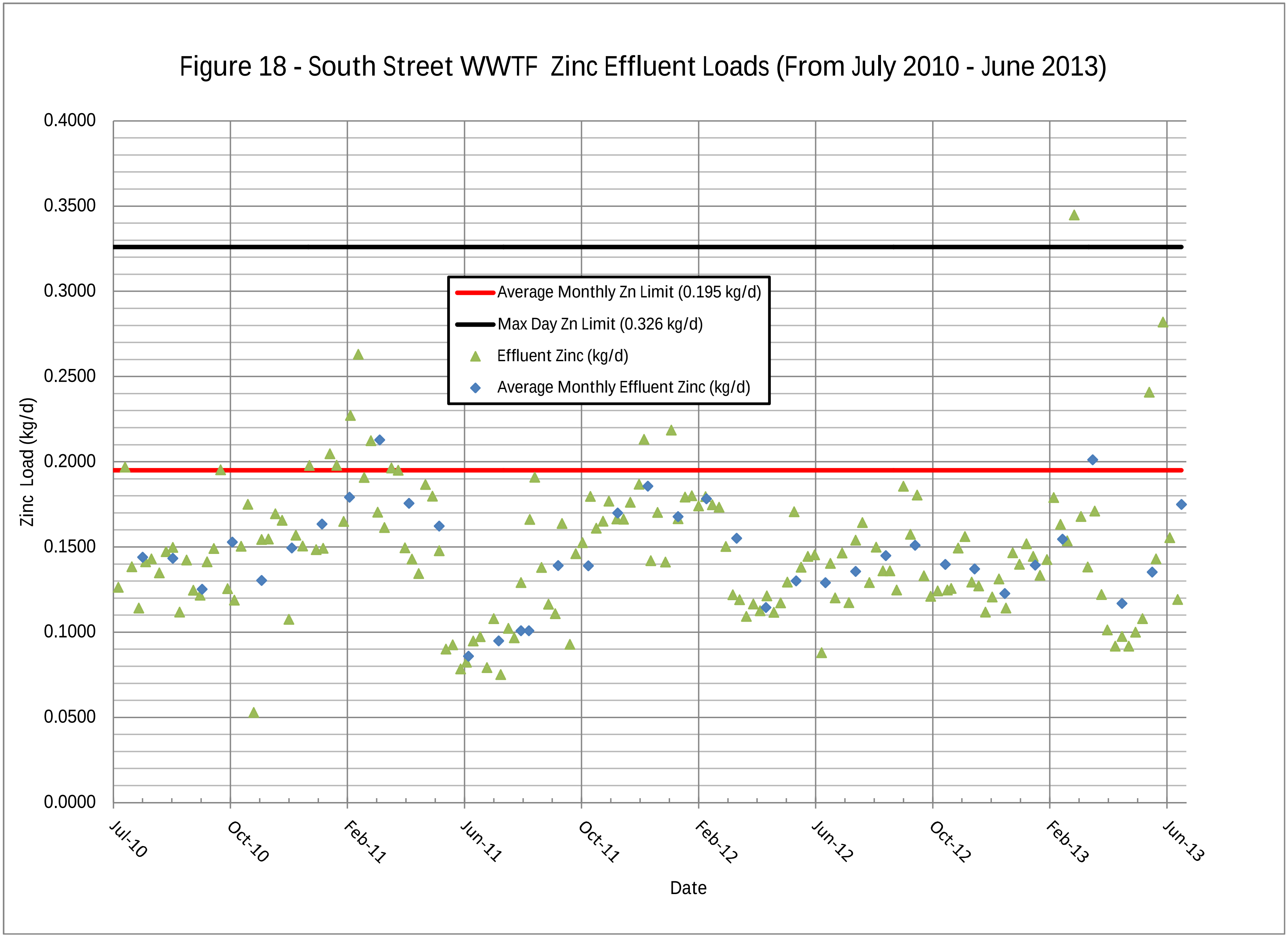


Figure 18 - South Street WWTF Zinc Effluent Loads (From July 2010 - June 2013)
Zinc Load (kg/d)
0.4000
0.3500
0.3000
0.2500
0.2000
0.1500
0.1000
0.0500
0.0000
Average Monthly Zn Limit (0.195 kg/d)
Max Day Zn Limit (0.326 kg/d)
Effluent Zinc (kg/d)
Average Monthly Effluent Zinc (kg/d)
Jul-10
Oct-10
Feb-11
Jun-11
Oct-11
Feb-12
Jun-12
Oct-12
Feb-13
Jun-13
Date

Loading Limitations. Based on the loading capacity analysis it appears that the aeration tanks would limit the ability to re-rate the WWTF. At the current two aeration tank operating condition, the evaluation identified the unit process as having a capacity limitation at the current maximum month loading conditions. This limitation is based on the potential loss of nitrification and predicting increase in NO_2-N in the effluent.

Under the four Aeration Tank operating condition, there was sufficient capacity at current conditions and at loading rates in excess of the 1.0 mgd design flows and loads. However as previously noted Aeration Tanks No. 1 and No. 2 are not operable as some of their mechanical and electrical systems require upgrade or repair to allow for use of the tanks. Since these tanks are not currently operable they cannot be considered in any plant re-rating analysis.

Finally, the last component of the WWTF that would limit the ability of the WWTF to be re-rated is the aeration tank aerators. Both the model and actual plant data under the two aeration tank configuration suggest that the first aerobic zone (Zone 3A) is underaerated. The average DO in this zone ranged from 0.2 to 0.6 during the evaluation period, far less than the 1.5 to 2.0 generally considered optimal for supporting nitrification.

APPENDIX A
ROUTE 7 WWTF MASS BALANCES

Ridgefield Route 7 WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	M. Formica
CHECKED BY	J . Pearson

July 2000 to June 2013
AVERAGE DAY CONDITIONS

ITERATION #			1	2	3	4	5
INFLUENT LOADING							
CURRENT AVERAGE DAILY FLOW (mgd)			0.053	0.053	0.053	0.053	0.053
INFLUENT LOAD							
BOD_5 Concentration (mg/l)			280	280	280	280	280
BOD_5 (lbs/day)			124	124	124	124	124
TSS Concentration (mg/l)			226	226	226	226	226
TSS (lbs/day)			100	100	100	100	100
FSS Concentration (mg/l)			45.2	45.2	45.2	45.2	45.2
FSS (lbs/day)	% of TSS	0.20	20	20	20	20	20
VSS Concentration (mg/l)			180.8	180.8	180.8	180.8	180.8
VSS (lbs/day)	% of TSS	0.80	80	80	80	80	80
TKN Concentration (mg/l)			33.0	33.0	33.0	33.0	33.0
TKN (lbs/day)			15	15	15	15	15
TP Concentration (mg/l)			6.0	6.0	6.0	6.0	6.0
TP (lbs/day)			3	3	3	3	3
PRIMARY SYSTEM							
PRIMARY INF. LOAD (INCLUDES RECYCLE)							
Flow (mgd)			0.053	0.053	0.053	0.053	0.053
BOD_5 Concentration (mg/l)			279	294	295	295	295
BOD_5 (lbs/day)			124	130	131	131	131
TSS Concentration (mg/l)			225	250	252	253	253
TSS (lbs/day)			100	111	112	112	112
FSS Concentration (mg/l)			45	50	50	50	50
FSS (lbs/day)			20	22	22	22	22
VSS Concentration (mg/l)			180	201	202	203	203
VSS (lbs/day)			80	89	90	90	90
TKN Concentration (mg/l)			32.9	33.9	33.9	33.9	33.9
TKN			15	15	15	15	15
TP Concentration (mg/l)			6.0	6.3	6.3	6.3	6.3
TP			2.6	2.8	2.8	2.8	2.8
% PRIMARY REMOVALS							
BOD_5			34	34	34	34	34
TSS			52	52	52	52	52
FSS			52	52	52	52	52
VSS			52	52	52	52	52
TKN			6	6	6	6	6
TP			25	25	25	25	25
PRIMARY EFFLUENT LOAD							
Flow (mgd)			0.053	0.052	0.052	0.052	0.052
BOD_5 Concentration (mg/l)			186.5	196.8	197.4	197.4	197.4
BOD_5 (lbs/day)			82	86	86	86	86
TSS Concentration (mg/l)			109.5	121.8	122.8	122.9	122.9
TSS (lbs/day)			48	53	54	54	54
FSS Concentration (mg/l)			21.9	24.1	24.3	24.3	24.3
FSS (lbs/day)			10	11	11	11	11
VSS Concentration (mg/l)			87.6	97.7	98.5	98.6	98.6
VSS (lbs/day)			38	43	43	43	43
TKN Concentration (mg/l)			31.3	32.3	32.3	32.3	32.3
TKN (lbs/day)			14	14	14	14	14
TP Concentration (mg/l)			4.5	4.8	4.8	4.8	4.8
TP (lbs/day)			2	2	2	2	2
PRIMARY SLUDGE (lbs/day)							
BOD_5			42	44	44	44	44
TSS			52	58	58	58	58
FSS			10	11	12	12	12
VSS			42	46	47	47	47
TKN			0.9	0.9	0.9	0.9	0.9
TP			0.6	0.7	0.7	0.7	0.7
% TS			0.97	0.97	0.97	0.97	0.97
PRIMARY SLUDGE GPD			642	713	719	720	720

Ridgefield Route 7 WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	M. Formica
CHECKED BY	J . Pearson

July 2000 to June 2013
AVERAGE DAY CONDITIONS

ITERATION #			1	2	3	4	5
SECONDARY SYSTEM							
SECONDARY EFFLUENT (lbs/day)							
Flow (mgd)			0.052	0.052	0.052	0.052	0.052
BOD_5 (lbs/day @ 4.2 mg/L)	4.2	mg/l	1.9	1.9	1.9	1.9	1.9
TSS (lbs/day @ 2.6 mg/L)	2.6	mg/l	1.1	1.1	1.1	1.1	1.1
FSS (lbs/day)	0.26	mg/l	0.1	0.1	0.1	0.1	0.1
VSS (lbs/day)	2.34	mg/l	1.0	1.0	1.0	1.0	1.0
TKN (lbs/day @ 2 mg/L)	1.7	mg/l	0.8	0.8	0.8	0.8	0.8
NO_3 (lbs/day)	27	mg/l	11.9	11.9	11.9	11.9	11.9
TP (lbs/day) @5.1 mg/l	5.10	mg/l	2.2	2.2	2.2	2.2	2.2
FSS Removed (lbs/day)			9	10	11	11	11
BOD_5 Removed (lbs/day)			80	84	84	84	84
NH4-N Removed (lbs/day)			13	13	13	13	13
Observed Yield (lbs VSS/lbs BOD5 Removed)			0.60	0.60	0.60	0.60	0.60
Biological Sludge (lbs VSS/ day)			48	51	51	51	51
Projected Yield (lbs VSS/lbs NH4-N Removed)			0.12	0.12	0.12	0.12	0.12
Biological Sludge (lbs VSS/ day)			2	2	2	2	2
Total Biological Sludge (lbs VSS/day)			49	52	52	52	52
TOTAL SLUDGE PRODUCTION (lbsTSS/ day)			59	63	63	63	63
Sludge Concentration (% by weight)			0.84	0.84	0.84	0.84	0.84
WASTE SLUDGE VOLUME (gpd)			841	893	897	897	897
BOD_5 in WSL (lbs/day)	0.50	% of VSS	25	26	26	26	26
TSS in WSL (lbs/day)			59	63	63	63	63
FSS in WSL (lbs/day)			9	10	11	11	11
VSS in WSL (lbs/day)			49	52	52	52	52
TKN in WSL (lbs/day)	0.070	% of VSS	3	4	4	4	4
TP in WSL (lbs/day)	0.015	% of VSS	1	1	1	1	1
SLUDGE STORAGE TANKS							
Feed Solids Flow (gpd)			1,483	1,607	1,616	1,617	1,617
FEED SOLIDS LOAD (lbs/day)							
BOD_5			67	70	71	71	71
TSS			111	120	121	121	121
FSS			20	22	22	22	22
VSS			91	98	99	99	99
TKN			4	5	5	5	5
TP			1	1	1	1	1
% Solids			0.90	0.90	0.90	0.90	0.90
SLUDGE OFF SITE TRUCKING							
Feed Solids Flow (gpd)	90%		1,409	1,526	1,535	1,536	1,536
FEED SOLIDS LOAD (lbs/day)	90%						
BOD_5			60	63	64	64	64
TSS			100	108	109	109	109
FSS			18	20	20	20	20
VSS			82	89	89	89	89
TKN			3.9	4.1	4.1	4.1	4.1
TP			1.2	1.3	1.3	1.3	1.3
% Solids			0.85	0.85	0.85	0.85	0.85
SLUDGE STORAGE TANK SUPERNATANT (lbs/day)							
Supernatant Flow (gpd)	10%		148	161	162	162	162
RECYCLE LOAD (lbs/day)	10%						
BOD_5			6.7	7.0	7.1	7.1	7.1
TSS			11.1	12.0	12.1	12.1	12.1
FSS			2.0	2.2	2.2	2.2	2.2
VSS			9.1	9.8	9.9	9.9	9.9
TKN			0.4	0.5	0.5	0.5	0.5
TP			0.1	0.1	0.1	0.1	0.1
Concentration of solids (% by weight)			0.90	0.90	0.90	0.90	0.90

Ridgefield Route 7 WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	M. Formica
CHECKED BY	J. Pearson

July 2000 to June 2013
MAXIMUM MONTH CONDITIONS

ITERATION #			1	2	3	4	5
INFLUENT LOADING							
CURRENT MAXIMUM MONTH FLOW (mgd)			0.080	0.080	0.080	0.080	0.080
INFLUENT LOAD							
BOD_5 Concentration (mg/l)			425	425	425	425	425
BOD_5 (lbs/day)			284	284	284	284	284
TSS Concentration (mg/l)			303	303	303	303	303
TSS (lbs/day)			202	202	202	202	202
FSS Concentration (mg/l)			60.6	60.6	60.6	60.6	60.6
FSS (lbs/day)	% of TSS	0.20	40	40	40	40	40
VSS Concentration (mg/l)			242.4	242.4	242.4	242.4	242.4
VSS (lbs/day)	% of TSS	0.80	162	162	162	162	162
TKN Concentration (mg/l)			46.2	46.2	46.2	46.2	46.2
TKN (lbs/day)			31	31	31	31	31
TP Concentration (mg/l)			8.5	8.5	8.5	8.5	8.5
TP (lbs/day)			6	6	6	6	6

PRIMARY SYSTEM	1	2	3	4	5
PRIMARY INF. LOAD (INCLUDES RECYCLE)					
Flow (mgd)	0.080	0.080	0.080	0.080	0.080
BOD_5 Concentration (mg/l)	423	446	447	447	447
BOD_5 (lbs/day)	284	299	300	300	300
TSS Concentration (mg/l)	302	337	340	341	341
TSS (lbs/day)	202	226	228	228	228
FSS Concentration (mg/l)	60	66	67	67	67
FSS (lbs/day)	40	44	45	45	45
VSS Concentration (mg/l)	241	271	273	274	274
VSS (lbs/day)	162	182	183	183	183
TKN Concentration (mg/l)	46.0	47.5	47.5	47.5	47.5
TKN	31	32	32	32	32
TP Concentration (mg/l)	8.5	8.9	9.0	9.0	9.0
TP	5.7	6.0	6.0	6.0	6.0
% PRIMARY REMOVALS					
BOD_5	34	34	34	34	34
TSS	52	52	52	52	52
FSS	52	52	52	52	52
VSS	52	52	52	52	52
TKN	6	6	6	6	6
TP	25	25	25	25	25
PRIMARY EFFLUENT LOAD					
Flow (mgd)	0.079	0.079	0.079	0.079	0.079
BOD_5 Concentration (mg/l)	284.0	299.8	300.7	300.7	300.7
BOD_5 (lbs/day)	187	197	198	198	198
TSS Concentration (mg/l)	147.2	164.9	166.4	166.5	166.5
TSS (lbs/day)	97	108	109	110	110
FSS Concentration (mg/l)	29.4	32.4	32.7	32.8	32.8
FSS (lbs/day)	19	21	22	22	22
VSS Concentration (mg/l)	117.8	132.4	133.7	133.7	133.7
VSS (lbs/day)	78	87	88	88	88
TKN Concentration (mg/l)	44.0	45.4	45.5	45.5	45.5
TKN (lbs/day)	29	30	30	30	30
TP Concentration (mg/l)	6.5	6.8	6.8	6.8	6.8
TP (lbs/day)	4	4	4	5	5
PRIMARY SLUDGE (lbs/day)					
BOD_5	96	102	102	102	102
TSS	105	118	119	119	119
FSS	21	23	23	23	23
VSS	84	94	95	95	95
TKN	1.8	1.9	1.9	1.9	1.9
TP	1.3	1.4	1.4	1.4	1.4
% TS	0.97	0.97	0.97	0.97	0.97
PRIMARY SLUDGE GPD	1,299	1,453	1,466	1,467	1,467

Ridgefield Route 7 WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	M. Formica
CHECKED BY	J. Pearson

July 2000 to June 2013
MAXIMUM MONTH CONDITIONS

ITERATION #			1	2	3	4	5
SECONDARY SYSTEM							
SECONDARY EFFLUENT (lbs/day)							
Flow (mgd)			0.077	0.077	0.077	0.077	0.077
BOD_5 (lbs/day @ 4.2 mg/L)	4.2	mg/l	2.8	2.8	2.8	2.8	2.8
TSS (lbs/day @ 2.6 mg/L)	2.6	mg/l	1.7	1.7	1.7	1.7	1.7
FSS (lbs/day)	0.26	mg/l	0.2	0.2	0.2	0.2	0.2
VSS (lbs/day)	2.34	mg/l	1.6	1.6	1.6	1.6	1.6
TKN (lbs/day @ 2 mg/L)	1.7	mg/l	1.1	1.1	1.1	1.1	1.1
NO_3 (lbs/day)	27	mg/l	18.0	18.0	18.0	18.0	18.0
TP (lbs/day) @5.1 mg/l	5.10	mg/l	3.3	3.3	3.3	3.3	3.3
FSS Removed (lbs/day)			19	21	21	21	21
BOD_5 Removed (lbs/day)			184	194	195	195	195
NH4-N Removed (lbs/day)			28	29	29	29	29
Observed Yield (lbs VSS/lbs BOD5 Removed)			0.60	0.60	0.60	0.60	0.60
Biological Sludge (lbs VSS/ day)			111	117	117	117	117
Projected Yield (lbs VSS/lbs NH4-N Removed)			0.12	0.12	0.12	0.12	0.12
Biological Sludge (lbs VSS/ day)			3	3	3	3	3
Total Biological Sludge (lbs VSS/day)			114	120	120	120	120
TOTAL SLUDGE PRODUCTION (lbsTSS/ day)			133	141	142	142	142
Sludge Concentration (% by weight)			0.84	0.84	0.84	0.84	0.84
WASTE SLUDGE VOLUME (gpd)			1,901	2,017	2,025	2,025	2,025
BOD_5 in WSL (lbs/day)	0.50	% of VSS	57	60	60	60	60
TSS in WSL (lbs/day)			133	141	142	142	142
FSS in WSL (lbs/day)			19	21	21	21	21
VSS in WSL (lbs/day)			114	120	120	120	120
TKN in WSL (lbs/day)	0.070	% of VSS	8	8	8	8	8
TP in WSL (lbs/day)	0.015	% of VSS	2	2	2	2	2
SLUDGE STORAGE TANKS							
Feed Solids Flow (gpd)			3,201	3,470	3,490	3,492	3,492
FEED SOLIDS LOAD (lbs/day)							
BOD_5			153	162	162	162	162
TSS			238	259	260	261	261
FSS			40	44	45	45	45
VSS			198	215	216	216	216
TKN			10	10	10	10	10
TP			3	3	3	3	3
% Solids			0.89	0.89	0.89	0.89	0.89
SLUDGE OFF SITE TRUCKING							
Feed Solids Flow (gpd)	90%		3,041	3,296	3,316	3,317	3,317
FEED SOLIDS LOAD (lbs/day)	90%						
BOD_5			138	146	146	146	146
TSS			214	233	234	234	234
FSS			36	40	40	40	40
VSS			178	193	194	194	194
TKN			8.8	9.3	9.3	9.3	9.3
TP			2.8	2.9	2.9	2.9	2.9
% Solids			0.85	0.85	0.85	0.85	0.85
SLUDGE STORAGE TANK SUPERNATANT (lbs/day)							
Supernatant Flow (gpd)	10%		320	347	349	349	349
RECYCLE LOAD (lbs/day)	10%						
BOD_5			15.3	16.2	16.2	16.2	16.2
TSS			23.8	25.9	26.0	26.1	26.1
FSS			4.0	4.4	4.5	4.5	4.5
VSS			19.8	21.5	21.6	21.6	21.6
TKN			1.0	1.0	1.0	1.0	1.0
TP			0.3	0.3	0.3	0.3	0.3
Concentration of solids (% by weight)			0.89	0.89	0.89	0.89	0.89

APPENDIX B
ROUTE 7 WWTF BASIC DESIGN DATA SHEETS

Ridgefield Route 7 WWTF Phase 1 Facilities Plan
Aerated Grit Chamber - Basic Design Data

JOB #	60299267
DATE	11/24/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

AERATED GRIT CHAMBER

Design Criteria	Existing	Design	Allowable Capacity
Flow			
Average Daily Flow, mgd	0.06	0.12	**0.58**
Maximum Hour, mgd	0.08	**0.75**	**0.58**
Maximum Hour, gpm	56	521	403
Detention Time @ Maximum Hour*	3.00	3.00	3.00
Volume Required, gal	167	1,562	1,208
Volume Required, ft^3	22	209	161
Final Design			
Number of Units	1	1	1
Number in Service	1	1	1
Required Unit Volume, ft^3	22	208.76	161
Depth, ft* (triangular chamber depth varies ave depth shown)	5.05	5.05	5.05
Width, ft *	2.50	2.50	2.50
L/W Ratio *	5.06	5.06	5.06
Design Length, ft	12.66	12.66	12.66
Actual Unit Volume, ft^3	160	160	160
Volume in Service, ft^3	160	160	160
Detention Time, min			
Average Daily Flow	28.71	14.36	**2.97**
Maximum Hour	21.53	**2.30**	**2.97**
Air Flow Rate 1.5 - 4.5 cfm/lf			
Minimum Air, cfm	19	19	19
Maximum Air, cfm	57	57	57

Ridgefield Route 7 WWTF Facilities Plan
Primary Settling Tank - Basic Design Data

JOB #	60299267
DATE	11/24/2014
CALC. BY	M. Formica
Checkd By	J Pearson

RECTANGULAR PRIMARY SETTLING TANKS

DESIGN CRITERIA	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE CAPACITY Both Units	ALLOWABLE CAPACITY One Unit
FLOW, MGD					
AVERAGE DAILY	0.05	0.08	0.12	**0.27**	**0.14**
MAXIMUM HR	0.36	0.36	0.75	**1.12**	**0.56**
OVERFLOW RATE GPD/SQ. FT.					
AVERAGE FLOW	600	600	600	600	600
MAXIMUM HR	2500	2500	2500	2500	2500
TOTAL AREA REQD. SQ. FT.					
AVERAGE FLOW	88	132	200	450	225
MAXIMUM HR	144	144	300	448	224
INFLUENT LOADS LBS/D					
BOD	124	172	400	632	294
TSS	102	135	400	520	231
TP	2.7	4.0	12	14	7
TKN	14.7	20.5	40	75	35
FINAL DESIGN DATA					
NUMBER OF TANKS INSTALLED	2	2	2	2	2
NUMBER OF TANKS IN SERVICE	2	2	2	2	1
UNIT AREA REQD. SQ. FT.	72	72	150	224	224
WIDTH FT.	7.0	7.0	7.0	7.0	7.0
CALC. LENGTH FT.	10	10	21	32	32
DESIGN LENGTH FT.	32	32	32	32	32
CHANNEL WIDTH					
# OF CHANNELS PER TANK					
DEPTH FT	8	8	8	8	8
UNIT AREA SQ.FT	224	224	224	224	224
TOTAL AREA SQ. FT	448	448	448	448	224
UNIT VOLUME CFT.	1792	1792	1792	1792	1792
TOTAL VOLUME CFT	3584	3584	3584	3584	1792
TOTAL VOLUME GAL	26808	26808	26808	26808	13404
OVERFLOW RATES GPD/SQFT					
AVERAGE FLOW	118	176	268	**603**	**603**
MAX HR FLOW	804	804	1674	**2500**	**2500**
DETENTION TIME HRS					
AVERAGE FLOW	12.14	8.14	5.36	2.38	2.38
MAX HR FLOW	1.79	1.79	0.86	0.57	0.57

Ridgefield Route 7 WWTF Facilities Plan
Primary Settling Tank - Basic Design Data

% REMOVALS					
BOD	34%	34%	31%	31%	31%
TSS	52%	52%	31%	31%	31%
TP	25%	25%	31%	31%	31%
TKN	6%	6%	38%	38%	38%
EFFLUENT LOADS LBS/D					
BOD	82	114	276	436	203
TSS	49	65	276	359	159
TP	2	3	8	10	5
TKN	14	19	25	47	22
SLUDGE REMOVAL					
AVG DAY LBS/D	53	70	124	161	72
% SOLIDS	0.97	0.97	0.97	0.97	0.97
SLUDGE FLOW GPD	656	868	1534	1991	884
SLUDGE FLOW GPM	0.46	0.60	1.07	1.38	0.61
MAX.DAY LBS/D					
SLUDGE PUMPS					
# INSTALLED	1	1	1	1	1
CAPACITY GPM	30	30	30	30	30

Ridgefield Route 7 WWTF
Rotating Biological Contactors - Basic Design Data

JOB #	60299267
DATE	11/24/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

ROTATING BIOLOGICAL CONTACTORS

DESIGN CRITERIA	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE CAPACITY Both Units	ALLOWABLE CAPACITY One Unit
FLOW MGD					
AVERAGE DAILY	0.05	0.08	0.12	**0.18**	**0.09**
MAXIMUM HOUR	0.30	0.30	0.30	0.30	0.30
BOD CONC. MG/L	185	172	275	185	185
BOD LOAD LBS/D					
AVERAGE DAY	82	114	275	278	139
MAX. DAY	463	431	688	463	463
% SOLUBLE BOD *	50	50	50	50	50
SOLUBLE BOD MG/L	93	86	138	93	93
SOLUBLE BOD LBS/D					
AVERAGE DAY	41	57	138	139	69
MAXIMUM DAY	232	216	344	231	231
NH3 CONC. MG/L	22.0	22.0	25.0	22.0	23.0
NH3 LOAD LBS/D					
AVERAGE DAY	10	14	25	33	17
MAXIMUM DAY	55	55	63	55	58
DESIGN TEMP. C.	13	13	13	13	13
HYDRAULIC LOADING GPD/SQ. FT.					
AVERAGE DAY *	1.37	1.37	1.37	1.37	1.37
MAXIMUM HOUR	1.37	1.37	1.37	1.37	1.37
SOL. BOD LOADING LBS/D/1000 SQ. FT					
AVERAGE DAY *	1.10	1.10	1.10	1.10	1.10
NH4 LOADING LBS/D/1000 SQFT					
AVERAGE DAY *	0.23	0.23	0.23	0.23	0.23
SURFACE AREA REQD. SQFT					
HYD. LOAD BASIS	38,686	57,664	87591	131387	65693
BOD LOAD BASIS	37,200	51,600	125100	126237	63119
TKN LOAD BASIS	43,220	64,422	111200	146784	76728

Ridgefield Route 7 WWTF
Rotating Biological Contactors - Basic Design Data

FINAL DESIGN DATA

NUMBER OF TRAINS	2	2	2	2	2
NUMBER OF SHAFTS / TRAIN	1	1	1	1	1
DIAMETER FT	12	12	12	12	12
LENGTH FT	25	25	25	25	25
LOW DENSITY SHAFTS					
AREA PER SHAFT SQ. FT	100,000	100,000	100000	100000	100000
NUMBER	2	2	2	2	2
HI DENSITY SHAFTS					
AREA PER SHAFT	150,000	150,000	150000	150000	150000
NUMBER	0	0	0	0	0
TOTAL AREA SQ. FT	200,000	200,000	200000	200000	200000
NUMBER IN SERVICE	2	2	2	2	1
AREA IN SERVICE SQ. FT	200,000	200,000	200000	200000	100000
HYDRAULIC LOADING GPD/SQ.FT					
AVERAGE DAY	0.27	0.40	0.60	0.90	0.90
MAXIMUM HOUR	1.50	1.50	1.50	1.50	3.00
BOD LOADING LBS/1000SQ FT					
AVERAGE DAY	0.41	0.57	1.38	1.39	1.39
MAXIMUM DAY	2.32	2.16	3.44	2.31	4.63
SOLUBLE BOD LOADING					
LBS/D/1000 SQFT					
AVERAGE DAY	0.20	0.28	0.69	0.69	0.69
MAXIMUM DAY	1.16	1.08	1.72	1.16	2.31
NH4 LOADING LBS/D/1000SQ FT					
AVERAGE DAY	0.05	0.07	0.13	0.17	0.17
MAXIMUM DAY	0.28	0.28	0.31	0.28	0.58
FIRST STAGE ORGANIC LOADING					
NUMBER OF SHAFTS	2	2	2	2	1
AVAILABLE AREA SQ.FT	50,000	50,000	50,000	50,000	25,000
BOD LOADING LBS/D 1000SQFT					
AVE DAY	1.64	2.27	5.50	**5.55**	**5.55**
MAX DAY	9.26	8.62	13.76	9.26	18.51
SBOD LOADING LBS/D 1000SQFT					
AVE DAY	0.82	1.14	2.75	2.78	2.78
MAX DAY	4.63	4.31	6.88	4.63	9.26
CONNECTED HP *	5	5	5	5	5
OPERATING HP *	5	5	5	5	5
SLUDGE PRODUCTION					
YIELD LBS VSS/LB BOD *	0.6	0.6	0.6	0.6	0.6
LBS/D	46	64	159	158	79
YIELD LBS VSS/LB NH4	0.12	0.12	0.12	0.12	0.12
LBS/D	1.17	1.74	3.00	3.96	2.07
Total Production	47.6	65.9	162.1	161.6	80.9

Ridgefield Route 7 WWTF Facilities Plan
Secondary Settling Tanks - Basic Design Data

JOB #	60299267
DATE	11/24/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

RECTANGULAR SECONDARY SETTLING TANKS

DESIGN CRITERIA	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE Both Units	ALLOWABLE One Unit
FLOW, MGD					
AVERAGE DAILY	0.05	0.08	0.12	**0.16**	**0.08**
MAXIMUM HR	0.36	0.36	0.30	**0.32**	**0.16**
OVERFLOW RATE GPD/SQ. FT.					
AVERAGE FLOW	400	400	400	400	400
MAXIMUM HR	800	800	800	800	800
TOTAL AREA REQD. SQ. FT.					
AVERAGE FLOW	133	198	300	400	200
MAXIMUM HR	450	450	375	400	200
FINAL DESIGN DATA					
NUMBER OF TANKS INSTALLED	2	2	2	2	2
NUMBER OF TANKS IN SERVICE	2	2	2	2	1
UNIT AREA REQD. SQ. FT.	225	225	188	200	200
WIDTH FT.	7.0	7.0	7.0	7.0	7.0
CALC. LENGTH FT.	16	16	13	14	29
DESIGN LENGTH FT.	28	28	28	28	28
DEPTH FT	7	7	7	7	7
UNIT AREA SQ.FT	196	196	196	196	196
TOTAL AREA SQ. FT	392	392	392	392	196
UNIT VOLUME CFT.	1372	1372	1372	1372	1372
TOTAL VOLUME CFT	2744	2744	2744	2744	1372
TOTAL VOLUME GAL	20525	20525	20525	20525	10263
OVERFLOW RATES GPD/SQFT					
AVERAGE FLOW	**135**	**202**	**306**	**408**	**408**
MAX HR FLOW	**918**	**918**	**765**	**816**	**816**
DETENTION TIME HRS					
AVERAGE FLOW	9.29	6.24	4.11	3.08	3.08
MAX HR FLOW	1.37	1.37	1.64	1.54	1.54

Ridgefield Route 7 WWTF Facilities Plan
Ultraviolet Disinfection- Basic Design Data

JOB #	60299267
DATE	11/24/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

ULTRAVIOLET DISINFECTION

Design Criteria	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE
Flow				
Average Flow, mgd	0.05	0.08	0.12	**0.56**
Peak Flow, mgd	0.36	0.36	0.30	**0.56**
Disinfection Limit				
Escherichia coli per 100 ml				
Any Sample	410	410	410	410
30 Day Geometric Mean	126	126	126	126
System Design				
Design Dose, mJ/cm^2	30	30	30	30
UV Transmitance (assumed)	65%	65%	65%	65%
End of Lamp Lifer Factor (assumed)	0.8	0.8	0.8	0.8
Fouling Factor	0.9	0.9	0.9	0.9
Reactor Design				
Number of Channels	1	1	1	1
Number of Banks per Channel	1	1	1	1
Modules	4	4	4	4
Lamps Per Module	2	2	2	2
Total Lamps	8	8	8	8
Estimated Flow per lamp, mgd	0.07	0.07	0.07	0.07
Max flow with available lamps, mgd	0.56	0.56	0.56	0.56

APPENDIX C
SOUTH STREET WWTF MASS BALANCES

Ridgefield South Street WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	J. Reade
CHECKED BY	M. Formica

YEAR 2013
AVERAGE DAY CONDITIONS

ITERATION #			1	2	3	4	5
INFLUENT LOADING							
YEAR 2013 AVERAGE DAY FLOW (mgd)			0.85	0.85	0.85	0.85	0.85
INFLUENT LOAD							
BOD_5 Concentration (mg/l)			218.6	218.6	218.6	218.6	218.6
BOD_5 (lbs/day)			1550	1550	1550	1550	1550
TSS Concentration (mg/l)			232	232	232	232	232
TSS (lbs/day)			1643	1643	1643	1643	1643
FSS Concentration (mg/l)			57.9	57.9	57.9	57.9	57.9
FSS (lbs/day)	% of TSS	0.25	411	411	411	411	411
VSS Concentration (mg/l)			173.8	173.8	173.8	173.8	173.8
VSS (lbs/day)	% of TSS	0.75	1232	1232	1232	1232	1232
TKN Concentration (mg/l)			24.8	24.8	24.8	24.8	24.8
TKN (lbs/day)			176	176	176	176	176
TP Concentration (mg/l)			4.0	4.0	4.0	4.0	4.0
TP (lbs/day)			28	28	28	28	28
GRIT REMOVAL							
Assumed Grit Production, ft3/MG			4	4	4	4	4
Grit Production, lbs/d			340	340	340	340	340
SECONDARY SYSTEM							
SECONDARY INF. LOAD (INCLUDES RECYCLE)							
Flow (mgd)			0.85	0.90	0.90	0.90	0.90
BOD_5 Concentration (mg/l)			218.6	211.2	211.2	211.2	211.2
BOD_5 (lbs/day)			1550	1582	1583	1583	1583
TSS Concentration (mg/l)			183.8	191.1	191.8	191.8	191.8
TSS (lbs/day)			1303	1431	1438	1438	1438
FSS Concentration (mg/l)			10.0	13.4	13.7	13.7	13.7
FSS (lbs/day)			71	100	103	103	103
VSS Concentration (mg/l)			173.8	177.7	178.1	178.1	178.1
VSS (lbs/day)			1232	1331	1335	1335	1335
TKN Concentration (mg/l)			24.8	24.7	24.7	24.7	24.7
TKN			176	185	185	185	185
TP Concentration (mg/l)			4.0	4.4	4.4	4.4	4.4
TP			28	33	33	33	33
SECONDARY EFFLUENT (lbs/day)							
Flow (mgd)			0.85	0.89	0.89	0.89	0.89
BOD_5 (lbs/day @ x mg/L)	5	mg/l	35.4	37.1	37.1	37.1	37.1
TSS (lbs/day @ x mg/L)	15	mg/l	106.3	111.2	111.2	111.2	111.2
FSS (lbs/day)	3.8	mg/l	26.6	27.8	27.8	27.8	27.8
VSS (lbs/day)	11.3	mg/l	79.8	83.4	83.4	83.4	83.4
TKN (lbs/day @ x mg/L)	2.5	mg/l	17.7	18.5	18.5	18.5	18.5
NO_3 (lbs/day @ x mg/L)	4.4	mg/l	31.2	32.6	32.6	32.6	32.6
TP (lbs/day@ x mg/L)	0.60	mg/l	4.3	4.4	4.4	4.4	4.4
FSS Removed (lbs/day)			44	72	75	75	75
BOD_5 Removed (lbs/day)			1515	1545	1546	1546	1546
NH4-N Removed (lbs/day)			158	167	167	167	167
Projected Yield (lbs VSS/lbs BOD5 Removed)			0.40	0.40	0.40	0.40	0.40
Biological Sludge (lbs VSS/ day)			606	618	619	619	619
Projected Yield (lbs VSS/lbs NH4-N Removed)			0.12	0.12	0.12	0.12	0.12
Biological Sludge (lbs VSS/ day)			19	20	20	20	20
Total Biological Sludge (lbs VSS/day)			625	638	639	639	639
Chemical Phosphorus Removal							
Chemical Sludge (lbs Al(OH)3-TSS/day)			9	9	9	9	9
Chemical Sludge (lbs AlPO4-TSS/day)			21	21	21	21	21
Total Chemical Sludge (lbs/TSS/day)			30	30	30	30	30
Final Chemical Sludge (lbs/TSS/day)			41	41	41	41	41
TOTAL SLUDGE PRODUCTION (lbsTSS/ day)			710	751	755	755	755
Sludge Concentration (% by weight)			0.93	0.93	0.93	0.93	0.93

Ridgefield South Street WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	J. Reade
CHECKED BY	M. Formica

YEAR 2013
AVERAGE DAY CONDITIONS

ITERATION #		1	2	3	4	5
WASTE SLUDGE VOLUME (gpd)		9,153	9,688	9,728	9,730	9,730
BOD_5 in WSL (lbs/day)	0.33 % of VSS	206	211	211	211	211
TSS in WSL (lbs/day)		710	751	755	755	755
FSS in WSL (lbs/day)		85	113	116	116	116
VSS in WSL (lbs/day)		625	638	639	639	639
TKN in WSL (lbs/day)	0.070 % of VSS	44	45	45	45	45
TP in WSL (lbs/day)		28	28	28	28	28

DYNASAND FILTERS

INFLUENT LOAD (lbs/day)		1	2	3	4	5
Flow (mgd)		0.85	0.89	0.89	0.89	0.89
BOD_5 (lbs/day @ x mg/L)		35.4	37.1	37.1	37.1	37.1
TSS (lbs/day @ x mg/L)		106.3	111.2	111.2	111.2	111.2
FSS		26.6	27.8	27.8	27.8	27.8
VSS		79.8	83.4	83.4	83.4	83.4
TKN (lbs/day @ x mg/L)		17.7	18.5	18.5	18.5	18.5
NO_3 (lbs/day @ x mg/L)		31.2	32.6	32.6	32.6	32.6
TP (lbs/day@ x mg/L)		4.25	4.45	4.45	4.45	4.45

FILTER EFFLUENT LOAD		1	2	3	4	5
Flow (mgd)		0.81	0.85	0.85	0.85	0.85
BOD_5 Concentration (mg/l)	2 mg/l	2	2	2	2	2
BOD_5 (lbs/day)		13.5	14.1	14.1	14.1	14.1
TSS Concentration (mg/l)	2 mg/l	2	2	2	2	2
TSS (lbs/day)		13.5	14.1	14.1	14.1	14.1
FSS Concentration (mg/l)	0.2 mg/l	0.2	0.2	0.2	0.2	0.2
FSS (lbs/day)		1.3	1.4	1.4	1.4	1.4
VSS Concentration (mg/l)	1.8 mg/l	1.8	1.8	1.8	1.8	1.8
VSS (lbs/day)		12.1	12.7	12.7	12.7	12.7
TKN Concentration (mg/l)	1.6 mg/l	1.6	1.6	1.6	1.6	1.6
TKN (lbs/day)		10.8	11.3	11.3	11.3	11.3
NO3-N Concentration (mg/l)	4.4 mg/l	4.4	4.4	4.4	4.4	4.4
NO3-N (lbs/day)		29.6	31.1	31.1	31.1	31.1
TP Concentration (mg/l)	0.20 mg/l	0.20	0.20	0.20	0.20	0.20
TP (lbs/day)		1.35	1.41	1.41	1.41	1.41

FILTER BACKWASH		1	2	3	4	5
BOD_5 (lbs/day)		22.0	23.0	23.0	23.0	23.0
BOD5 Concentration (mg/l)		62.0	64.8	64.8	64.8	64.8
TSS (lbs/day)		92.9	97.1	97.1	97.1	97.1
TSS Concentration (mg/l)		262	274	274	274	274
FSS (lbs/day)		25.24	26.39	26.39	26.39	26.39
FSS Concentration (mg/l)		71	74	74	74	74
VSS (lbs/day)		67.63	70.71	70.71	70.71	70.71
VSS Concentration (mg/l)		191	199	199	199	199
TKN (lbs/day)		6.95	7.24	7.24	7.24	7.24
TKN Concentration (mg/l)		20	20	20	20	20
NO3-N (lbs/day)		1.6	1.6	1.6	1.6	1.6
NO3-N Concentration (mg/l)		4.4	4.4	4.4	4.4	4.4
TP (lbs/day)		2.91	3.04	3.04	3.04	3.04
TP Concentration (mg/l)		8.2	8.6	8.6	8.6	8.6
SAND FILTER RECYCLE VOLUME (gpd)		42,500	42,500	42,500	42,500	42,500

Ridgefield South Street WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	J. Reade
CHECKED BY	M. Formica

YEAR 2013
AVERAGE DAY CONDITIONS

ITERATION #	1	2	3	4	5
BELT FILTER THICKENING SYSTEM					
FEED SOLIDS (lbs/day)					
BOD_5	206	211	211	211	211
TSS	710	751	755	755	755
FSS	85	113	116	116	116
VSS	625	638	639	639	639
TKN	44	45	45	45	45
TP	28	28	28	28	28
Total feed solids (tons/day)	0.4	0.4	0.4	0.4	0.4
Total feed solids (gals/day)	9,153	9,688	9,728	9,730	9,730
BELT PRESS EFFICIENCY (% CAPTURE)	95	95	95	95	95
MASS OF THICKENED SLUDGE (lbs/day)					
BOD_5	196	200	200	200	200
TSS	674	714	717	717	717
FSS	81	108	110	110	110
VSS	594	606	607	607	607
TKN	42	42	42	42	42
TP	27	27	27	27	27
Concentration of Thickened Sludge (% by weight)	2.4%	2.4%	2.4%	2.4%	2.4%
THICKENED SLUDGE VOLUME (gpd)	3,400	3,400	3,400	3,400	3,400
THICKENED SLUDGE FILTRATE LOAD (lbs/day)					
BOD_5	10	11	11	11	11
BOD5 Concentration (mg/l)	211	198	197	196	196
TSS	35	38	38	38	38
TSS Concentration (mg/l)	727	705	704	704	704
FSS	4	6	6	6	6
FSS Concentration (mg/l)	87	106	108	108	108
VSS	31	32	32	32	32
VSS Concentration (mg/l)	640	599	596	595	595
TKN	2.2	2.2	2.2	2.2	2.2
TKN Concentration (mg/l)	44.8	41.9	41.7	41.7	41.7
TP	1.4	1.4	1.4	1.4	1.4
TP Concentration (mg/l)	29.1	26.5	26.5	26.5	26.5
BELT THICKENER FILTRATE VOLUME (gpd)	5,753	6,288	6,328	6,330	6,330
Washwater Volume (gpd)	100	100	100	100	100
TOTAL FILTRATE VOLUME (gpd)	5,853	6,388	6,428	6,430	6,430
RECYCLES					
TOTAL RECYCLE LOAD (lbs/day)					
BOD_5	32	33	33	33	33
BOD5 Concentration (mg/l)	80	82	82	82	82
TSS	128	135	135	135	135
TSS Concentration (mg/l)	319	331	331	331	331
FSS	29	32	32	32	32
FSS Concentration (mg/l)	73	79	79	79	79
VSS	99	103	103	103	103
VSS Concentration (mg/l)	246	252	252	252	252
TKN	9	9	9	9	9
TKN Concentration (mg/l)	23	23.3	23.3	23.3	23.3
TP	4	4	4	4	4
TP Concentration (mg/l)	10.8	10.9	10.9	10.9	10.9
TOTAL RECYCLE VOLUME (gpd)					
DYNASAND FILTER BACKWASH	42,500	42,500	42,500	42,500	42,500
THICKENER FILTRATE	5,753	6,288	6,328	6,330	6,330
Total:	48,253	48,788	48,828	48,830	48,830

Ridgefield South Street WWTF
Solids Balance

JOB #	60299267
DATE	1/15/2015
CALC. BY	J. Reade
CHECKED BY	M. Formica

YEAR 2013
MAX MONTH CONDITIONS

ITERATION #			1	2	3	4	5
INFLUENT LOADING							
YEAR 2013 MAXIMUM MONTH FLOW (mgd)			1.83	1.83	1.83	1.83	1.83
INFLUENT LOAD							
BOD_5 Concentration (mg/l)			360	360	360	360	360
BOD_5 (lbs/day)			2405	2405	2405	2405	2405
TSS Concentration (mg/l)			182	182	182	182	182
TSS (lbs/day)			2776	2776	2776	2776	2776
FSS Concentration (mg/l)			54.6	54.6	54.6	54.6	54.6
FSS (lbs/day)	% of TSS	0.30	833	833	833	833	833
VSS Concentration (mg/l)			127.3	127.3	127.3	127.3	127.3
VSS (lbs/day)	% of TSS	0.70	1943	1943	1943	1943	1943
TKN Concentration (mg/l)			16.3	16.3	16.3	16.3	16.3
TKN (lbs/day)			249	249	249	249	249
TP Concentration (mg/l)			3.1	3.1	3.1	3.1	3.1
TP (lbs/day)			47	47	47	47	47
GRIT REMOVAL							
Assumed Grit Production, ft3/MG			4	4	4	4	4
Grit Production, lbs/d			732	732	732	732	732
SECONDARY SYSTEM							
SECONDARY INF. LOAD (INCLUDES RECYCLE)							
Flow (mgd)			1.83	1.93	1.93	1.93	1.93
BOD_5 Concentration (mg/l)			157.6	155.2	155.4	155.4	155.4
BOD_5 (lbs/day)			2405	2500	2505	2505	2505
TSS Concentration (mg/l)			133.9	139.9	140.5	140.5	140.5
TSS (lbs/day)			2044	2253	2265	2265	2265
FSS Concentration (mg/l)			6.6	9.8	10.1	10.1	10.1
FSS (lbs/day)			101	157	162	163	163
VSS Concentration (mg/l)			127.3	130.1	130.4	130.4	130.4
VSS (lbs/day)			1943	2096	2103	2103	2103
TKN Concentration (mg/l)			16.3	17.7	17.7	17.7	17.7
TKN			249	284	286	286	286
TP Concentration (mg/l)			3.1	3.1	3.1	3.1	3.1
TP			47	50	50	50	50
SECONDARY EFFLUENT (lbs/day)							
Flow (mgd)			1.83	1.92	1.92	1.92	1.92
BOD_5 (lbs/day @ x mg/L)	10	mg/l	152.6	160.0	160.0	160.0	160.0
TSS (lbs/day @ x mg/L)	15	mg/l	228.9	240.0	240.0	240.0	240.0
FSS (lbs/day)	3.8	mg/l	57.2	60.0	60.0	60.0	60.0
VSS (lbs/day)	11.3	mg/l	171.7	180.0	180.0	180.0	180.0
TKN (lbs/day @ x mg/L)	6	mg/l	91.6	96.0	96.0	96.0	96.0
NO_3 (lbs/day @ x mg/L)	5.8	mg/l	88.5	92.8	92.8	92.8	92.8
TP (lbs/day@ x mg/L)	0.60	mg/l	9.2	9.6	9.6	9.6	9.6
FSS Removed (lbs/day)			44	97	102	103	103
BOD_5 Removed (lbs/day)			2252	2340	2345	2345	2345
NH4-N Removed (lbs/day)			157	188	190	190	190
Projected Yield (lbs VSS/lbs BOD5 Removed)			0.40	0.40	0.40	0.40	0.40
Biological Sludge (lbs VSS/ day)			901	936	938	938	938
Projected Yield (lbs VSS/lbs NH4-N Removed)			0.12	0.12	0.12	0.12	0.12
Biological Sludge (lbs VSS/ day)			19	23	23	23	23
Total Biological Sludge (lbs VSS/day)			920	959	961	961	961
Chemical Phosphorus Removal							
Chemical Sludge (lbs Al(OH)3-TSS/day)			20	20	20	20	20
Chemical Sludge (lbs AlPO4-TSS/day)			45	45	45	45	45
Total Chemical Sludge (lbs/TSS/day)			65	65	65	65	65
Final Chemical Sludge (lbs/TSS/day)			88	88	88	88	88
TOTAL SLUDGE PRODUCTION (lbsTSS/ day)			1052	1144	1151	1151	1152
Sludge Concentration (% by weight)			0.93	0.93	0.93	0.93	0.93

Ridgefield South Street WWTF
Solids Balance

WASTE SLUDGE VOLUME (gpd)			13,557	14,753	14,843	14,846	14,846
BOD_5 in WSL (lbs/day)	0.33	% of VSS	304	316	317	317	317
TSS in WSL (lbs/day)			1052	1144	1151	1151	1152
FSS in WSL (lbs/day)			132	185	191	191	191
VSS in WSL (lbs/day)			920	959	961	961	961
TKN in WSL (lbs/day)	0.070	% of VSS	64	67	67	67	67
TP in WSL (lbs/day)			42	40	40	40	40
DYNASAND FILTERS							
INFLUENT LOAD (lbs/day)							
Flow (mgd)			1.83	1.92	1.92	1.92	1.92
BOD_5 (lbs/day @ x mg/L)			152.6	160.0	160.0	160.0	160.0
TSS (lbs/day @ x mg/L)			228.9	240.0	240.0	240.0	240.0
FSS			57.2	60.0	60.0	60.0	60.0
VSS			171.7	180.0	180.0	180.0	180.0
TKN (lbs/day @ x mg/L)			91.6	96.0	96.0	96.0	96.0
NO_3 (lbs/day @ x mg/L)			88.5	92.8	92.8	92.8	92.8
TP (lbs/day@ x mg/L)			9.16	9.60	9.60	9.60	9.60
FILTER EFFLUENT LOAD							
Flow (mgd)			1.74	1.83	1.83	1.83	1.83
BOD_5 Concentration (mg/l)	5	mg/l	5	5	5	5	5
BOD_5 (lbs/day)			72.5	76.2	76.2	76.2	76.2
TSS Concentration (mg/l)	5	mg/l	5	5	5	5	5
TSS (lbs/day)			72.5	76.2	76.2	76.2	76.2
FSS Concentration (mg/l)	0.5	mg/l	0.5	0.5	0.5	0.5	0.5
FSS (lbs/day)			7.2	7.6	7.6	7.6	7.6
VSS Concentration (mg/l)	4.5	mg/l	4.5	4.5	4.5	4.5	4.5
VSS (lbs/day)			65.2	68.6	68.6	68.6	68.6
TKN Concentration (mg/l)	4.1	mg/l	4.1	4.1	4.1	4.1	4.1
TKN (lbs/day)			59.4	62.5	62.5	62.5	62.5
NO3-N Concentration (mg/l)	5.8	mg/l	5.8	5.8	5.8	5.8	5.8
NO3-N (lbs/day)			84.1	88.4	88.4	88.4	88.4
TP Concentration (mg/l)	0.60	mg/l	0.60	0.60	0.60	0.60	0.60
TP (lbs/day)			8.70	9.14	9.14	9.14	9.14
FILTER BACKWASH							
BOD_5 (lbs/day)			80.1	83.8	83.8	83.8	83.8
BOD5 Concentration (mg/l)			105.0	109.8	109.8	109.8	109.8
TSS (lbs/day)			156.4	163.8	163.8	163.8	163.8
TSS Concentration (mg/l)			205	215	215	215	215
FSS (lbs/day)			49.98	52.37	52.37	52.37	52.37
FSS Concentration (mg/l)			66	69	69	69	69
VSS (lbs/day)			106.45	111.41	111.41	111.41	111.41
VSS Concentration (mg/l)			140	146	146	146	146
TKN (lbs/day)			32.13	33.52	33.52	33.52	33.52
TKN Concentration (mg/l)			42	44	44	44	44
NO3-N (lbs/day)			4.4	4.4	4.4	4.4	4.4
NO3-N Concentration (mg/l)			5.8	5.8	5.8	5.8	5.8
TP (lbs/day)			0.46	0.46	0.46	0.46	0.46
TP Concentration (mg/l)			0.6	0.6	0.6	0.6	0.6
SAND FILTER RECYCLE VOLUME (gpd)			91,500	91,500	91,500	91,500	91,500

BELT FILTER THICKENING SYSTEM

FEED SOLIDS (lbs/day)					
BOD_5	304	316	317	317	317
TSS	1052	1144	1151	1151	1152
FSS	132	185	191	191	191
VSS	920	959	961	961	961
TKN	64	67	67	67	67
TP	42	40	40	40	40
Total feed solids (tons/day)	0.5	0.6	0.6	0.6	0.6
Total feed solids (gals/day)	13,557	14,753	14,843	14,846	14,846
BELT PRESS EFFICIENCY (% CAPTURE)	95	95	95	95	95
MASS OF THICKENED SLUDGE (lbs/day)					
BOD_5	288	301	301	301	301
TSS	999	1087	1094	1094	1094
FSS	125	176	181	181	181
VSS	874	911	913	913	913
TKN	61	64	64	64	64
TP	40	38	38	38	38
Concentration of Thickened Sludge (% by weight)	3.5%	3.5%	3.5%	3.5%	3.5%
THICKENED SLUDGE VOLUME (gpd)	3,400	3,400	3,400	3,400	3,400
THICKENED SLUDGE FILTRATE LOAD (lbs/day)					
BOD_5	15	16	16	16	16
BOD5 Concentration (mg/l)	177	166	165	165	165
TSS	53	57	58	58	58
TSS Concentration (mg/l)	615	599	598	598	598
FSS	7	9	10	10	10
FSS Concentration (mg/l)	77	97	99	99	99
VSS	46	48	48	48	48
VSS Concentration (mg/l)	538	502	499	499	499
TKN	3.2	3.4	3.4	3.4	3.4
TKN Concentration (mg/l)	37.6	35.1	34.9	34.9	34.9
TP	2.1	2.0	2.0	2.0	2.0
TP Concentration (mg/l)	24.6	21.1	20.9	20.9	20.9
BELT THICKENER FILTRATE VOLUME (gpd)	10,157	11,353	11,443	11,446	11,446
Washwater Volume (gpd)	100	100	100	100	100
TOTAL FILTRATE VOLUME (gpd)	10,257	11,453	11,543	11,546	11,546

RECYCLES

TOTAL RECYCLE LOAD (lbs/day)					
BOD_5	95	100	100	100	100
BOD5 Concentration (mg/l)	112	116	116	116	116
TSS	209	221	221	221	221
TSS Concentration (mg/l)	247	258	258	258	258
FSS	57	62	62	62	62
FSS Concentration (mg/l)	67	72	72	72	72
VSS	152	159	159	159	159
VSS Concentration (mg/l)	180	186	186	186	186
TKN	35	37	37	37	37
TKN Concentration (mg/l)	42	43.0	43.0	43.0	43.0
TP	3	2	2	2	2
TP Concentration (mg/l)	3.0	2.9	2.9	2.9	2.9
TOTAL RECYCLE VOLUME (gpd)					
DYNASAND FILTER BACKWASH	91,500	91,500	91,500	91,500	91,500
THICKENER FILTRATE	10,157	11,353	11,443	11,446	11,446
Total:	101,657	102,853	102,943	102,946	102,946

APPENDIX D
SOUTH STREET WWTF BASIS DESIGN DATA SHEETS

Ridgefield South Street WWTF
Final Settling Tanks - Basic Design Data

JOB #	60299267
DATE	1/6/2015
CALC. BY	M. Formica
CHECKED BY	J. Pearson

FINAL SETTLING TANKS

DESIGN CRITERIA	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE Both Units	ALLOWABLE One Unit
Flow, mgd (Q)					
Average Daily	0.85	1.83	1.00	**2.35**	**0.90**
Maximum Hour Flow	5.88	5.88	4.10	**4.50**	**1.64**
OVERFLOW RATE GPD/SQ. FT.					
AVERAGE FLOW	700	700	700	700	700
MAXIMUM HR	1600	1600	1600	1600	1600
SOLIDS LOADING RATE LB/FT^2*HR					
AVERAGE FLOW	1.0	1.0	1.0	1.0	1.0
MAXIMUM HR	1.6	1.6	1.6	1.6	1.6
Final Settling Tank Data					
Number of Tanks Installed	2	2	2	2	2
Number of Tanks In Service	2	2	2	2	1
Tank Diameter, ft	65	65	65	65	65
Depth, ft	13	13	13	13	13
Unit Surface Area, ft^2	3,317	3,317	3,317	3,317	3,317
Total Surface Area, ft^2	6,633	6,633	6,633	6,633	3,317
Unit Volume, ft^3	43,116.13	43,116	43,116	43,116	43,116
Total Volume in Service, ft^3	86,232	86,232	86,232	86,232	43,116
Total Volume in Service, gallons	645,017	645,017	645,017	645,017	322,509
Overflow Rates, gpd/ft^2					
Average Flow	128	276	151	354	271
Maximum Hour Flow	886	886	618	678	494
Detention Time, hours					
Average Flow	18.21	8.46	15.48	6.59	8.60
Maximum Hour Flow	2.63	2.63	3.78	3.44	4.72
Solids Loading Rate					
MLSS, mg/l	5,300	5,300	3,500	5,300	5,300
Average Flow					
Total Flow Ave Day (Q+R), mgd					
R=100% Influent or					
125% average design day	1.70	3.08	2.00	3.60	1.80
MLSS Load, lbs/day	75,143	136,142	58,380	159,127	79,564
Unit Solids Loading, lbs/ft^2/day	11.3	20.5	8.8	24.0	24.0
Unit Solids Loading, lbs/ft^2/hour	0.47	**0.86**	0.37	**1.00**	**1.00**
Peak Flow					
Total Flow Max Hr (Q+R), mgd					
R=100% Influent or					
125% average design day	7.13	7.13	5.35	5.75	2.89
MLSS Load, lbs/day	315,160	315,160	156,167	254,162	127,744
Unit Solids Loading, lbs/ft^2/day	47.5	47.5	23.5	38.3	38.5
Unit Solids Loading, lbs/ft^2/hour	**1.98**	**1.98**	0.98	**1.60**	**1.60**

Ridgefield South Street WWTF
Final Settling Tanks - Basic Design Data

JOB #	60299267
DATE	12/31/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

SAND FILTERS

DESIGN CRITERIA	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE All Units	ALLOWABLE One Cell Out of Sevice
Flow, mgd					
Average Daily	0.85	1.83	1.00	**1.50**	**1.25**
Maximum Hour Flow	**5.88**	**5.88**	4.10	**5.30**	**4.40**
Loading Rate, GPM/FT^2					
Average Day	1.5	1.5	1.5	1.5	1.5
Maximum Flow	5.3	5.3	5.3	5.3	5.3
FINAL DESIGN DATA					
Number of Filter Cells	6	6	6	6	6
Number of Filters per Cell	2	2	2	2	2
Average Cell Depth, ft	17.5	17.5	17.5	17.5	17.5
Width of Cell, ft	8.17	8.17	8.17	8.17	8.17
Length of Cell, ft	14.17	14.17	14.17	14.17	14.17
Unit Area, ft2	116	116	116	116	116
Cells on Line	6	6	6	6	5
Total Area (available), ft^2	694	694	694	694	579
Loading Rate, GPM/FT^2					
Average Daily	0.9	**1.8**	1.0	**1.5**	**1.5**
Maximum Hour Flow	**5.9**	**5.9**	4.1	**5.3**	**5.3**

Ridgefield South Street WWTF
Final Settling Tanks - Basic Design Data

JOB #	60299267
DATE	12/31/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

ULTRAVIOLET DISINFECTION

DESIGN CRITERIA	EXISTING AVERAGE	EXISTING MAX MONTH	DESIGN	ALLOWABLE
Flow				
Average Flow, mgd	0.85	1.83	1.00	
Peak Flow, mgd	5.88	5.88	4.50	**6.20**
Disinfection Limit				
Fecal Colifrom per 100 ml				
7 Day Geometric Mean	400	400	400	400
30 Day Geometric Mean	200	200	200	200
System Design				
Design Dose, mJ/cm^2	30	30	30	30
UV Transmitance	65%	65%	65%	65%
End of Lamp Lifer Factor	0.8	0.8	0.8	0.8
Fouling Factor	0.9	0.9	0.9	0.9
Reactor Design				
Number of Channels	1	1	1	1
Number of Banks per Channel	2	2	2	2
Total Number of Banks	2	2	2	2
Modules per Bank	11	11	11	11
Lamps per Module	8	8	8	8
Total Number of Lamps	176	176	176	176

Per Trojan the max flow to provide a dose of 30 mJ/cm2 is 6.2 mgd

JOB #	60299267
DATE	12/31/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

SLUDGE THICKENING

DESIGN CRITERIA	EXISTING AVERAGE	DESGIN AVERAGE	DESIGN MAX MONTH	ALLOWABLE
Plant Flow, mgd	**0.85**	**1.0**		
Design Criteria	One Unit	One Unit	One Unit	Two Units
Solids Loading, lbs/day	755	1,343	2,200	**2,572**
Solids Loading, lbs/week	5,285	9,400	15,400	**18,001**
Operating Days per Week				
Average	4.2	5	5	5
Design Maxumim	5	5	5	5
THICKENING ONLY				
Solids Loading, lbs/operating day	1,258	1,880	3,080	3,600
Feed Solids, % TS	0.93	0.75	0.75	0.93
Flow Rate, gpd	16,224	30,056	49,241	46,416
Desired Operating Hours per Day	6	6	6	6
Flow Rate, gpm	45.1	83.5	136.8	128.9
Solids Loading, lbs/meter/hr	600	600	600	600
Final Design Data				
Number of Units	1	1	1	1
Number in Service	1	1	1	1
Unit Width, meters	1.0	1.0	1.0	1.0
Total Width, meters	1.0	1.0	1.0	1.0
Width in Service, meters	1.0	1.0	1.0	1.0
Hours of Opertion per Operating Day	2.1	3.1	5.1	**6.0**
Hydraulic Loading				
Typical Loading, gpm/meter	160	160	160	160
Belt Width in Service, meters	1.0	1.0	1.0	1.0
Calculated Loading, gpm/meter	45.1	83.5	136.8	128.9

Ridgefield South Street WWTF
Final Settling Tanks - Basic Design Data

JOB #	60299267
DATE	12/31/2014
CALC. BY	M. Formica
CHECKED BY	J. Pearson

SLUDGE DEWATERING

DESIGN CRITERIA	EXISTING AVERAGE	DESGIN AVERAGE	DESIGN MAX MONTH	ALLOWABLE
Plant Flow, mgd	**0.85**	**1.0**		
Design Criteria	One Unit	One Unit	One Unit	Two Units
Solids Loading, lbs/day	755	1,343	2,200	**3,220**
Solids Loading, lbs/week	5,285	9,400	15,400	**22,540**
Operating Days per Week				
Average	4.2	5	5	5
Design Maxumim	5	5	5	5
DEWATERING ONLY				
Solids Loading, lbs/operating day	1,258.33	1,880	3,080	4,508
Feed Solids, % TS	3.00	3.00	3.00	3.00
Flow Rate, gpd	5,029	7,514	12,310	18,018
Desired Operating Hours per Day	6	6	6	6
Flow Rate, gpm	14.0	20.9	34.2	50.0
Solids Loading, lbs/meter/hr	750	750	750	750
Final Design Data				
Number of Units	1	1	1	1
Number in Service	1	1	1	1
Unit Width, meters	1.0	1.0	1.0	1.0
Total Width, meters	1.0	1.0	1.0	1.0
Width in Service, meters	1.0	1.0	1.0	1.0
Hours of Opertion per Operating Day	1.7	2.5	4.1	6.0
Hydraulic Loading				
Typical Loading, gpm/meter	50	50	50	50
Belt Width in Service, meters	1.0	1.0	1.0	1.0
Calculated Loading, gpm/meter	14.0	20.9	34.2	**50.0**

APPENDIX E

IN PLANT SAMPLING DATA SUMMARY

AECOM
701 Edgewater Drive
Wakefield, MA 01880
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781-246-5200 tel
781-245-6293 fax

Memorandum

To	Ms. Amy Siebert, Chairperson	Page	1
CC	Charles Fischer, Town Engineer; Jorge Pereira, United Water; Jason O'Brien, United Water; Diana Van Ness, WPCA Administrator		
Subject	Ridgefield Phase 1 Wastewater Facilities Plan In Plant Sampling Data Summary		
From	Jon Pearson and Matt Formica		
Date	December 18, 2013		

In Accordance with Task 1.3.4 of our Phase 1 Facilities Plan Scope of Services please find attached herein the summary of the data collected from the "In-Plant Sampling Efforts". The data will be used to supplement the data collected in Task 1 "Project Kickoff, Data Collection and Flow and Loading Data Review" for subsequent projection of future loads (Task 2.4) and analyzing the capacity of the South Street WWTF and the Route 7 WWTF (Task 3.2).

South Street WWTF

The South Street WWTF was sampled for six continuous days from September 23, 2013 to September 28, 2013. Included in Attachment A are daily summary sheets and a weekly average summary sheet of the samples collected by United Water and the laboratory analyses conducted by Con-Test Analytical Laboratory East Longmeadow, MA. Also included on these sheets are some operating data (WWTF flow, septage received, etc) obtained from the WWTF Monthly Operating Reports (MORs). In addition to the laboratory and MOR data presented in Attachment A, Attachment B contains field data that was collected by United Water that is not typically included in the WWTF MORs.

Route 7 WWTF

The Route 7 WWTF was sampled for six continuous days from October 28, 2013 to November 2, 2013. Included in Attachment C are daily summary sheets and a weekly average summary sheet of the samples collected by United Water and the laboratory analyses conducted by Con-Test Analytical Laboratory East Longmeadow, MA. Also included on these sheets are some operating data (WWTF flow, temperature, etc) obtained from the WWTF Monthly Operating Reports (MORs) and some select field data collected by United Water. In addition to the laboratory and MOR data presented in Attachment C, Attachment D contains the field data that was collected by United Water that is not typically included in the WWTF MORs.

ATTACHMENT A

SOUTH STREET WWTF
IN-PLANT SAMPLING (9/23/13 – 9/28/13)
LABORATORY AND MOR DATA
SUMMARY SHEETS

Ridgefield, CT South Street WWTF

Sampling Date: 9/23/2013

	Analyte	Sampling Location									
		Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
Solids	Total Suspended Solids (mg/L)	220	2,900	<5.0	<5.0	5,300	6,000	14	41	5,400	
	% Solids (% Wt)										5.3
	Percent Volatile Solids (%)	86	70			72		36	32	70	75
Organics	Biochemical Oxygen Demand (mg/L)	160	810								
	Carbonaceous BOD (mg/L)	140	280	<4.0	<4.0						
	Carbonaceous BOD, dissolved (mg/L)	34	<30								
	Chemical Oxygen Demand (mg/L & mg/kg)	180	2,300	<15	<15	450		32	<15	600	<28,000
	Chemical Oxygen Demand, soluble (mg/L)	62	53					21	24	190	
	Chemical Oxygen Demand, Floc Filtered (mg/L)	28	40	<15	<15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	28	180	<1.0	<1.0	310		5.80	2.90	260	39,000
	Total Kjeldahl Nitrogen, dissolved (0.45 μm filter) (mg/L)	16	13								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			<1.0	<1.0						
	Ammonia as N (mg/L & mg/kg)	16	11	0.43	<0.20			0.83	0.22	52	5,100
	Nitrate as N (mg/L & mg/kg)	0.1	0.56	4.6	4.9			<0.050	4.8	<0.050	<44
	Nitrite as N (mg/L)			<0.010	0.02						
	TN (calculated) (mg/l & mg/kg)	28	181	5.61	5.92	310		5.85	7.70	260	39,044
	Phosphorus, Total (mg/L & mg/kg)	3.4	7.4	0.21	<0.062			0.73	0.67	9.1	130
	Phosphorus, Total, dissolved (mg/L)	1.6	1.5	<0.062	<0.062						
	Orthophosphate as P (mg/L & mg/kg)	1.5	1.8	0.14	0.05			0.42	0.07	13	310
Miscelaneous	Alkalinity (mg/L & mg/kg)	210	190	120	110			140	110	380	<1,700

*Thickened Sludge analyzed as a solid matrix

Flow Data	Flow (gallons/day)
Plant Flow	760,000
Return Sludge	1,000,000
WAS	16,222
Sludge Shipped	-
Septage Received	10,500

	Analyte				
Location	MLSS (mg/L)	SVI (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
Influent	N/A	N/A	N/A	N/A	66.7
Tank 3A	5,747	87	1.6	0.5	N/A
Tank 3B	9,224	76	0.8	0.2	N/A
Tank 4A	5,912	71	5.9	2.7	N/A
Tank 4B	5,848	82	7.7	4.7	N/A

Ridgefield, CT South Street WWTF

Sampling Date: 9/24/2013

		Sampling Location									
	Analyte	Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
Solids	Total Suspended Solids (mg/L)	300	2,400	<5.0	<5.0	5,000	7,200	90	35	12,000	
	% Solids (% Wt)										6.3
	Percent Volatile Solids (%)	59	72			73		32	31	74	75
Organics	Biochemical Oxygen Demand (mg/L)	220	750								
	Carbonaceous BOD (mg/L)	170	470	<4.0	<4.0						
	Carbonaceous BOD, dissolved (mg/L)	<30	<30								
	Chemical Oxygen Demand (mg/L & mg/kg)	230	3,600	24	18	1,000		140	48	7,000	<1,600
	Chemical Oxygen Demand, soluble (mg/L)	74	42					30	24	510	
	Chemical Oxygen Demand, Floc Filtered (mg/L)	19	22	<15	<15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	33	200	<1.0	<1.0	260		6.1	2.1	230	1,800
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	19	13								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			<1.0	<1.0						
	Ammonia as N (mg/L & mg/kg)	17	12	0.47	0.24			1.0	<0.20	73	41
	Nitrate as N (mg/L & mg/kg)	0.07	0.46	4.2	5.2			<0.050	4.4	1.3	<2.5
	Nitrite as N (mg/L)			0.02	0.01						
	TN (calculated) (mg/l & mg/kg)	33	200	4.2	6.2	260		6.2	6.5	231	1,803
	Phosphorus, Total (mg/L & mg/kg)	2.4	7.7	0.25	0.07			0.51	0.6	13	6.2
	Phosphorus, Total, dissolved (mg/L)	2.1	1.6	<0.062	<0.062						
	Orthophosphate as P (mg/L & mg/kg)	1.6	1.7	<0.050	<0.050			0.21	0.38	37	5.2
Miscelaneous	Alkalinity (mg/L & mg/kg)	220	200	110	110			140	110	330	<100

*Thickened Sludge analyzed as a solid matrix

Flow Data	Flow (gallons/day)
Plant Flow	730,000
Return Sludge	940,000
WAS	16,149
Sludge Shipped	6,500
Septage Received	8,750

	Analyte				
Location	MLSS (mg/L)	SVI (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
Influent	N/A	N/A	N/A	N/A	65.7
Tank 3A	5,513	76	1.9	0.7	N/A
Tank 3B	7,138	76	0.8	0.2	N/A
Tank 4A	5,699	74	6.4	3.3	N/A
Tank 4B	5,749	77	7.6	5.1	N/A

Ridgefield, CT South Street WWTF

Sampling Date: 9/25/2013

	Analyte	Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
		Sampling Location									
Solids	Total Suspended Solids (mg/L)	310	2,400	<5.0	<5.0	4,300	5,400	<25	36	13,000	
	% Solids (% Wt)										6
	Percent Volatile Solids (%)	51	70			75		29	12	77	74
Organics	Biochemical Oxygen Demand (mg/L)	270	210								
	Carbonaceous BOD (mg/L)	250	90	<4.0	<4.0						
	Carbonaceous BOD, dissolved (mg/L)	85	<30								
	Chemical Oxygen Demand (mg/L & mg/kg)	400	290	<15	<15	390		74	23	8,900	<23,000
	Chemical Oxygen Demand, soluble (mg/L)	130	62					27	16	1,800	
	Chemical Oxygen Demand, Floc Filtered (mg/L)	22	31	<15	<15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	35	170	<1.0	<1.0	240		3.70	1.1	470	31,000
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	21	13								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			<1.0	<1.0						
	Ammonia as N (mg/L & mg/kg)	22	14	0.72	0.29			0.99	0.49	160	2,100
	Nitrate as N (mg/L & mg/kg)	0.58	0.77	4.50	5.80			<0.050	4.8	<0.050	<38
	Nitrite as N (mg/L)			0.03	<0.010						
	TN (calculated) (mg/l & mg/kg)	36	171	5.5	6.8	240		3.8	5.9	470	31,038
	Phosphorus, Total (mg/L & mg/kg)	4.0	5.2	0.23	<0.062			0.53	0.55	22	110
	Phosphorus, Total, dissolved (mg/L)	1.8	1.1	<0.062	<0.062						
	Orthophosphate as P (mg/L & mg/kg)	1.9	1.1	<0.050	<0.050			0.26	<0.050	17	140
Miscelaneous	Alkalinity (mg/L & mg/kg)	240	220	110	110			150	120	540	10,000

*Thickened Sludge analyzed as a solid matrix

Flow Data	Flow (gallons/day)
Plant Flow	710,000
Return Sludge	1,190,000
WAS	22,092
Sludge Shipped	6,500
Septage Received	8,500

Location	MLSS (mg/L)	SVI (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
	Analyte				
Influent	N/A	N/A	N/A	N/A	67.5
Tank 3A	5,225	77	2.4	0.5	N/A
Tank 3B	7,017	76	1.0	0.2	N/A
Tank 4A	5,458	70	6.7	4.2	N/A
Tank 4B	5,484	71	7.9	5.8	N/A

Ridgefield, CT South Street WWTF

Sampling Date: 9/26/2013

	Analyte	Sampling Location									
		Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
Solids	Total Suspended Solids (mg/L)	470	3,900	8.00	<5.0	4,800	6,100	23	31	4,200	
	% Solids (% Wt)										5.6
	Percent Volatile Solids (%)	46	70			71		29	41	73	74
Organics	Biochemical Oxygen Demand (mg/L)	350	560								
	Carbonaceous BOD (mg/L)	380	390	<4.0	<4.0						
	Carbonaceous BOD, dissolved (mg/L)	48	<30								
	Chemical Oxygen Demand (mg/L & mg/kg)	290	2,500	29	<15	480		66	51	890	<25,000
	Chemical Oxygen Demand, soluble (mg/L)	80	80					19	19	230	
	Chemical Oxygen Demand, Floc Filtered (mg/L)	43	<15	<15	<15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	42	230	1.10	<1.0	260		3.5	1.90	160	29,000
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	22	11								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			<1.0	<1.0						
	Ammonia as N (mg/L & mg/kg)	20	10	0.40	0.53			0.95	0.50	61	2,900
	Nitrate as N (mg/L & mg/kg)	0.37	0.84	4.2	6.2			<0.050	4.3	0.08	<44
	Nitrite as N (mg/L)			0.03	0.02						
	TN (calculated) (mg/l & mg/kg)	42	231	5.3	7.2	260		3.6	6.2	160	29,044
	Phosphorus, Total (mg/L & mg/kg)	4.5	6.5	0.13	<0.062			0.33	0.53	4.7	66
	Phosphorus, Total, dissolved (mg/L)	1.8	1.2	<0.062	<0.062						
	Orthophosphate as P (mg/L & mg/kg)	2.1	1.1	0.07	0.07			0.17	<0.050	6.5	50
Miscelaneous	Alkalinity (mg/L & mg/kg)	260	200	120	120			140	120	440	6,700

*Thickened Sludge analyzed as a solid matrix

Flow Data	Flow (gallons/day)
Plant Flow	690,000
Return Sludge	880,000
WAS	-
Sludge Shipped	6,500
Septage Received	8,500

	Analyte				
Location	MLSS (mg/L)	SVI (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
Influent	N/A	N/A	N/A	N/A	68.2
Tank 3A	5,022	76	1.1	0.6	N/A
Tank 3B	6,371	75	0.4	0.2	N/A
Tank 4A	4,869	70	5.3	3.7	N/A
Tank 4B	5,138	66	7.2	5.6	N/A

Ridgefield, CT South Street WWTF

Sampling Date: 9/27/2013

		Sampling Location									
	Analyte	Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
Solids	Total Suspended Solids (mg/L)	430	3,600	<5.0	<5.0	4,000	9,000		14	6,300	
	% Solids (% Wt)										
	Percent Volatile Solids (%)	60	70			73			34	79	
Organics	Biochemical Oxygen Demand (mg/L)	240	>600								
	Carbonaceous BOD (mg/L)	200	630	<4.0	<4.0						
	Carbonaceous BOD, dissolved (mg/L)	<30	<30								
	Chemical Oxygen Demand (mg/L & mg/kg)	460	610	20	18	690			26	2,600	
	Chemical Oxygen Demand, soluble (mg/L)	96	290						20	480	
	Chemical Oxygen Demand, Floc Filtered (mg/L)	16	<15	<15	<15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	38	270	1.4	<1.0	220			1.7	200	
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	20	13								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			1.1	2.1						
	Ammonia as N (mg/L & mg/kg)	22	12	0.24	0.33				0.28	73	
	Nitrate as N (mg/L & mg/kg)	0.16	0.12	4.2	5.5				4.5	0.24	
	Nitrite as N (mg/L)			0.02	<0.010						
	TN (calculated) (mg/l & mg/kg)	38	270	5.6	6.5	220			6.2	200	
	Phosphorus, Total (mg/L & mg/kg)	3.9	1.3	0.13	0.07				0.41	17	
	Phosphorus, Total, dissolved (mg/L)	2.2	0.52	0.11	0.06						
	Orthophosphate as P (mg/L & mg/kg)	1.2	1.3	0.13	0.06				<0.050	7.9	
Miscelaneous	Alkalinity (mg/L & mg/kg)	250	220	130	120				120	360	

*Thickened Sludge analyzed as a solid matrix

Flow Data	Flow (gallons/day)
Plant Flow	740,000
Return Sludge	1,200,000
WAS	-
Sludge Shipped	6,500
Septage Received	7,250

	Analyte				
Location	MLSS (mg/L)	SVI (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
Influent	N/A	N/A	N/A	N/A	68
Tank 3A	5,208	67	1.4	0.8	N/A
Tank 3B	6,625	71	0.3	0.2	N/A
Tank 4A	4,882	70	6.7	3.9	N/A
Tank 4B	4,736	72	7.3	5.8	N/A

Ridgefield, CT South Street WWTF

Sampling Date: 9/28/2013

	Analyte	Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
		Sampling Location									
Solids	Total Suspended Solids (mg/L)	260	3,000	7.0	5.0	4,000	8,800		23		
	% Solids (% Wt)										
	Percent Volatile Solids (%)	59	72			73			33		
Organics	Biochemical Oxygen Demand (mg/L)	150	>600								
	Carbonaceous BOD (mg/L)	130	300	<4.0	<4.0						
	Carbonaceous BOD, dissolved (mg/L)	31	<4.0								
	Chemical Oxygen Demand (mg/L & mg/kg)	300	1,100	26	20	1,300			38		
	Chemical Oxygen Demand, soluble (mg/L)	88	240						20		
	Chemical Oxygen Demand, Floc Filtered (mg/L)	22	<15	<15	<15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	33	190	1.10	<1.0	300			1.7		
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	22	23								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			<1.0	<1.0						
	Ammonia as N (mg/L & mg/kg)	16	8	0.37	0.32				0.37		
	Nitrate as N (mg/L & mg/kg)	0.07	0.12	3.2	5.7				3.7		
	Nitrite as N (mg/L)			0.02	<0.010						
	TN (calculated) (mg/l & mg/kg)	33	190	4.32	5.71	300			5.4		
	Phosphorus, Total (mg/L & mg/kg)	4.1	1.1	0.21	0.23				0.42		
	Phosphorus, Total, dissolved (mg/L)	2.6	0.23	0.18	0.22						
	Orthophosphate as P (mg/L & mg/kg)	1.8	1.3	<0.050	<0.050				<0.050		
Miscelaneous	Alkalinity (mg/L & mg/kg)	230	250	130	120				130		

*Thickened Sludge analyzed as a solid matrix

Flow Data	Flow (gallons/day)
Plant Flow	660,000
Return Sludge	1,520,000
WAS	-
Sludge Shipped	-
Septage Received	-

Location	MLSS (mg/L)	SV (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
	Analyte				
Influent	N/A	N/A	N/A	N/A	-
Tank 3A	-	-	-	-	
Tank 3B	-	-	-	-	
Tank 4A	-	-	-	-	
Tank 4B	-	-	-	-	

Ridgefield, CT South Street WWTF

Weekly Average
9/23/2013 - 9/28/2013

	Analyte	Influent Wastewater	Distribution Box Effluent	Secondary Effluent	Final Effluent	Mixed Liquor Effluent	RAS/WAS	Belt Thickener Filtrate	Sand Filter Backwash	Septage	Thickened Sludge* (mg/kg)
Solids	Total Suspended Solids (mg/L)	332	3,033	5.8	5.0	4,567	7,083	38	30	8,180	
	% Solids (% Wt)										6
	Percent Volatile Solids (%)	60	71			73		32	31	75	75
Organics	Biochemical Oxygen Demand (mg/L)	232	588								
	Carbonaceous BOD (mg/L)	212	360	4.0	4.0						
	Carbonaceous BOD, dissolved (mg/L)	43	25								
	Chemical Oxygen Demand (mg/L & mg/kg)	310	1,733	22	17	718		78	34	3,998	19,400
	Chemical Oxygen Demand, soluble (mg/L)	88	128					24	21	642	
	Chemical Oxygen Demand, Floc Filtered (mg/L)	25	23	15	15						
Nutrients	Total Kjeldahl Nitrogen (mg/L & mg/kg)	35	207	1.1	1.0	265		4.8	1.9	264	25,200
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	20	14								
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)			1.0	1.2						
	Ammonia as N (mg/L & mg/kg)	19	11	0	0			1	0	84	2,535
	Nitrate as N (mg/L & mg/kg)	0.22	0.48	4.2	5.6			0.05	4.4	0.34	32
	Nitrite as N (mg/L)			0.02	0.01						
	TN (calculated) (mg/l & mg/kg)	35	207	5.1	6.6	265		4.8	6.3	264	25,232
	Phosphorus, Total (mg/L & mg/kg)	3.7	4.9	0.19	0.09			0.53	0.53	13	78
	Phosphorus, Total, dissolved (mg/L)	2.0	1.0	0.09	0.09						
	Orthophosphate as P (mg/L & mg/kg)	1.7	1.4	0.08	0.06			0.27	0.11	16	126
Miscelaneous	Alkalinity (mg/L & mg/kg)	235	213	120	115			143	118	410	4,625

*Thickened Sludge analyzed as a solid matrix

NOTE: To create averages, any value reported as "<" or ">" was assumed to be equal to that value

Flow Data	Flow (gallons/day)
Plant Flow (6 day average)	715,000
Return Sludge (6 day average)	1,121,667
WAS (3 day average)	18,154
Sludge Shipped (4 day average)	6,500
Septage Received (5 day average)	8,700

	Analyte				
Location	MLSS (mg/L)	SVI (mg/L)	DO Hi (mg/L)	DO Lo (mg/L)	Temp (°F)
Influent	N/A	N/A	N/A	N/A	67
Tank 3A	5,343	77	1.68	0.62	N/A
Tank 3B	7,275	75	0.66	0.20	N/A
Tank 4A	5,364	71	6.2	3.6	N/A
Tank 4B	5,391	74	7.5	5.4	N/A

ATTACHMENT B

SOUTH STREET WWTF
IN-PLANT SAMPLING (9/23/13 – 9/28/13)
FIELD COLLECTED DATA
SUMMARY SHEETS

South Street WWTF Facilities Plan Sampling Effort
Pumped Plant Recycles Flow

Day of the Week	Date	Operating Pump No(s).	Daily Pump Run Time, min	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	9-23-13	1	24 HRS		577275	
Monday	"	2	18 HRS		418910	
Tuesday	9-24-13	1	24 HRS		569100	
Tuesday	"	2	18 HRS		369980	
Wednesday	9-25-13	1	24 HRS		738139	
Wednesday	"	2	18 HRS		447692	
Thursday	9-26-13	1	24 HRS		419960	
Thursday	"	2	24 HRS		462340	
Friday	9-27-13	1	24 HRS		560592	
Friday	"	2	24 HRS		637260	
Saturday	9-28-13	1	24 HRS		808854	
Saturday	"	2	24 HRS		713731	

South Street WWTF Facilities Plan Sampling Effort
Belt Thickener Washwater Flow

Please fill out this form anytime the pumps are operated

Day of the Week	Date	Operating Pump No(s).	Start Time	Stop Time	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	9/23/13	BFP	8:00am	2:00pm	56.7 GPM	16222	
Tuesday	9/24/13	BFP	8:00am	2:00pm	56.7 GPM	16149	
Wednesday	9/25/13	BFP	8:00am	2:00pm	56.7 GPM	22092	
Thursday	—						
Friday	—						
Saturday	—						

South Street WWTF Facilities Plan Sampling Effort Sand Filter Backwash Overflow					
Day of the Week	Date	Number of Filters on Line	Filter Discharge Weir Length, Inches (to the 1/8th if possible)	Water Depth Above the Top of the Weir, Inches (to the 1/8th if possible)	Notes
Monday	9/23/13	5	10" x 5"	1/8"	
Tuesday	9/24/13	5	10" x 5"	1/8"	
Wednesday	9/25/13	5	10" x 5"	1/8"	
Thursday	9/26/13	5	10" x 5"	1/8"	
Friday	9/27/13	5	10" x 5"	1/8"	
Saturday	9/28/13	5	10" x 5"	1/8"	

ATTACHMENT C

ROUTE 7 WWTF
IN-PLANT SAMPLING (10/28/13 – 11/2/13)
LABORATORY AND MOR DATA
SUMMARY SHEETS

Ridgefield, CT Route 7 WWTF

Sampling Date: 10/28/2013

	Analyte	Sampling Location						
		Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	250	100	95	<10	19000	4500	290
	Percent Volatile Solids (%)	48		32				49
Organics	Biochemical Oxygen Demand (mg/L)	330						
	Carbonaceous BOD (mg/L)	89	230		<4.0			
	Carbonaceous BOD, dissolved (mg/L)	82						
	Chemical Oxygen Demand (mg/L)	470	320	51	34			1100
	Chemical Oxygen Demand, soluble (mg/L)	300						600
	Chemical Oxygen Demand, Floc Filtered (mg/L)	68	59		38			
Nutrients	Total Kjeldahl Nitrogen (mg/L)	33	31	6.9	1.6			68
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	17						
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		23		<1.0			
	Ammonia as N (mg/L)	15	20		<0.20			50
	Nitrate as N (mg/L)	0.26	0.24		16			0.65
	Nitrite as N (mg/L)		0.014		0.18			
	TN (calculated) (mg/L)	33	31	6.90	17.78			68.65
	Phosphorus, Total (mg/L)	5.3	4.4		3.8			26
	Phosphorus, Total, dissolved (mg/L)	4.1	6.7		4.2			
	Orthophosphate as P (mg/L)	3.9	3.9		5.2			22
Miscelaneous	Alkalinity (mg/L)	360	380		250			510

Flow Data	Flow (MGD)
Plant Flow*	0.040
Max*	0.115
Min*	0.006

	Analyte	
Location	DO (mg/L)	Temp (°F)
Influent*	N/A	64.0
Primary Effluent	1.47	N/A
RBC Effluent	5.02	N/A
Effluent Discharged*	6.5	63.9

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

Ridgefield, CT Route 7 WWTF

Sampling Date: 10/29/2013

		Sampling Location						
	Analyte	Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	260	140	83	<5.0	11000	9700	490
	Percent Volatile Solids (%)	37		34				50
Organics	Biochemical Oxygen Demand (mg/L)	230						
	Carbonaceous BOD (mg/L)	210	81		<4.0			
	Carbonaceous BOD, dissolved (mg/L)	85						
	Chemical Oxygen Demand (mg/L)	500	310	58	38			1400
	Chemical Oxygen Demand, soluble (mg/L)	180						720
	Chemical Oxygen Demand, Floc Filtered (mg/L)	83	59		20			
Nutrients	Total Kjeldahl Nitrogen (mg/L)	37	31	6.9	1.1			86
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	24						
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		21		<1.0			
	Ammonia as N (mg/L)	27	22		0.28			49
	Nitrate as N (mg/L)	<0.050	0.3		13			0.51
	Nitrite as N (mg/L)		0.012		0.17			
	TN (calculated) (mg/L)	37	31	6.90	14.27			86.51
	Phosphorus, Total (mg/L)	6.1	6		4			25
	Phosphorus, Total, dissolved (mg/L)	4.5	3.8		4.1			
	Orthophosphate as P (mg/L)	5.1	4.3		4.8			23
Miscelaneous	Alkalinity (mg/L)	370	380		250			510

Flow Data	Flow (MGD)
Plant Flow*	0.052
Max*	0.108
Min*	0.013

	Analyte	
Location	DO (mg/L)	Temp (°F)
Influent*	N/A	64.4
Primary Effluent	1.47	N/A
RBC Effluent	6.20	N/A
Effluent Discharged*	6.5	63.9

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

Ridgefield, CT Route 7 WWTF

Sampling Date: 10/30/2013

		Sampling Location						
	Analyte	Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	220	52	52	<5.0	3800	5000	290
	Percent Volatile Solids (%)	42		28				43
Organics	Biochemical Oxygen Demand (mg/L)	190						
	Carbonaceous BOD (mg/L)	180	97		<4.0			
	Carbonaceous BOD, dissolved (mg/L)	55						
	Chemical Oxygen Demand (mg/L)	490	300	65	48			1100
	Chemical Oxygen Demand, soluble (mg/L)	89						440
	Chemical Oxygen Demand, Floc Filtered (mg/L)	65	62		35			
Nutrients	Total Kjeldahl Nitrogen (mg/L)	31	34	10	2.1			65
	Total Kjeldahl Nitrogen, dissolved (0.45 μm filter) (mg/L)	19						
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		23		<1.0			
	Ammonia as N (mg/L)	17	23		0.7			45
	Nitrate as N (mg/L)	0.29	0.15		14			0.14
	Nitrite as N (mg/L)		<0.010		0.39			
	TN (calculated) (mg/L)	31	34	10.00	16.49			65.14
	Phosphorus, Total (mg/L)	6.2	7		4.6			29
	Phosphorus, Total, dissolved (mg/L)	3.5	4.1		4.6			
	Orthophosphate as P (mg/L)	3.8	4.1		4.3			26
Miscelaneous	Alkalinity (mg/L)	370	390		240			500

Flow Data	Flow (MGD)
Plant Flow*	0.040
Max*	0.122
Min*	0.032

	Analyte	
Location	DO (mg/L)	Temp (°F)
Influent*	N/A	64.4
Primary Effluent	1.97	N/A
RBC Effluent	6.80	N/A
Effluent Discharged*	6.7	64.0

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

Ridgefield, CT Route 7 WWTF

Sampling Date: 10/31/2013

	Analyte	Sampling Location						
		Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	180	92	100	<5.0	6800	11000	240
	Percent Volatile Solids (%)	33		41				44
Organics	Biochemical Oxygen Demand (mg/L)	180						
	Carbonaceous BOD (mg/L)	160	89		<4.0			
	Carbonaceous BOD, dissolved (mg/L)	75						
	Chemical Oxygen Demand (mg/L)	430	210	77	48			1100
	Chemical Oxygen Demand, soluble (mg/L)	150						570
	Chemical Oxygen Demand, Floc Filtered (mg/L)	100	47		41			
Nutrients	Total Kjeldahl Nitrogen (mg/L)	35	32	11	1.8			76
	Total Kjeldahl Nitrogen, dissolved (0.45 μm filter) (mg/L)	26						
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		22		1.2			
	Ammonia as N (mg/L)	23	22		0.56			47
	Nitrate as N (mg/L)	0.38	0.45		13			0.51
	Nitrite as N (mg/L)		0.011		0.53			
	TN (calculated) (mg/L)	35	32	11.00	15.33			76.51
	Phosphorus, Total (mg/L)	8.1	5.9		5.6			28
	Phosphorus, Total, dissolved (mg/L)	4	4.5		4.3			
	Orthophosphate as P (mg/L)	4.7	4.2		4.7			25
Miscelaneous	Alkalinity (mg/L)	360	380		250			460

Flow Data	Flow (MGD)
Plant Flow*	0.043
Max*	0.122
Min*	0.009

Location	Analyte	
	DO (mg/L)	Temp (°F)
Influent*	N/A	65.8
Primary Effluent	1.74	N/A
RBC Effluent	6.64	N/A
Effluent Discharged*	6.2	64.2

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

Ridgefield, CT Route 7 WWTF

Sampling Date: 11/1/2013

		Sampling Location						
	Analyte	Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	250	120	100	6	8200	12000	340
	Percent Volatile Solids (%)	33		26				44
Organics	Biochemical Oxygen Demand (mg/L)	230						
	Carbonaceous BOD (mg/L)	190	89		<4.0			
	Carbonaceous BOD, dissolved (mg/L)	75						
	Chemical Oxygen Demand (mg/L)	400	290	250	46			1200
	Chemical Oxygen Demand, soluble (mg/L)	110						400
	Chemical Oxygen Demand, Floc Filtered (mg/L)	110	47		35			
Nutrients	Total Kjeldahl Nitrogen (mg/L)	37	33	11	1.8			74
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	26						
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		25		1.8			
	Ammonia as N (mg/L)	24	22		0.74			50
	Nitrate as N (mg/L)	1.3	0.49		13			0.51
	Nitrite as N (mg/L)		0.014		0.56			
	TN (calculated) (mg/L)	38	34	11.00	15.36			74.51
	Phosphorus, Total (mg/L)	5.5	5.3		5.4			13
	Phosphorus, Total, dissolved (mg/L)	3.8	3.3		4.3			
	Orthophosphate as P (mg/L)	2.8	2		1.4			4.3
Miscelaneous	Alkalinity (mg/L)	370	380		250			460

Flow Data	**Flow (MGD)**
Plant Flow*	No Data Available
Max*	No Data Available
Min*	No Data Available

	Analyte	
Location	DO (mg/L)	Temp (°F)
Influent*	N/A	68.2
Primary Effluent	1.84	N/A
RBC Effluent	6.76	N/A
Effluent Discharged*	5.7	66.4

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

Ridgefield, CT Route 7 WWTF

Sampling Date: 11/2/2013

	Analyte	Sampling Location						
		Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	120	150	88	<10	-	-	-
	Percent Volatile Solids (%)	28		28		-	-	-
Organics	Biochemical Oxygen Demand (mg/L)	130				-	-	-
	Carbonaceous BOD (mg/L)	140	90		<4.0	-	-	-
	Carbonaceous BOD, dissolved (mg/L)	46				-	-	-
	Chemical Oxygen Demand (mg/L)	380	290	130	40	-	-	-
	Chemical Oxygen Demand, soluble (mg/L)	110				-	-	-
	Chemical Oxygen Demand, Floc Filtered (mg/L)	100	59		23	-	-	-
Nutrients	Total Kjeldahl Nitrogen (mg/L)	34	34	5	1.7	-	-	-
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	25				-	-	-
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		22		1.2	-	-	-
	Ammonia as N (mg/L)	23	22			-	-	-
	Nitrate as N (mg/L)	0.3	0.22		15	-	-	-
	Nitrite as N (mg/L)		<0.010		0.98	-	-	-
	TN (calculated) (mg/L)	34	34	5.00	17.68	-	-	-
	Phosphorus, Total (mg/L)	5.9	5.9		5.2	-	-	-
	Phosphorus, Total, dissolved (mg/L)	3.4	4.5		5.1	-	-	-
	Orthophosphate as P (mg/L)	1.3	1.5		1.4	-	-	-
Miscelaneous	Alkalinity (mg/L)	370	390		240	-	-	-

Flow Data	Flow (MGD)
Plant Flow*	No Data Available
Max*	No Data Available
Min*	No Data Available

Location	Analyte	
	DO (mg/L)	Temp (°F)
Influent*	N/A	No Data Available
Primary Effluent	No Data Available	No Data Available
RBC Effluent	No Data Available	No Data Available
Effluent Discharged*	No Data Available	No Data Available

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

Ridgefield, CT Route 7 WWTF

Weekly Average
10/28/2013 - 11/2/2013

	Sampling Location							
	Analyte	Influent Wastewater	Primary Effluent	RBC Effluent	Final Effluent	Primary Sludge	Secondary Sludge	Sludge Holding Tank Supernatant
Solids	Total Suspended Solids (mg/L)	213	109	86.3	6.83	9,760	8,440	330
	Percent Volatile Solids (%)	36.8		31.5				46.0
Organics	Biochemical Oxygen Demand (mg/L)	215						
	Carbonaceous BOD (mg/L)	162	113		4.00			
	Carbonaceous BOD, dissolved (mg/L)	69.7						
	Chemical Oxygen Demand (mg/L)	445	287	105	42.3			1,180
	Chemical Oxygen Demand, soluble (mg/L)	157						546
	Chemical Oxygen Demand, Floc Filtered (mg/L)	87.7	55.5		32.0			
Nutrients	Total Kjeldahl Nitrogen (mg/L)	34.5	32.5	8.47	1.68			73.8
	Total Kjeldahl Nitrogen, dissolved (0.45 µm filter) (mg/L)	22.8						
	Total Kjeldahl Nitrogen, dissolved (GF) (mg/L)		22.7		1.20			
	Ammonia as N (mg/L)	21.5	21.8		0.50			48.2
	Nitrate as N (mg/L)	0.43	0.31		14.0			0.46
	Nitrite as N (mg/L)		0.01		0.47			
	TN (calculated) (mg/L)	34.9	32.8	8.47	16.2			74.3
	Phosphorus, Total (mg/L)	6.18	5.75		4.77			24.2
	Phosphorus, Total, dissolved (mg/L)	3.88	4.48		4.43			
	Orthophosphate as P (mg/L)	3.60	3.33		3.63			20.1
Miscelaneous	Alkalinity (mg/L)	367	383		247			488

NOTE: To create averages, any value reported as "<" or ">" was assumed to be equal to that value.

Flow Data	Flow (MGD)
Plant Flow* (4 Day Average)	0.044
Max* (4 Day Average)	0.117
Min* (4 Day Average)	0.015

	Analyte	
Location	DO (mg/L)	Temp (°F)
Influent* (5 Day Average)	N/A	65.4
Primary Effluent (5 Day Average)	1.7	N/A
RBC Effluent (5 Day Average)	6.3	N/A
Effluent Dishcarged* (5 Day Average)	6.3	64.5

*Values in these rows taken from MORs, all other values from Plant Field Data Sheets

ATTACHMENT D

ROUTE 7 WWTF
IN-PLANT SAMPLING (10/28/13 – 11/2/13)
FIELD COLLECTED DATA
SUMMARY SHEETS

Route 7 WWTF Facilities Plan Sampling Effort
Primary Sludge Flow

Day of the Week	Date	Operating Pump No(s).	Start Time	Stop Time	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	10 28 13	no pump #5	10 13am	10 25a	1100 gAl	40087	
Tuesday	10 29 13	" "	9:59am	10:25	1700 gAl	51991	
Wednesday	10 30 13		9:56	10:14	1100 gAl	40071	
Thursday	10 31 13		9:58	10:10	900 gAl	42543	
Friday	11 1 13		10:02	10:35	1300 gAl		
Saturday							

Route 7 WWTF Facilities Plan Sampling Effort
Secondary Sludge Flow

Day of the Week	Date	Operating Pump No(s).	Start Time	Stop Time	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	102613	NO pump ~~#5~~	10:49	11:10	1000gAl	40087	
Tuesday	102913	" "	10:45	11:10	1000gAl	51991	
Wednesday	103013		10:55	11:11	800 gAl	40071	
Thursday	103113		10:38	11:08	1000gal	42543	
Friday	11 1 13		11:11	11:26	900 gAl		
Saturday							

Route 7 WWTF Facilities Plan Sampling Effort
Primary Scum Flow

Day of the Week	Date	Operating Pump No(s).	Start Time	Stop Time	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	10 28 13	no pump #5	10:28	10:40	300gal	40087	
Tuesday	10 29 13	" "	10:28	10:36	400gal	51991	
Wednesday	10 30 13		10:17	10:23	400 gal	40071	
Thursday	10 31 13		10:13	10:20	300gal	42543	
Friday	11 1 13		10:35	10:47	400gal		
Saturday							

Route 7 WWTF Facilities Plan Sampling Effort
Secondary Scum Flow

Day of the Week	Date	Operating Pump No(s).	Start Time	Stop Time	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	10:28	no pump #5	11:50	12:06	1000gal	40087	
Tuesday	10:29	" "	11:13	11:22	900gal	51991	
Wednesday	103013		11:14	11:23	800gal	40071	
Thursday	103113		11:10	11:29	800gal	42543	
Friday	11113		11:28	11:42	500gal		
Saturday							

Route 7 WWTF Facilities Plan Sampling Effort
Field Process Data

Day of the Week	Date	Dissolved Oxygen, mg/l			Notes
		Primary Effluent	RBC Influent	RBC Effluent	
Monday	mg/l 10-28 13	1.47 mg/L	Same	5.02 mg/L	
Tuesday	10 29 13	1.47	Same	6.20	
Wednesday	10 30 13	1.97	Same	6.80	
Thursday	10 31 13	1.72	Same	6.64	
Friday	11 1 13	1.84	Same	6.76	
Saturday					

Route 7 WWTF Facilities Plan Sampling Effort
Sludge Holding Tanks Supernatant

Day of the Week	Date	Operating Pump No(s).	Start Time	Stop Time	Estimated Discharge Flow Rate (include units (ex. gpm, gpd, etc))	Estimated Total Daily Flow (gpd)	Notes
Monday	102813	No pump #5	11:20	11:32	1100 gal	40087	
Tuesday	102913		10:20	10:38	1200 gal	51991	
Wednesday	10 30 13		10:21	10:38	1200 gal	40071	
Thursday	10 31 13		10:44	11:06	1200 gal	42543	
Friday	11 1 13		10:45	11:08	1100 gal		
Saturday							